GEOTECHNICAL ENGINEERING ASSESSMENT

for proposed repairs/upgrades to the

230kV SECTION

of the

CHUGACH ELECTRIC ASSOCIATION
POINT MACKENZIE SUBSTATION
POINT MACKENZIE, 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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APRIL 2019

April 16, 2019 NGE-TFT Project # 5298-19

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

Attn: Tim Conrad, P.E.

RE: GEOTECHNICAL ENGINEERING ASSESSMENT FOR PROPOSED REPAIRS AND UPGRADES TO EARTHQUAKE-DAMAGED POWER TRANSMISSION EQUIPMENT LOCATED WITHIN THE 230KV SECTION OF THE CEA PT. MACKENZIE ELECTRICAL SUSBSTATION - PT. MACKENZIE, ALASKA

Tim,

We (Northern Geotechnical Engineering, Inc. *d.b.a.* Terra Firma Testing) have completed a geotechnical engineering assessment of the repairs/upgrades planned for the 230kV section of the Chugach Electric Association (CEA) Pt. MacKenzie Substation (PMSS). In general, the native sand and silt deposits which occur across the project site are suitable to support the proposed repairs/upgrades. It is our understanding that both conventional poured-concrete foundations and driven pipe pile foundations may be utilized for the proposed repairs/upgrades. The existing soils are suitable for supporting both types of foundations, however, some of the foundation soils are frost susceptible, and will require provisions in the foundation design to help reduce the potential for frost-related foundation damages. We summarize the findings of our subsurface exploration and laboratory testing programs in the following report, as well as detail our geotechnical engineering conclusions and recommendations; as they pertain to the proposed repairs/upgrades.

In the following report we provide alternatives that can be considered for the design of the planned repairs/upgrades. Our recommendations are intended solely for the development of project specifications and presentation of the geotechnical data, and we should be allowed to review the final design once complete to ensure that our original recommendations still apply.

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

STATE OF ALASKA
49TH
16 April 2019
Keith F. Mobley
CE - 5066
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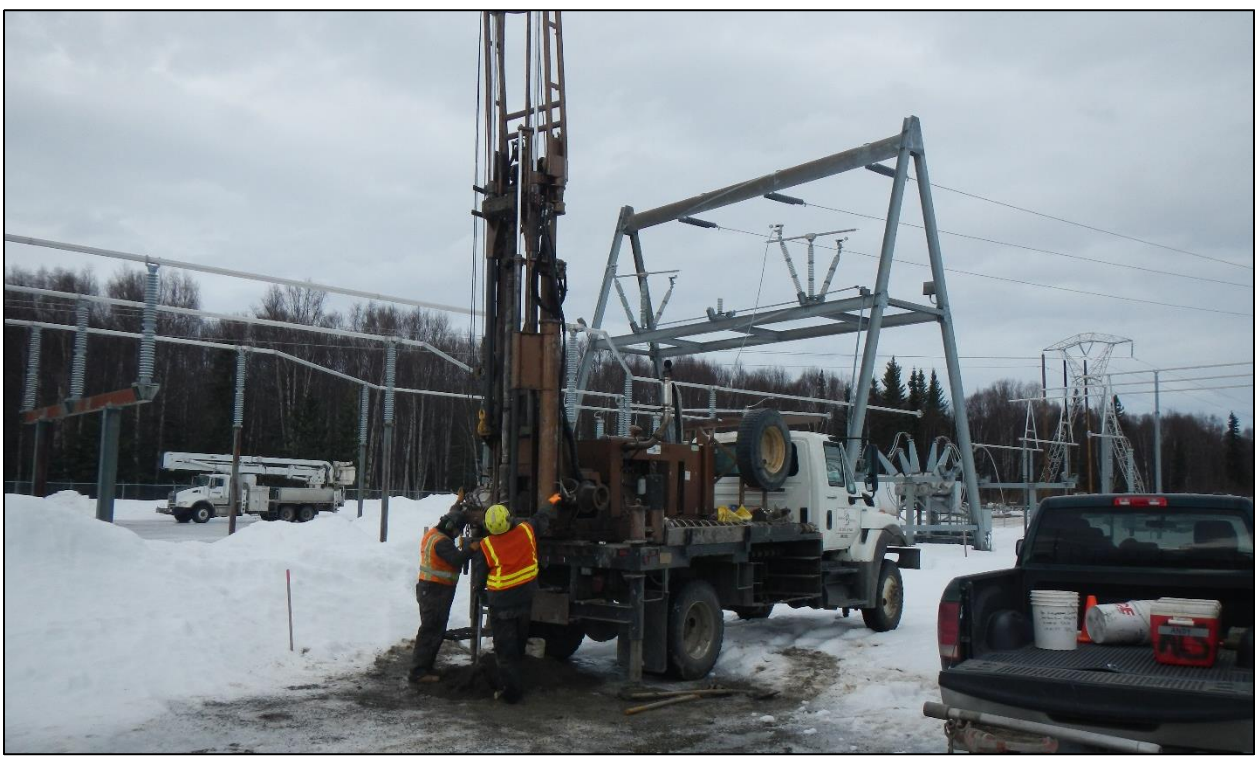

1.0 INTRODUCTION

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 the Chugach Electric Association (CEA) Point MacKenzie Substation (PMSS); which we hereafter refer to as “the project site”. We provided our professional service in accordance with our service fee proposal #19-014.R1 which we submitted to our client, Electric Power Systems (EPS), on January 21, 2019. EPS authorized our proposed scope of service on March 11, 2019 via Purchase Order No. 190111-001.

EPS contracted us to evaluate the subsurface conditions within the 230kV portion of the project site to aid in the design and construction of proposed repairs/upgrades to electrical substation equipment that was damaged during the November 30, 2018 (magnitude 7.1) Anchorage Earthquake. In particular, EPS is interested in assessing the suitability of the existing subgrade to support the proposed repairs/upgrades.

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 proposed repairs/upgrades. We also present our recommendations and alternatives for the design and construction of the proposed repairs/upgrades.

2.0 PROJECT OVERVIEW

As we detail in Figure 1 of this report, the project site is located along the northern shore of Cook Inlet, approximately 1.5 miles west of the geographic Point MacKenzie, Alaska. The legal description of the project site (as we understand it to be) includes portions of Lots A2, A4, A6, and A8, Section 5, T13N, R4W. The project site is approximately seven acres in total area and is the location of the CEA PMSS: an electrical transmission substation consisting of various power transmission equipment (e.g., breakers, buses, etc.) We have included a drawing of the general layout of the PMSS in Figure 2 of this report. The PMSS can be subdivided into two primary sections (a.k.a. yards):

1. The northern 230 kV yard; and
2. The southern 138 kV yard.

Some of the electrical transmission equipment located within the 230 kV yard of the PMSS was damaged during the recent Anchorage Earthquake and CEA is currently in the initial stages of repairing/upgrading the earthquake-damaged equipment.

According to the original design drawings provided to us by CEA, the majority of the existing substation equipment is supported by cold (i.e., unheated) deep foundation systems consisting of driven steel pipe piling ranging in size from approximately 6.625 to 24 inches in diameter and 19

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to 40 feet in overall length. Actual pile embedment depths are not documented on the original CEA construction drawings. EPS and CEA anticipate needing new foundations for some (or all) of the proposed replacement equipment, and driven steel pipe pile foundations will likely be the preferred foundation approach, although some conventional poured-concrete foundations may also be utilized.

3.0 PREVIOUS SUBSURFACE EXPLORATION

DOWL, LLC (formerly DOWL Engineers) conducted a geotechnical soil investigation at the project site in 1984 as part of the original 230kV expansion of the PMSS as detailed in their September 1984 report entitled *SOILS INVESTIGATION FOR CHUGACH ELECTRIC ASSOCIATION POINT MACKENZIE SUBSTATION, POINT MACKENZIE, ALASKA*. As part of their soil investigation, DOWL advanced a total of six soil borings at the project site (five of which are located with the boundaries of the 230kV Yard of the PMSS) to depths ranging from approximately 20 to 30 feet below the existing ground surface (bgs). DOWL also conducted geotechnical laboratory testing of the soil samples that they collected during their subsurface exploration program. We have included copies of the five relevant DOWL borehole logs, along with their associated laboratory test data, in Appendix A of this report for reference. We have also plotted the approximate location of the five relevant DOWL soil borings in Figure 3 of this report. DOWL did not include any engineering conclusions or recommendations in their 1984 report.

4.0 CURRENT SUBSURFACE EXPLORATION

We conceived, coordinated, and directed a subsurface exploration program at the project site in an effort to characterize the subsurface conditions of the project site as they currently exist and to correlate the existing subsurface conditions to those which DOWL identified during their 1984 subsurface soil investigation. We subcontracted Discovery Drilling, Inc (our drilling contractor) to provide the necessary geotechnical exploration services. 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. Under our direction, our drilling contractor advanced a total of two soil borings at the project site on March 20, 2019 to depths ranging from approximately 40 to 45 feet bgs. We have plotted the approximate location of both soil borings in Figure 3 of this report.

Our drilling contractor performed a Standard Penetration Test (SPT) at regular intervals during the drilling of each borehole. A SPT can be used to assess the consistency of a soil interval and to collect representative soil samples. A SPT is performed by driving a 2.0-inch O.D. (1.5-inch I.D.) split-spoon sampler at least 18 inches past the bottom of the advancing augers with blows from a 140-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 standard split-spoon sampler the entire length of each sample interval, or until sampler refusal was encountered. We

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have provided the field blow count data for each sample interval (in six-inch increments) on the graphical borehole logs contained in Appendix B 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 SPT. 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 4 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 B 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.

5.0 LABORATORY TESTING

We collected a total of 21 soil samples from the two soil borings that our drilling contractor advanced at the project site and submitted all of the soil samples that we collected 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); and
- 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

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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 inaccurate gradational and frost classification results. 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.

The laboratory test results, along with the observations that we made during our subsurface exploration efforts (and the 1984 DOWL logs), 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 proposed repairs. We have included the results of our geotechnical laboratory analyses on our graphical exploration logs contained in Appendix B of this report and on our laboratory data sheets contained in Appendix C of this report.

6.0 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 B). 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.

6.1 General Subsurface Profile

In general, the project site is underlain by varying thicknesses of poorly-graded sand deposits which are interbedded with layers of silt and sandy silt (ranging from approximately 2 to 10 feet in thickness). The sand deposits extend to depths of at least 45 feet bgs, have a relatively low fraction of silt (typically less than approximately 15% silt by mass), and generally classify as non-frost susceptible (NFS) to slightly frost susceptible (S2) on the US Army Corps of Engineers (USACOE) frost classification system. Some of the more silt-rich sand interbeds, however, classify as highly frost susceptible (F3) on the USACOE frost classification system. The sand deposits are relatively dense and contain intermittent amounts of gravel (generally less than one inch in diameter) throughout. Some thin, low-grade coal seams also occur within the sand deposits, between depths of approximately 5 to 10 feet bgs. The interbedded silt layers exhibit a relatively stiff consistency, are generally non-plastic, and likely classify as F3 to F4 on the USACOE frost classification system.

6.2 Groundwater

We did not observe any indications of groundwater during our recent subsurface exploration effort. DOWL did, however, observe some perched groundwater at a depth of approximately 10 feet bgs

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across the central portion of the PMSS 230kV Yard; although the perched groundwater appears to be localized, and not laterally extensive across the project site. We expect that the groundwater table likely occurs at depths greater than 45 feet bgs across the entire project site.

6.3 Frozen Soils

We observed some seasonal ground frost during our subsurface exploration effort down to depths of approximately 4 to 5 feet bgs, and seasonal ground frost depths likely vary across the project site depending upon variations in seasonal snow cover. We did not, however, observe any indications of permafrost and we do not expect permafrost to occur anywhere across the project site.

7.0 ENGINEERING CONCLUSIONS

7.1 General Site Conclusions

Based on the findings of DOWL's 1984 soil investigation and our recent subsurface exploration and laboratory testing efforts, it is our conclusion that the native sand and/or silt deposits which underlie the project site are generally suitable to support the proposed repairs; provided that our concerns and recommendations that we present in this report are addressed by the design and construction processes.

7.2 Earthworks

Earthworks conducted in conjunction with the proposed repairs/upgrades should be relatively minimal. Any of the existing sand deposits which are excavated from the project site during construction can be re-used onsite as structural fill assuming that the excavated material is placed using proper placement and compaction techniques. The existing silt-rich soils (>15% silt by mass), however, may not be suitable for re-use as structural fill as they may be difficult to handle and/or properly compact (depending upon their moisture content at the time of placement/compaction). We detail our earthworks recommendations in greater detail in Sections 8.1 and 9.1 of this report.

7.3 Foundations

The equipment that is to be repaired/upgraded (and their associated foundations) will all be exposed to freezing temperatures during winter months, and as such, any cold (i.e., unheated) foundations will need to be designed to resist damage from frost heaving forces and/or any associated thaw-related settlements. This is typically accomplished through the use of deep foundation systems which transfer foundation loads down though the active layer (i.e., the soils which are exposed to seasonal freezing) to continuously-thawed bearing soils, and protect the foundations from seasonal (i.e., frost-related) ground movements.

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It is our understanding that the majority of the proposed repairs/upgrades will likely be founded onto driven steel pipe piling, similar to the existing pipe pile foundations. Driven steel pipe piling are the most common type of deep foundation materials utilized in Alaska, however, driven steel "H" piling, helical piers, and drilled concrete shafts also serve as suitable alternatives.

We also understand that more conventional, poured-concrete foundations may also be used in certain applications at the project site. Any poured concrete foundation constructed at the project site (which is not enclosed in a continuously heated space) will need to be buried below the active layer (or be adequately insulated) so that it is not subjected to any 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 8.2/8.3 and 9.2/9.3 of this report.

7.4 Foundation Settlement

Settlements for conventional poured-concrete foundations should be within tolerable limits, provided that they are placed directly onto the undisturbed (continuously-thawed) sand and/or silt deposits (or properly placed NFS structural fill located directly above the undisturbed sand and/or silt soils). We anticipate a total settlement for shallow concrete foundations placed on either the undisturbed (continuously-thawed) sand and/or silt soils and/or or NFS structural fill placed above the undisturbed sand and/or silt soils (as we discuss in Section 8.2 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 and/or if the compression of any sub-foundation insulation is not accounted for by the foundation design (See Section 8.2.5 for more details regarding insulation compression). Most of the settlements should occur as the foundation 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 8.3 of this report) should be negligible.

7.5 Seismic Design Parameters

We have assumed that the International Building Code (IBC) 2015 will be used for the design of the proposed repairs/upgrades. The seismic site classification for the project site is *D* based on the $(N_1)_{60}$ values that we calculated for the relatively dense sand deposits 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 $F_a = 1.000$ ($S_s = 1.500$ g) and $F_v = 1.500$ ($S_1 = 0.669$ g). A copy of the SEAOC Design Maps report for the project site is contained in Appendix D of this report.

Based on our findings, we expect there to be little to no potential for soil liquefaction at the project site, given the relative lack of shallow groundwater at the project site. The potential for earthquake-induced lateral spreading and/or the development of pressure ridges are also unlikely.

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8.0 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.

8.1 Earthworks

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

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 (generally less than 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 9.1 of this report for more details).

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.

8.2 Conventional Foundations

For the purposes of this report, a conventional foundation can be considered any a poured-concrete foundation/footing which is founded below the seasonal active layer at the project site. Conventional foundations can be buried at shallower depths but will require varying amounts of insulation and/or NFS fill (or other forms of frost protection) to prevent damage from frost heaving and/or any thaw-related settlements that may ensue.

8.2.1 Soil Bearing Pressure

Concrete foundations placed on either the undisturbed native sand and/or silt deposits or on structural fill pads (constructed directly above the undisturbed native sand and/or silt deposits) may be designed for an allowable soil bearing pressure of 2,500 pounds per square foot (psf). The design soil bearing pressure may be increased by one-third (1/3) to accommodate short-term wind and/or seismic loads. Larger footings (smallest dimension greater than two feet in plan dimension) may be designed for greater bearing pressures at a rate of 300 psf for every additional horizontal linear foot of footing up to a maximum value of 4,300 psf.

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8.2.2 Foundation Footings

Foundation footings can be founded directly onto either: 1) the undisturbed native sand and/or silt deposits, or 2) properly placed structural fill (located directly above the undisturbed native sand and/or silt deposits). Footing burial depths will vary, however, based on whether or not the foundation subgrade will be allowed to freeze during winter months (See Sections 8.2.4 and 8.2.5 of this report for more details regarding foundation insulation and cold foundations).

8.2.3 Concrete Slabs

Concrete slabs can also be founded directly onto the undisturbed native sand and/or silt deposits or properly placed structural fill located directly above the undisturbed native sand and/or silt deposits. Any thickened slab edges (i.e., perimeter slab footings) 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.

Concrete slabs constructed directly on the undisturbed native sand and/or silt deposits or on properly constructed granular fill pads (located directly above the undisturbed native sand and/or silt deposits), as we described above, may be designed using a modulus of subgrade reaction of k_1=90 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 sand and/or silt deposits or on properly placed granular structural fill located directly above the undisturbed native sand and/or silt deposits:

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

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

The following equation (2) can be used for a rectangular load having the dimensions $B \times L$ (in feet) with similar bearing soils as the square footprint loading equation above (1).

$$k_{(B\ x\ L)} = \frac{k_{(B\ x\ B)}\left(1+0.5\frac{B}{L}\right)}{1.5} \tag{2}$$

Where:

$k_{(B\ x\ B)}$ = the modulus of subgrade reaction for a $B \times B$ square load footprint
$k_{(B\ x\ L)}$ = the modulus of subgrade reaction for $B \times L$ rectangular load footprint
B = the least horizontal dimension of a rectangular load footprint
L = the larger horizontal dimension of a rectangular load footprint

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8.2.4 Footing Uplift

Conventional foundations should be buried sufficiently deep so as to resist any anticipated uplift/overturning forces (e.g. wind, seismic, frost jacking, etc.). The uplift capacity of a foundation is a function of its weight, configuration, and depth. The ultimate uplift load can be calculated by using 80 percent of the weight of the foundation plus 80 percent of the weight of the effective soil mass located above the footing. In Figure 5 of this report, we illustrate the impact that effective soil mass has on the uplift capacity of a shallow foundation footing. An effective unit weight of 130 pcf can be used for granular structural backfill material. The ultimate uplift load includes any short-term load factors, so no increase in uplift capacity should be added for short-term loading.

Frost heaving forces can generate significant footing uplift loads. As such, conventional foundations need to be buried sufficiently deep and/or be adequately insulated so as to reduce the potential for freezing of the foundation subgrade and any associated frost heaving forces. For the project site, the minimum burial depth for any uninsulated cold (i.e., unheated) conventional foundation footings should be 96 inches (D_3 in Figures 6 and 7), measured from the bottom of the footing to the ground surface – including any floor slabs). The minimum burial depth (D_3) can be reduced if the cold conventional foundation is placed above a properly constructed NFS fill pad and/or proper amounts of artificial insulation (See Section 8.2.5 of this report for more details regarding cold conventional foundation design).

8.2.1 Foundation Insulation

Artificial insulation can be used to decrease minimum burial depths for unheated foundations by helping to reduce the potential for freezing of foundation soils. Any subsurface insulation should consist of extruded polystyrene such as DOW Styrofoam™ Highload or UC Industries Foamular. Any subsurface insulation used under structural slabs should be closed cell, board stock with a minimum compressive strength of 60 psi at five percent deflection. Subsurface insulation around foundations should have a minimum compressive strength of 25 psi at five percent deflection. The insulation should not absorb more than two percent water per ASTM Test Method C-272. The thermal conductivity (k) of the insulation should not exceed 0.25 BTU-in/hr-ft^2-°F when tested at 75°F. Proper bedding material should be used to provide a flat, smooth surface for the insulation.

8.2.2 Cold (Unheated) Conventional Foundations

It is difficult to predict the depth of frost penetration and extent of ice lens formation at any given site. Therefore, we do not recommend the construction of cold (i.e., unheated) conventional foundations as the formation of ice lenses beneath of a foundation can result in deformation to the overlying foundation. Therefore, avoid placing conventional foundation footings in unheated areas so as to reduce the potential for differential movements. If cold conventional foundations are required, then they should be placed on granular structural pads constructed of NFS fill material (NFS material should have less than 6% of the material passing a #200 sieve) which extends vertically from the minimum cold foundation burial depth (D_3) a minimum of five feet. Insulation may be incorporated into the cold conventional foundation design to help protect the foundation subgrade from freezing. Insulation may be used in lieu of some of the NFS backfill. In terms of insulating properties, one inch of rigid board insulation can be considered equivalent to one foot of NFS fill.

A minimum of 18 inches of NFS fill must be present between the bottom of any conventional concrete footing and the top of any insulation to help protect the insulation from damage. Furthermore, the compression of any sub-foundation insulation should be factored into the design of any cold conventional foundation. The design load of the foundation plus the mass of the overlying backfill will induce some insulation compression. For 60 psi insulation, the maximum compression (at 60 psi) is approximately five percent. For loads less than 60 psi, the compression ratio can be assumed to be linear.

We have detailed our recommended insulation configurations for cold conventional foundations in Figure 7 of this report (configurations E and F). We do not recommend the construction of a cold concrete slab foundation unless it is supported by an appropriately constructed NFS/insulated structural pad (as we discuss above).

8.3 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 (without the need for excessive earthworks – as is the case for cold conventional foundations). As we discuss in Section 7.3 of this report, deep foundations (e.g., pile foundations, etc.) are a suitable alternative to conventional poured-concrete foundations.

We only provide recommendations for driven steel pipe piling in this report, however driven steel beam piling, helical piers, and drilled concrete are suitable deep foundation alternatives to driven steel pipe piles. It is not feasible 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 pipe piling), then we can provide relevant recommendations for the alternative deep foundation system at that time.

8.3.1 Driven Steel Pipe Piles

The most common type of deep foundation system in the Alaska consists of driven steel pipe piling. Steel pipe piling can be obtained in a variety of diameters and wall thicknesses to accommodate a wide-range of applications and is relatively inexpensive and readily available. Steel pipe piles are typically installed open-ended so that the soil can penetrate the inside of the pile, which helps facilitate efficient pile driving activities. Open-ended steel pipe pile can be driven with or without the use of a re-enforced/hardened drive shoe; which protects the end of the pile from damage during the driving activities. Steel pipe piles can also be installed close-ended, which helps to increase pile bearing capacities in soft, fine-grained soils. Any pile installation should be completed with quality control inspection to verify the pile configuration and final penetration rate.

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The final penetration rate is used to determine that the individual piles have the required axial capacity.

8.3.2 Pile Bearing Load

We used the computer program ALLPile7 (developed by CivilTech software) to analyze the estimated vertical bearing load for each of the pile diameters/sizes that we detail in Figure 8 of this report. Our estimated vertical bearing loads assume open-ended piles as closed-ended piles would be difficult to drive into the medium dense soil that we observed at the project site. Since the foundation size/loads have not yet been finalized for this project, we estimated the vertical pile bearing loads (as a function of driven depth) for a range of common pipe pile sizes. We can refine our vertical pile bearing loads once the anticipated pile loads are known and a preferred pile diameter/size has been selected.

When multiple piles are installed in close proximity to one another, then pile group efficiency should be considered. We discuss pile group efficiency in further detail in Section 8.3.2.3 of this report.

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

8.3.2.1 **Pile Uplift Load**

Cold pile foundations (i.e., pile foundations where the soils surrounding individual piles are allowed to freeze) will need to be installed to greater depths than those installed within continuously heated spaces in order to resist frost jacking (i.e., uplift) forces. A minimum pile embedment of 20 feet bgs is required for any cold piles installed at the project site in order to resist frost jacking forces. The allowable short-term uplift load of each pile may be taken as one-half (1/2) of the long-term bearing load as we detail in Figure 8 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 8.3.2.3 of this report.

8.3.2.2 **Lateral Pile Load**

We again used ALLPile7 to analyze the allowable lateral load for each of the pile diameters/sizes that we present in Figure 8 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). We have listed the ultimate and allowable lateral loads for each pile diameter/size at the ground surface (with no pile stickup) in Table 1 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 greater detail in Section 8.3.2.3 of this report.

Table 1: Allowable Free-Head Lateral Pile Loads

PILE TYPE	MAX. DEFLECTION (in)	MIN. DEPTH (ft)	ULTIMATE CAPACITY (kips)*	ALLOWABLE CAPACITY (kips)*
6-in SCH. 80	1	20	7.6	3.8
8-in SCH. 80	1	20	12.8	6.4
10-in SCH. 80	1	20	19.0	9.5
12-in SCH. 80	1	20	25.4	12.7
14-in SCH. 80	1	20	36.8	18.4
18-in SCH. 80	1	20	54.2	27.1
20-in SCH. 80	1	20	63.6	31.8
24-in SCH. 80	1	20	88.0	44.0

*Lateral pile loads calculated using ALLPile7 and 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 8 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	EFFECTIVE UNIT WEIGHT (PCF)	INTERNAL FRICTION ANGLE (DEGREES)	MODULUS OF SUBGRADE REACTION (K)
*SAND (0-15 FT)	120	30	70
SAND (15-50 FT)	125	32	90

*Lower soil parameter values due to increased silt content from 0 to 15 ft bgs.

8.3.2.3 **Pile Group Efficiency**

Group efficiency of steel pipe piles is a function of the spacing of the individual piles. In Table 2 of this report, we present pile group efficiency parameters (as a function of pile diameter). The allowable pile loads that we provide in Figure 8 of this report should be adjusted as necessary according to the spacing of individual piles.

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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 Pile

In Table 3 of this report we provide pile group efficiency parameters for lateral loads. The allowable lateral loads that we provide in Table 1 of this report 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 Pile

8.3.2.4 Pile Foundations with Connecting Structural Members

Cold pile foundations are not recommended with the use of any grade-level structural members as frost heaving forces can damage the structural members and/or result in failures at connections between pile foundations and structural members. We recommend that a minimum air gap of six inches be maintained between the ground surface and any structural members that span between cold pile foundations. We should be consulted in the event that the structural design cannot accommodate a sub-structural member air gap so that we can evaluate any frost heaving pressures that may develop, so that they can be accounted for by the structural design.

9.0 CONSTRUCTION RECOMMENDATIONS

We have presented our construction recommendations in the general order that the proposed repairs/upgrades will most likely be conducted. Our construction recommendations are intended to aid the construction contractor(s) during the construction process.

9.1 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.

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

9.2 Cold (Unheated) Conventional Foundations

The frost susceptibility of the native sand and silt soils (as we describe in Section 6.1 of this report) range from NFS to F4. Therefore, some of the foundation soils are unsuitable to support any cold (unheated) conventional foundations without freeze protection, as they may experience ice lens development and/or thaw-weakening, which could result in damages to the proposed foundations. As we mention in Section 8.2.4 of this report, the minimum cold foundation burial depth (D_3) can be reduced, if the foundation is placed on a structural pad constructed of NFS fill (minimum of five-feet in thickness). The NFS structural pad thickness may be reduced by using insulation at a rate of one inch of insulation to one foot of NFS material.

9.3 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.

Piles may be allowed to freeze and/or be installed in frozen soils, if they are driven to a minimum depth of 20 feet bgs for cold pile foundations (assuming no grade-level structural members are connected to adjoining pile foundations – See Section 8.3.2.4 of this report for more detail).

9.4 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.

10.0 PROJECT SPECIFICATIONS

Poor or incomplete design/construction specifications for the geotechnical aspects of any project can result in higher construction estimates, change order claims, and delays in construction. Presentation of the geotechnical data is a useful portion of the final plans and specifications that will be used by contractors to prepare their bids to construct a given project. However, this report was prepared before the design for the proposed repairs/upgrades has been finalized and our report includes design alternatives that may not be pertinent to the final design.

Prior to the completion of the final design (typically at the 90% design level), we should (at a minimum) be contracted to review the plans and specifications with respect to our geotechnical recommendations. Furthermore, we recommend that we be allowed to prepare the specifications pertaining to any foundations and/or earthworks for this project as we are intimately familiar with the geotechnical site conditions. For this project, we recommend that we be involved with the development of specifications for the following aspects of construction:

- pile driving criteria;
- conventional foundation earthworks preparation;
- site grading; and
- special inspections for piling, concrete, and earthworks.

11.0 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.

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.

12.0 CLOSURE

We (Northern Geotechnical Engineering, Inc. d.b.a. Terra Firma Testing) prepared this report exclusively for the use of our client, Electric Power Systems, and their client (Chugach Electric Association). for use in the design and construction of the proposed substation repairs/upgrades. We should be notified if significant changes are to occur in the nature, design, or location of the proposed repairs/upgrades 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.

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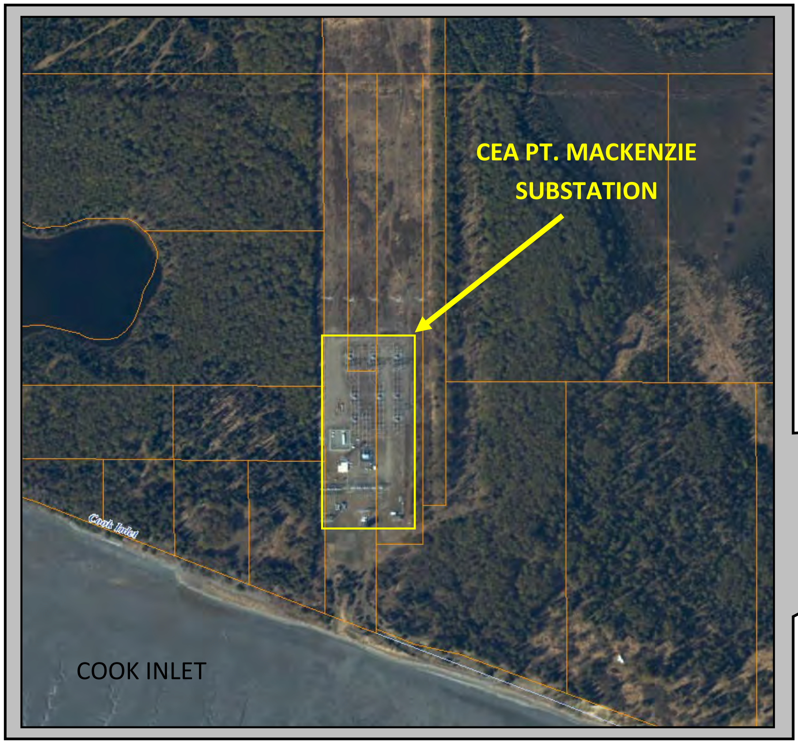
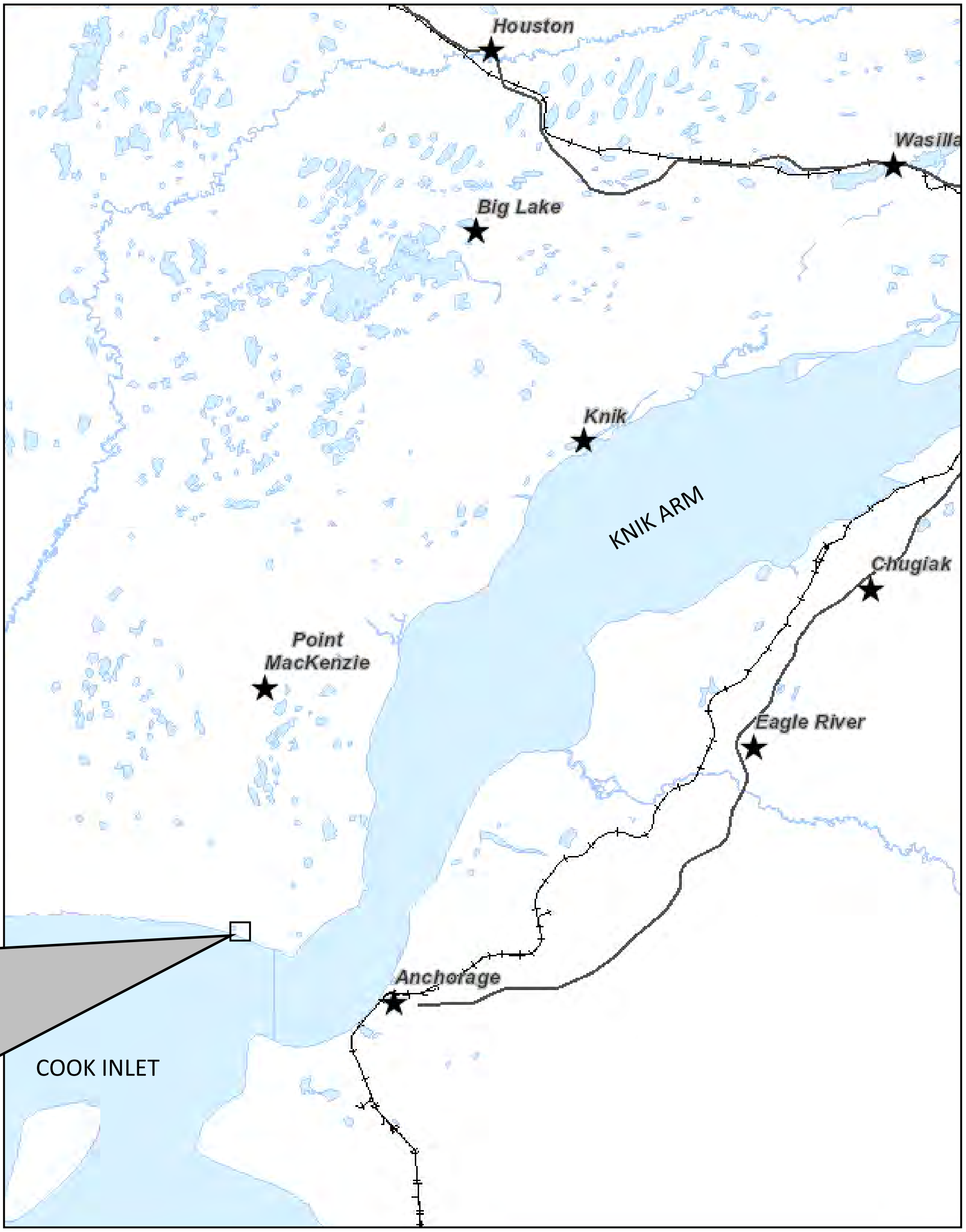
This report should always be read and/or distributed in its entirety (including all figures, exploration logs, appendices, etc.) so that all of the pertinent information contained within is effectively disseminated. Otherwise, an incomplete or misinterpreted understanding of the site conditions and/or 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; but only once we have been allowed to review the final design and ensure that our original recommendations still apply.

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 10.0 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.

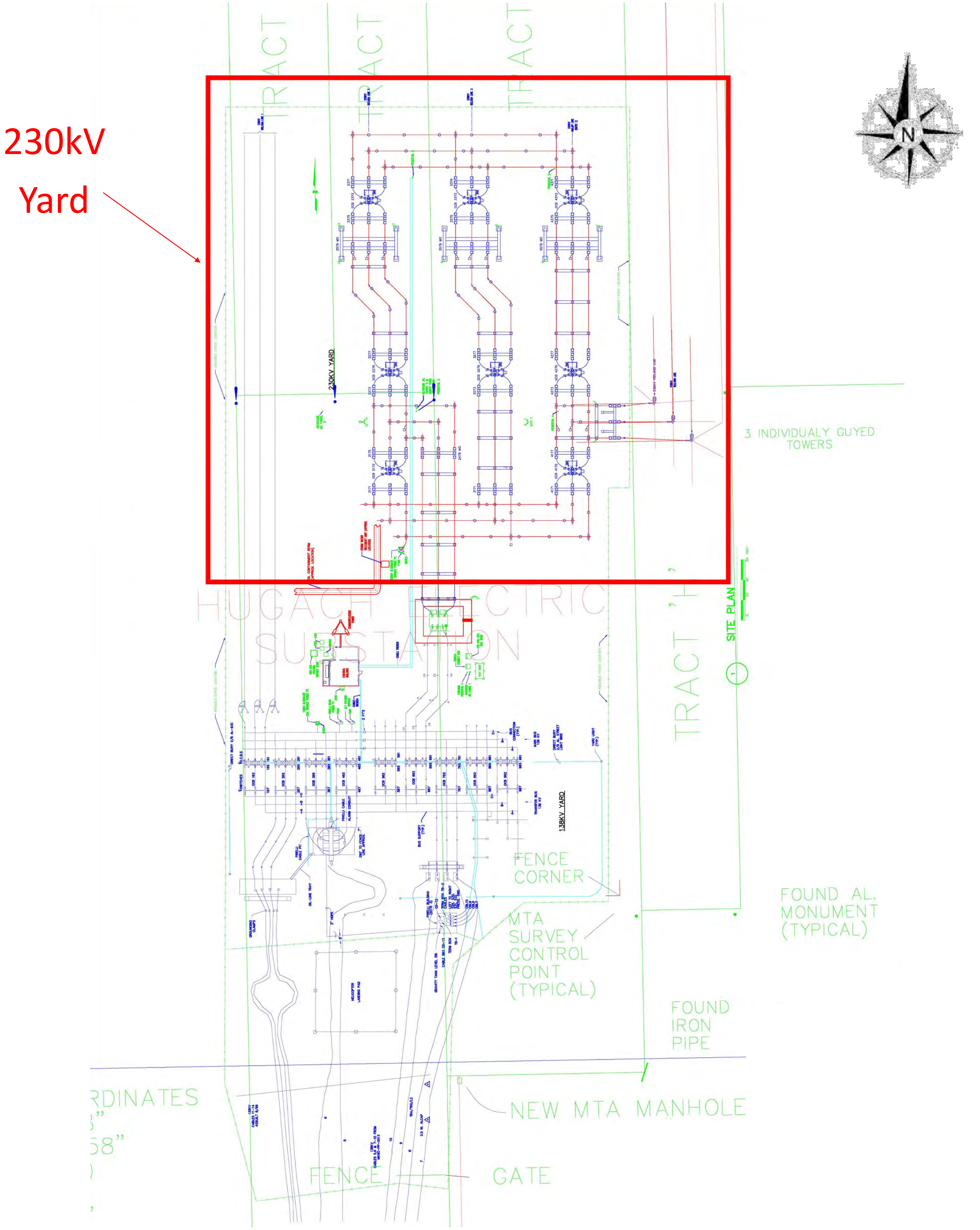
REPORT
FIGURES





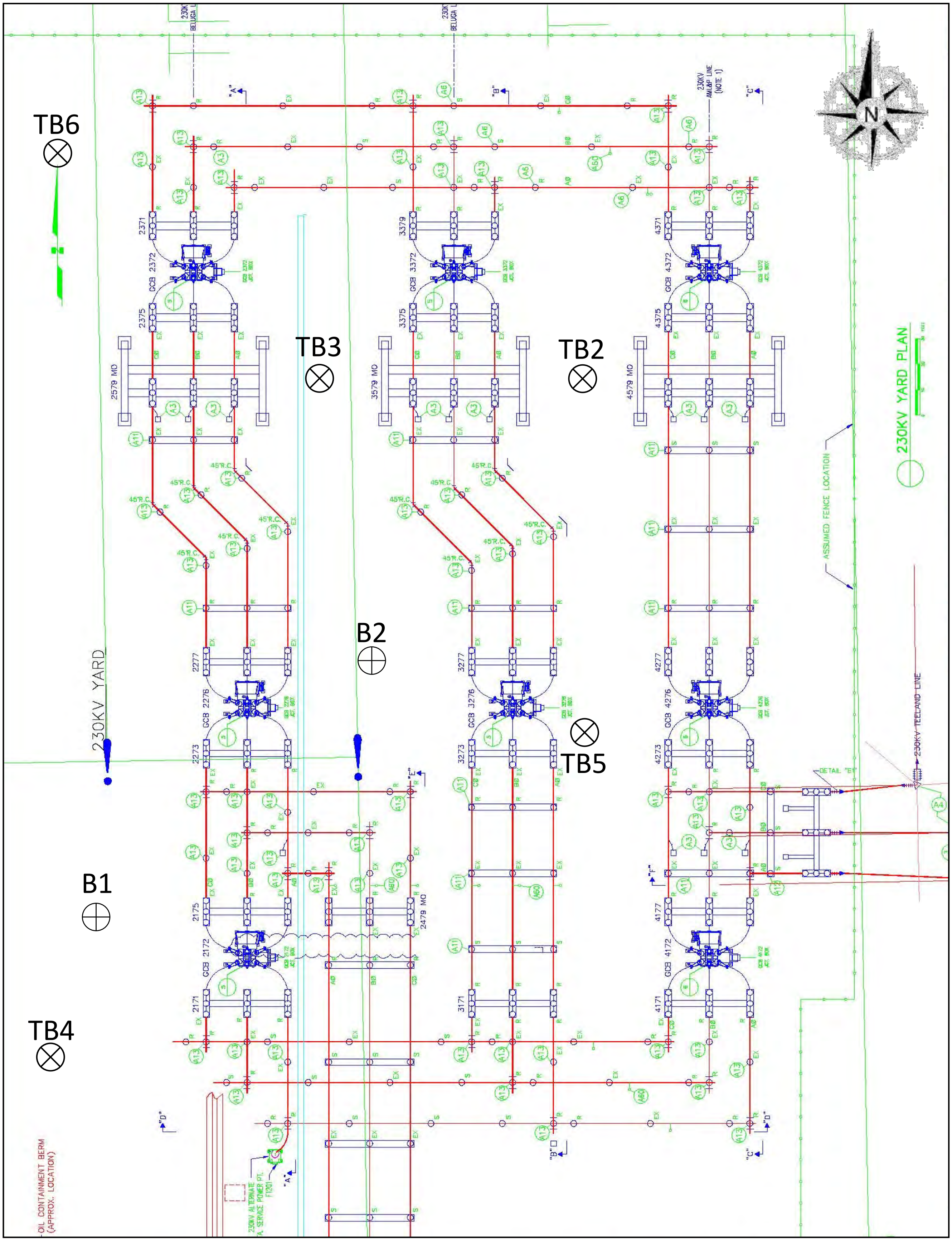
NORTHERN GEOTECHNICAL ENGINEERING, INC.
TERRA FIRMA TESTING

FIGURE TITLE: PROJECT SITE LOCATION MAP	
PROJECT NAME: CEA PMSS - 230KV YARD REPAIRS/UPGRADES	PROJECT ID: 5298-19
PROJECT LOCATION: PT MACKENZIE, ALASKA	FIGURE NUMBER: 1



NORTHERN GEOTECHNICAL ENGINEERING, INC.
TERRA FIRMA TESTING

FIGURE TITLE: PROJECT SITE LAYOUT	
PROJECT NAME: CEA PMSS - 230KV YD REPAIRS/UPGRADES	PROJECT ID: 5298-19
PROJECT LOCATION: PT. MACKENZIE, ALASKA	FIGURE NUMBER: 2



⊕ = APPROX. SOIL BORING LOCATION (NGE-TFT 2019)

⊗ = APPROX. SOIL BORING LOCATION (DOWL 1984)

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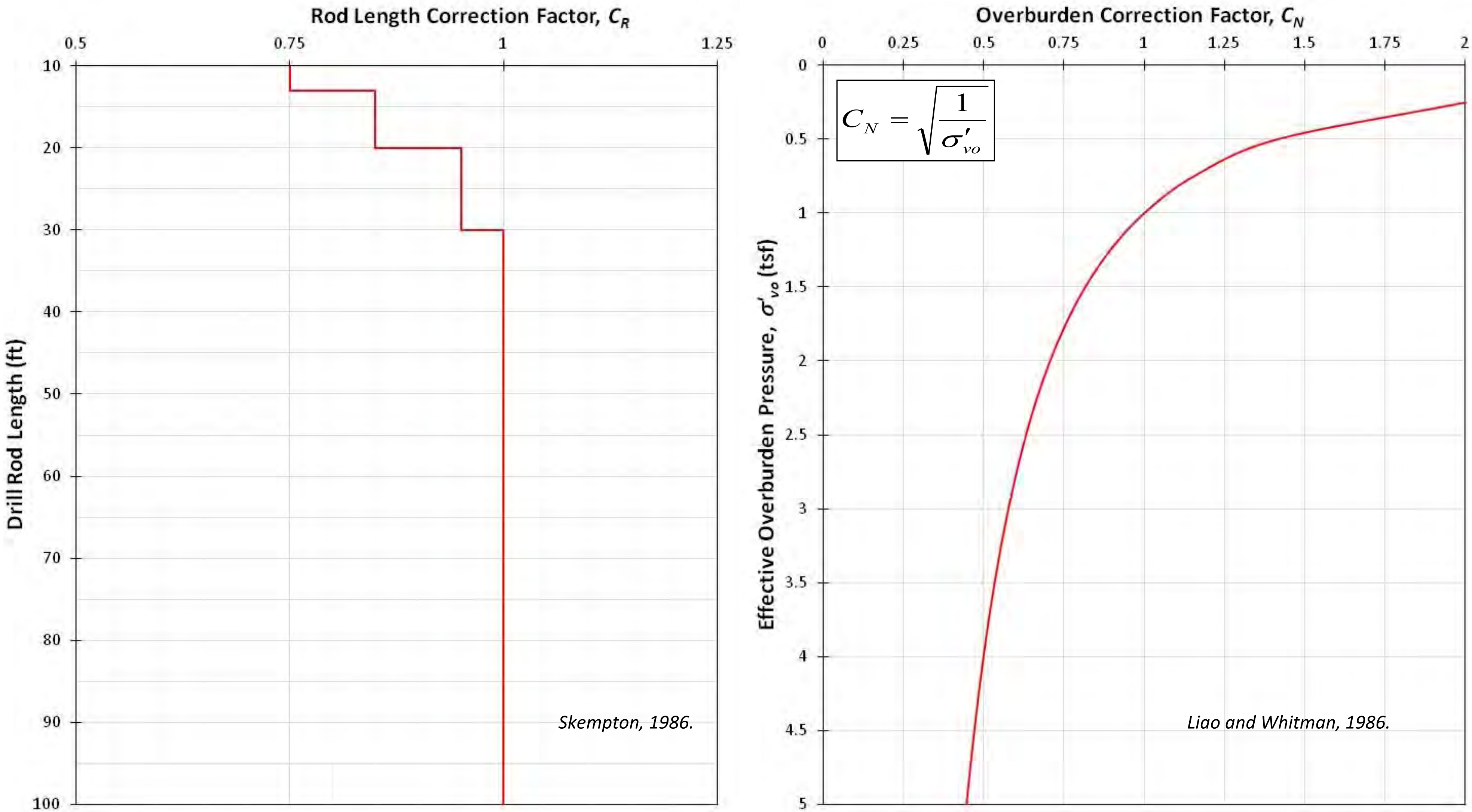
TERRA FIRMA TESTING

FIGURE TITLE: 230KV YARD DETAIL AND SOIL BORING LOCATIONS	
PROJECT NAME: CEA PMSS - 230KV YD REPAIRS/UPGRADES	PROJECT ID: 5298-19
PROJECT LOCATION: PT. MACKENZIE, 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)

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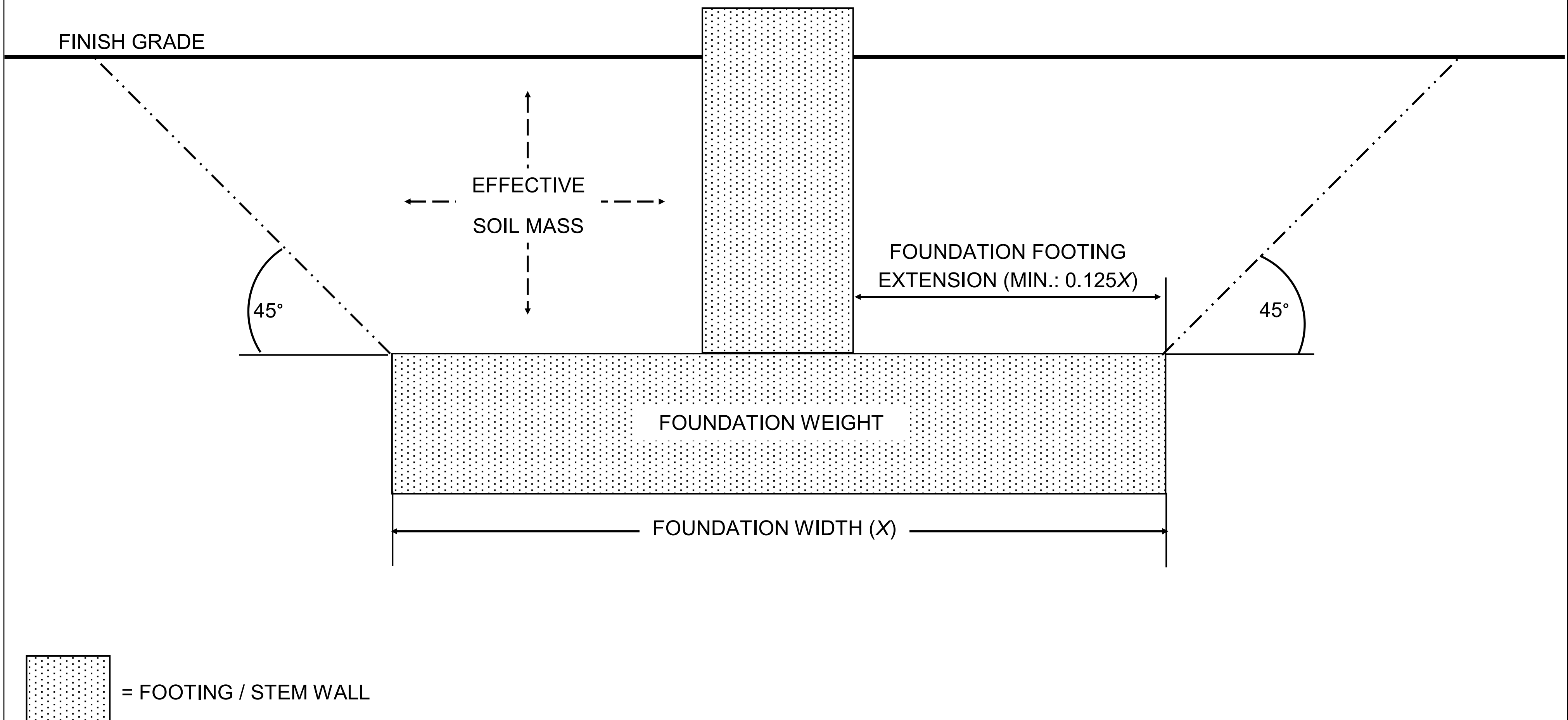
TERRA FIRMA TESTING

FIGURE TITLE: BLOW COUNT CORRECTIONS	
PROJECT NAME: CEA PMSS - 230KV YARD REPAIRS/UPGRADES	PROJECT ID: 5298-19
PROJECT LOCATION: PT. MACKENZIE, ALASKA	FIGURE NUMBER: 4

UPLIFT LOAD = 0.8 × (EFFECTIVE SOIL WEIGHT + WEIGHT OF FOUNDATION)

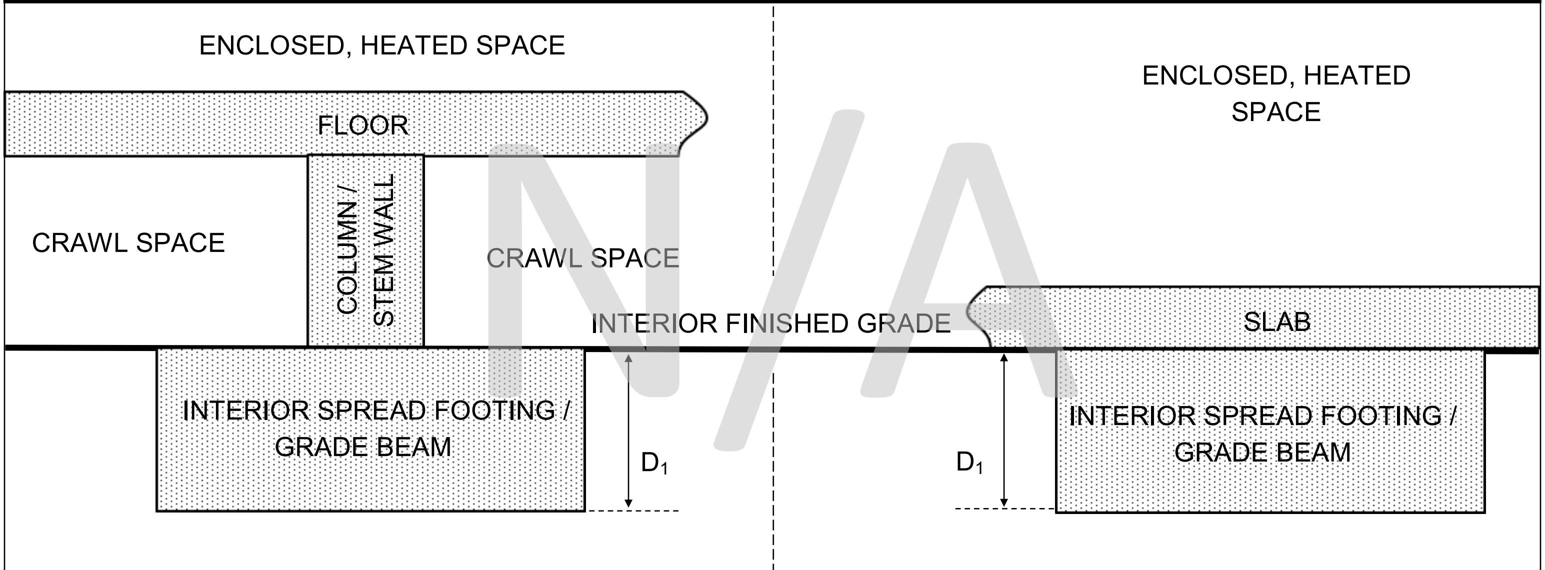


DRAWING NOT TO SCALE

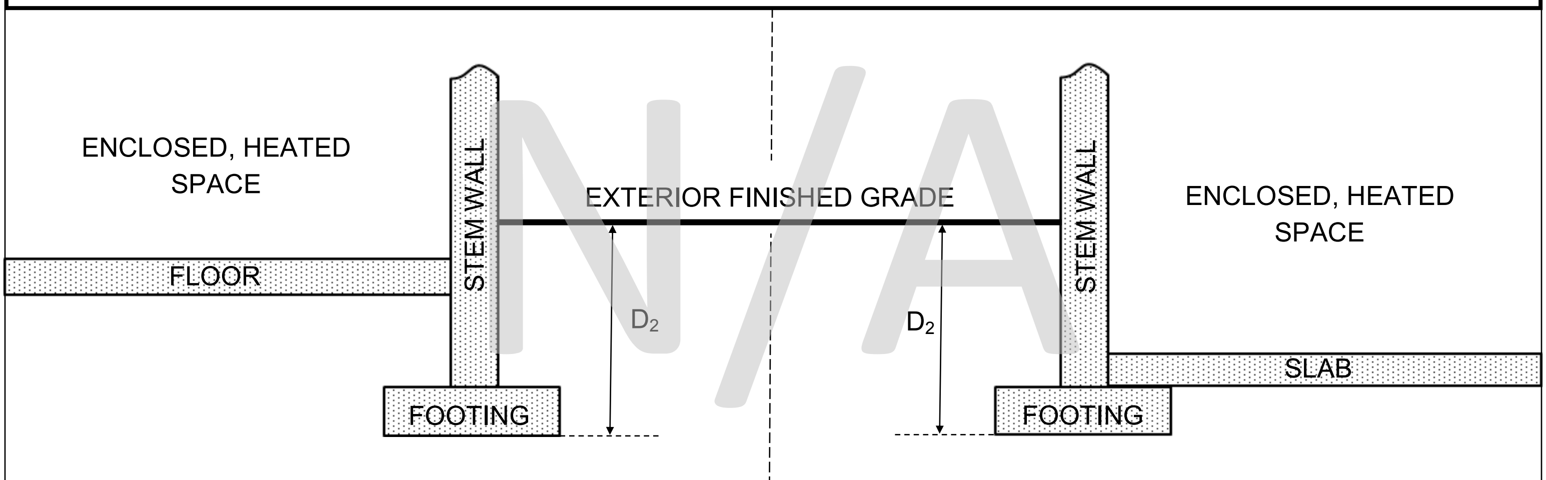
NORTHERN GEOTECHNICAL ENGINEERING, INC.
TERRA FIRMA TESTING

FIGURE TITLE: CONVENTIONAL FOUNDATION FOOTING UPLIFT LOAD DIAGRAM	
PROJECT NAME: CEA PMSS - 230KV YARD REPAIRS/UPGRADES	PROJECT ID: 5298-19
PROJECT LOCATION: PT. MACKENZIE, ALASKA	FIGURE NUMBER: 5

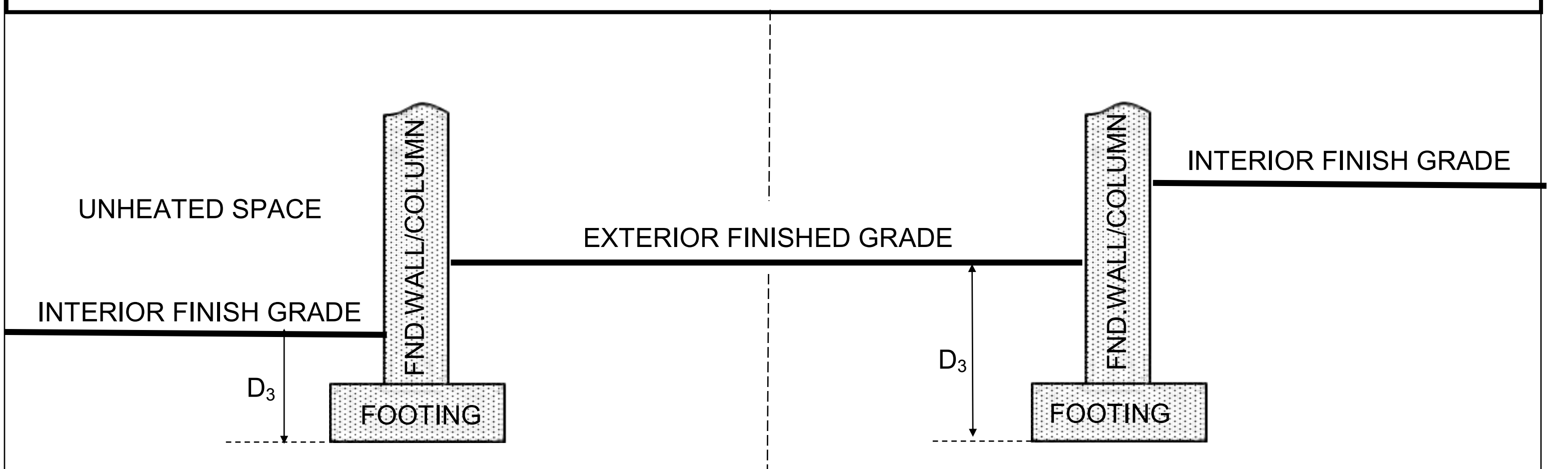
SHALLOW FOUNDATION FOOTING LOCATED ENTIRELY WITHIN AN ENCLOSED, CONTINUOUSLY HEATED SPACE*



SHALLOW FOUNDATION FOOTING LOCATED ALONG THE PERIMETER OF AN ENCLOSED, CONTINUOUS HEATED SPACE*



SHALLOW FOUNDATION FOOTING EXPOSED TO FREEZING TEMPERATURES



*HEATED FOUNDATION TEMPERATURE MUST BE CONTINUOUSLY MAINTAINED AT/ABOVE 40^0F

DRAWING NOT TO SCALE

NORTHERN GEOTECHNICAL ENGINEERING, INC.
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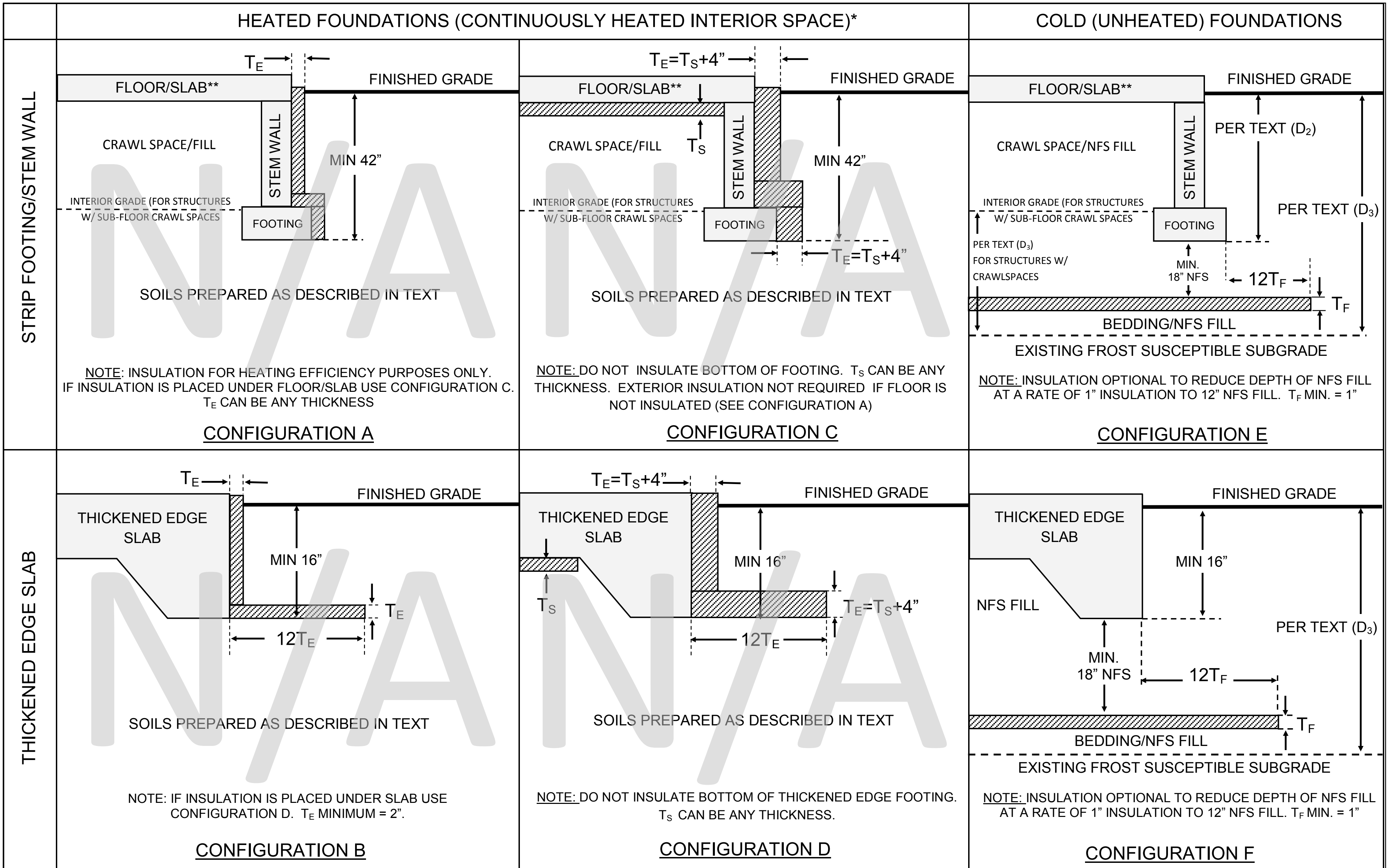
FIGURE TITLE: UNINSULATED CONVENTIONAL FOUNDATION DETAILS	
PROJECT NAME: CEA PMSS - 230KV YD REPAIRS/UPGRADES	PROJECT ID: 5298-19
PROJECT LOCATION: PT. MACKENZIE, ALASKA	FIGURE NUMBER: 6



T_F = INSULATION THICKNESS UNDER ENTIRE FOUNDATION (INCHES)

T_S= INSULATION THICKNESS UNDER FLOOR/SLAB ONLY (INCHES)

T_E = INSULATION ALONG EXTERIOR OF FOUNDATION (INCHES)

*HEATED FOUNDATION TEMPERATURE MUST BE CONTINUOUSLY MAINTAINED AT/ABOVE 40°F

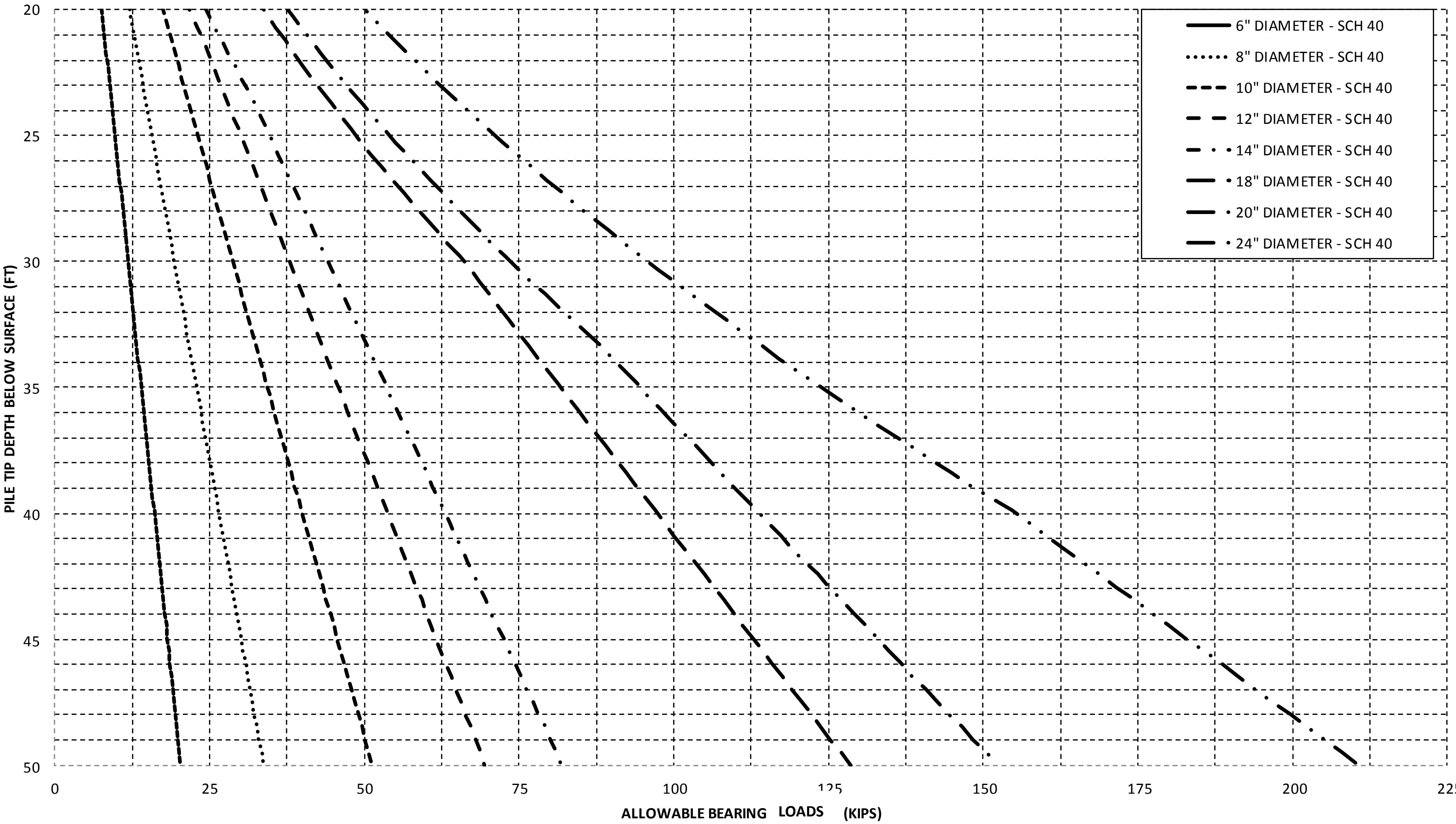
**FLOOR SYSTEM CAN BE STRUCTURAL (W/ CRAWLSPACE) OR SLAB-ON-GRADE

= RIGID BOARD INSULATION

DRAWING NOT TO SCALE

NORTHERN GEOTECHNICAL ENGINEERING, INC.
TERRA FIRMA TESTING

FIGURE TITLE: INSULATED CONVENTIONAL FOUNDATION CONFIGURATIONS	
PROJECT NAME: CEA PMSS - 230KV YARD REPAIRS/UPGRADES	PROJECT ID: 5298-19
PROJECT LOCATION: PT. MACKENZIE, ALASKA	FIGURE NUMBER: 7



NOTES:
FACTOR OF SAFETY = 2
ALLOWABLE BEARING LOADS MAY BE INCREASED BY 1/3 FOR SHORT-TERM WIND AND SEISMIC LOADS
ALLOWABLE UPLIFT LOADS = 1/2 OF THE ALLOWABLE BEARING LOADS
DO NOT ADD 1/3 FOR SHORT-TERM WIND AND SEISMIC LOADS FOR UPLIFT CONDITIONS

NORTHERN GEOTECHNICAL ENGINEERING, INC.
TERRA FIRMA TESTING

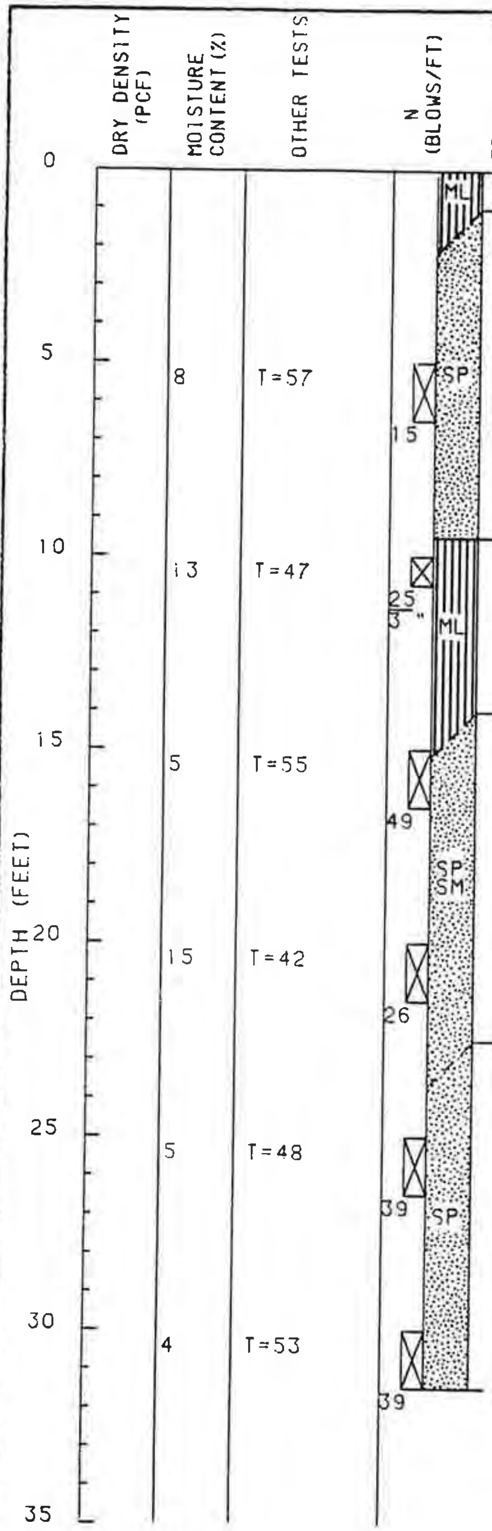
FIGURE TITLE: ESTIMATED ALLOWABLE BEARING LOADS FOR VARIOUS PIPE PILE DIA.	
PROJECT NAME: CEA PMSS - 230KV YARD REPAIRS/UPGRADES	PROJECT ID: 5298-19
PROJECT LOCATION: PT. MACKENZIE, ALASKA	FIGURE NUMBER: 8

APPENDIX A

SOIL BORING LOGS AND LABORATORY TEST DATA (DOWL 1984)

TEST BORING 2

LOCATION=SEE TEST BORING LOCATION MAP
ELEVATION=77 ± ESTIMATED FROM CONTOUR MAP



DEPTH	
	F4. BROWN SANDY SILT. NONPLASTIC. DAMP. STIFF
1.0	NFS. BROWN SAND. CLEAN. POORLY GRADED. MEDIUM GRAIN SIZE. DAMP. MEDIUM DENSE
9.5	F4. SANDY GRAVELLY SILT. NONPLASTIC. ABOUT 15% FINE SAND. SCATTERED SUBROUND GRAVEL AND COBBLES. DAMP. HARD
14.0	NFS/F2. BROWN SAND. CLEAN TO WITH ABOUT 5% TO 10% NONPLASTIC FINES. THIN LENSES OF SANDY SILT. DAMP. DENSE
22.5	NFS. LIGHT BROWN SAND. CLEAN. POORLY GRADED. MEDIUM GRAIN SIZE. DAMP. DENSE
31.5	TEST BORING COMPLETED 7/30/84 NO GROUNDWATER OBSERVED WHILE DRILLING

KEY

MA = MECHANICAL ANALYSIS
LL = LIQUID LIMIT
PI = PLASTIC INDEX
PP = POCKET PENETROMETER (TSF)
TV = TORVANE (TSF)
□ = GRAB SAMPLE
⧄ = SPT SAMPLE
[S] = SHELBY TUBE-PUSHED
⊠ = 2.5" I.D. SPOON SAMPLE 340# WEIGHT. 30" FALL
T = SAMPLE TEMPERATURE (°F) PROBABLY AFFECTED BY SAMPLING PROCEDURE

DOWL ENGINEERS

LOG OF BORING

LOGGED BY T.BARBER
W.O. D51343

FIGURE 15

TEST BORING 3

LOCATION=SEE TEST BORING LOCATION MAP
ELEVATION=74 ± ESTIMATED FROM CONTOUR MAP

DEPTH (FEET)	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	OTHER TESTS	N (BLOWS/FT)	FROST DEPTH
0					ML
5		11		18	SP-SM
5		12	T=53	22	SP-SM
10		13	T=50	30	SP-SM
15		5	T=53 MA COMBINED WITH SAMPLE 5	34	SP-SM
20		7	T=52 MA COMBINED WITH SAMPLE ABOVE	30	SP-SM
25		4	T=52	48	SP-SM
30		3	T=51 MA	39	SP-SM
35					

DESCRIPTION	DEPTH
F4. BROWN SANDY SILT. NONPLASTIC. DAMP. DENSE	
	2.0
NFS/F1. BROWN SAND. CLEAN TO WITH ABOUT 5% TO 15% NONPLASTIC FINES. THIN LENSES OF SANDY SILT. DAMP TO WET AT ABOUT 10'. MEDIUM DENSE TO DENSE	
	13.5
NFS. LIGHT BROWN SAND. CLEAN TO WITH ABOUT 5% TO 10% NONPLASTIC FINES. CLEANER WITH DEPTH. DAMP. DENSE	
	31.5

TEST BORING COMPLETED 7/30/84

PERCHED GROUNDWATER TABLE AT 10'. NO GENERAL GROUNDWATER TABLE AT THIS LOCATION

KEY

MA = MECHANICAL ANALYSIS
LL = LIQUID LIMIT
PI = PLASTIC INDEX
PP = POCKET PENETROMETER (TSF)
TV = TORVANE (TSF)
☐ = GRAB SAMPLE
⍁ = SPT SAMPLE
[S] = SHELBY TUBE-PUSHED
☒ = 2.5" I.D. SPOON SAMPLE 340# WEIGHT, 30" FALL
T = SAMPLE TEMPERATURE (°F) PROBABLY AFFECTED BY SAMPLING PROCEDURE

DOWL ENGINEERS

LOG OF BORING

LOGGED BY T.BARBER
W.O. D51343

FIGURE 16

TEST BORING 4

LOCATION=SEE TEST BORING LOCATION MAP
ELEVATION=70± ESTIMATED FROM CONTOUR MAP

Sample depth (approx., ft)	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	OTHER TESTS	N (BLOWS/FT)	
0–2.0					ML
3–4.5		7	T=52	8	SP
5.5–7		4	T=53 MA	12	SP
10.5–12		5	T=54	47	SP
15.5–17		5	T=47	30	SP
20.5–21.5		3	T=51	47	SP

DEPTH (FEET)

	DEPTH
F4. BROWN SANDY SILT. NONPLASTIC. SCATERED COAL AND OTHER ORGANICS. DAMP. STIFF	2.0
NFS. BROWN SAND. CLEAN. POORLY GRADED. COARSE TO MEDIUM GRAIN SIZE. SCATTERED COAL. DAMP. MEDIUM DENSE	9.5
NFS. LIGHT BROWN SAND. CLEAN. POORLY GRADED. MEDIUM GRAIN SIZE. THIN INTERLAYERS OF SANDY SILT. DAMP. DENSE GRAVELLY FROM ABOUT 13.5' TO 15'	21.5

TEST BORING COMPLETED 7/31/84
NO GROUNDWATER OBSERVED WHILE DRILLING .

KEY
- MA = MECHANICAL ANALYSIS
- LL = LIQUID LIMIT
- PI = PLASTIC INDEX
- PP = POCKET PENETROMETER (TSF)
- TV = TORVANE (TSF)
- □ = GRAB SAMPLE
- ▨ = SPT SAMPLE
- [S] = SHELBY TUBE-PUSHED
- ☒ = 2.5" I.D. SPOON SAMPLE 340# WEIGHT. 30" FALL
- T = SAMPLE TEMPERATURE (°F) PROBABLY AFFECTED BY SAMPLING PROCEDURE

DOWL ENGINEERS

LOG OF BORING

LOGGED BY T.BARBER
W.O. D51343

FIGURE 17

TEST BORING 5

LOCATION=SEE TEST BORING LOCATION MAP
ELEVATION=73 ± ESTIMATED FROM CONTOUR MAP

DEPTH (FEET)	DRY DENSITY (PCF)	MOISTURE CONTENT (%)	OTHER TESTS	N (BLOWS/FT)	FROST DEPTH	SOIL
0						ML
		8	T=56 MA/TRIAX COMBINED WITH SAMPLE 2	17		
5		11	T=54 MA/TRIAX COMBINED WITH SAMPLE ABOVE	6		SP SM
10				51/6"		
15		6	T=46 MA	28		SP SM
20		5	T=51	29		
25						
30						
35						

DESCRIPTION	DEPTH
F4. BROWN SANDY SILT. NONPLASTIC. DAMP. STIFF	1.0
NFS. BROWN SAND. CLEAN TO WITH ABOUT 10% NON-PLASTIC FINES. SCATTERED COAL. DAMP. MEDIUM DENSE GRAVELLY FROM ABOUT 9.5' TO 12.5'	12.5
NFS. LIGHT BROWN SAND. CLEAN TO WITH ABOUT 5% NON-PLASTIC FINES. POORLY GRADED. DAMP. DENSE	21.5

TEST BORING COMPLETED 7/30/84

PERCHED GROUNDWATER TABLE AT ABOUT 10'. NO GENERAL GROUNDWATER TABLE AT THIS LOCATION

KEY
- MA = MECHANICAL ANALYSIS
- LL = LIQUID LIMIT
- PI = PLASTIC INDEX
- PP = POCKET PENETROMETER (TSF)
- TV = TORVANE (TSF)
- □ = GRAB SAMPLE
- ⧄ = SPT SAMPLE
- S = SHELBY TUBE-PUSHED
- ⊠ = 2.5" I.D. SPOON SAMPLE 340# WEIGHT. 30" FALL
- T = SAMPLE TEMPERATURE (°F) PROBABLY AFFECTED BY SAMPLING PROCEDURE

DOWL ENGINEERS

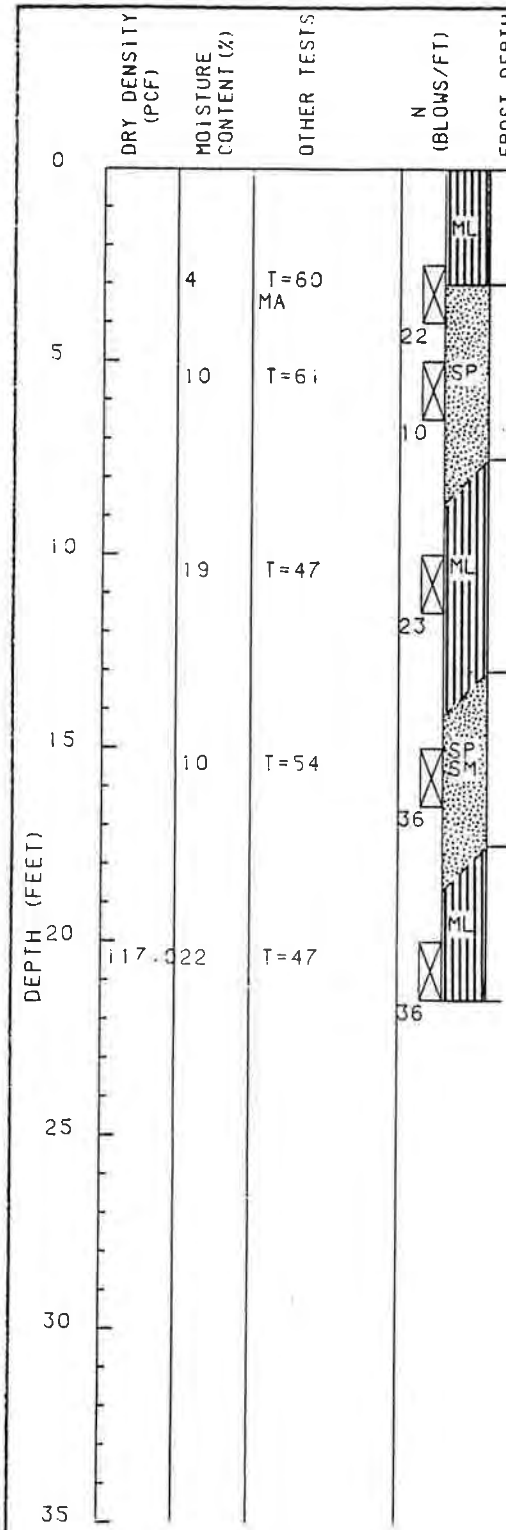
LOG OF BORING

LOGGED BY T.BARBER
W.O. D51343

FIGURE 18

TEST BORING 6

LOCATION=SEE TEST BORING LOCATION MAP
ELEVATION=73 ± ESTIMATED FROM CONTOUR MAP



DEPTH	
	F4. BROWN SANDY SILT. NONPLASTIC. DAMP. STIFF
3.0	
	NFS. BROWN SAND. CLEAN. POORLY GRADED. MEDIUM GRAIN SIZE. SCATTERED COAL. DAMP. MEDIUM DENSE
7.5	
	F4. BROWN SANDY SILT. NONPLASTIC. ABOUT 5% FINE SAND. SCATTERED GRAVELS AND COBBLES FROM ABOUT 11' TO 13'. DAMP BECOMING WET TO SATURATED AT ABOUT 10'. HARD
	∇ GROUNDWATER LEVEL WHILE DRILLING (10.0 FT)
13.0	
	NFS/F2. BROWN SAND. CLEAN TO WITH ABOUT 10% NON-PLASTIC FINES. FINE GRAINED. THIN INTERLAYERS OF SANDY SILT. SATURATED. MEDIUM DENSE TO DENSE
17.5	
	F4. BROWN SANDY SILT. NONPLASTIC. ABOUT 15% FINE SAND. SATURATED. HARD
21.5	
	TEST BORING COMPLETED 7/30/84

KEY

MA = MECHANICAL ANALYSIS
LL = LIQUID LIMIT
PI = PLASTIC INDEX
PP = POCKET PENETROMETER (TSF)
TV = TORVANE (TSF)
□ = GRAB SAMPLE
⍁ = SPT SAMPLE
S = SHELBY TUBE-PUSHED
⊠ = 2.5" I.D. SPOON SAMPLE
340# WEIGHT. 30" FALL
T = SAMPLE TEMPERATURE (°F) PROBABLY AFFECTED BY SAMPLING PROCEDURE

DOWL ENGINEERS

LOG OF BORING

LOGGED BY T.BARBER
W.O. D51343

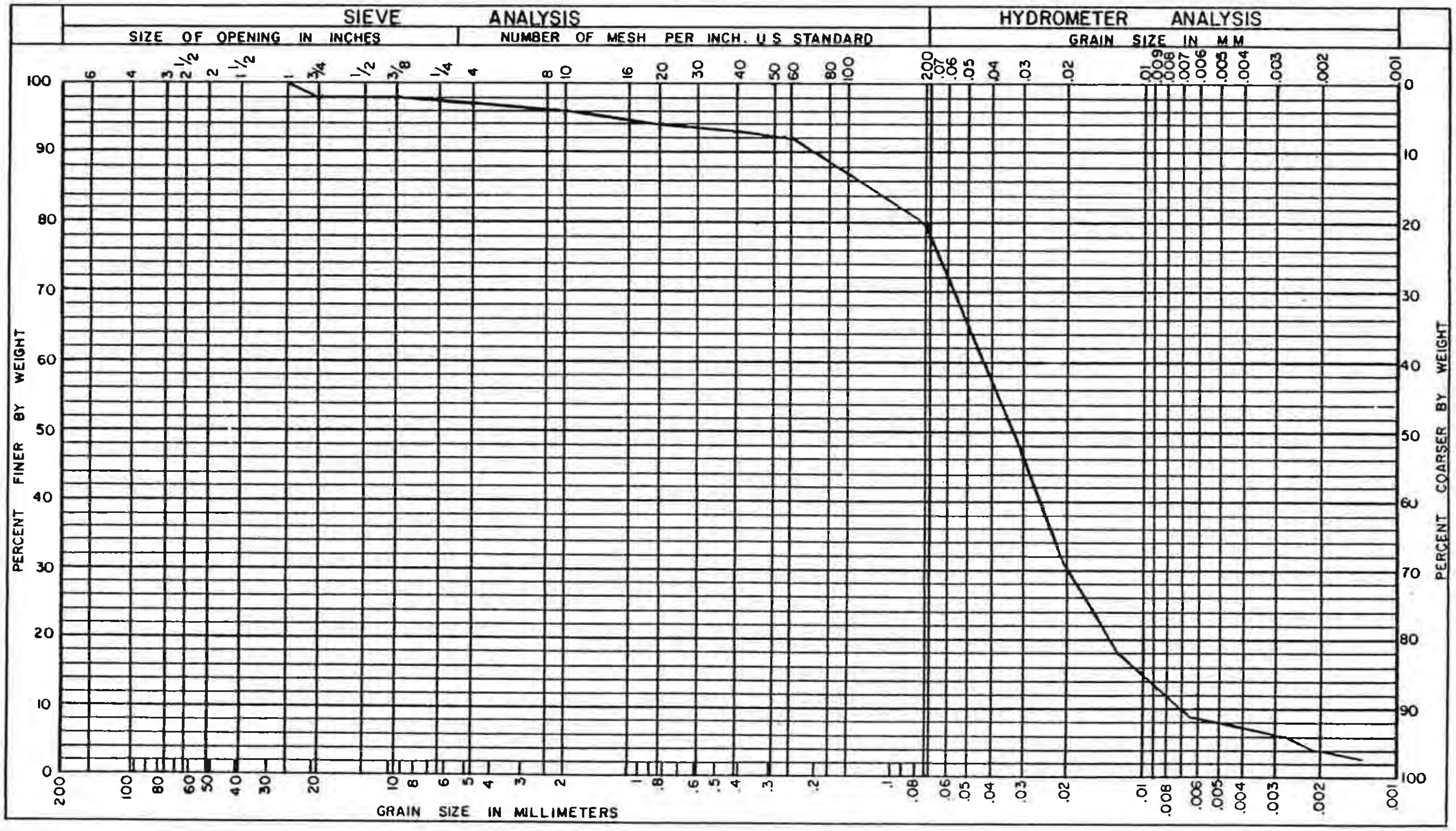
FIGURE 19

Alaska Testlab 4040 "B" Street Anchorage, Alaska 99503 Phone (907) 278-1551

Sheet 1 of 10
W. O. No. D51343
Date Aug.29,1984
Technician

Textural Class Sandy Silt
Frost Class F4 Unified Class ML
Plastic Properties
Date Received

Client Chugach Electric Association
Project Point MacKenzie Substation
Sample Number
Location TB 1, Sa. 4&5, combined, 15' to 21.5'
Sample Taken By



US STD SIEVE	CUM % PASS
3	
2	
1 1/2	
1	100
3/4	98
1/2	98
3/8	98
4	97
10	96
20	95
40	94
100	88
200	80
0.02MM	30

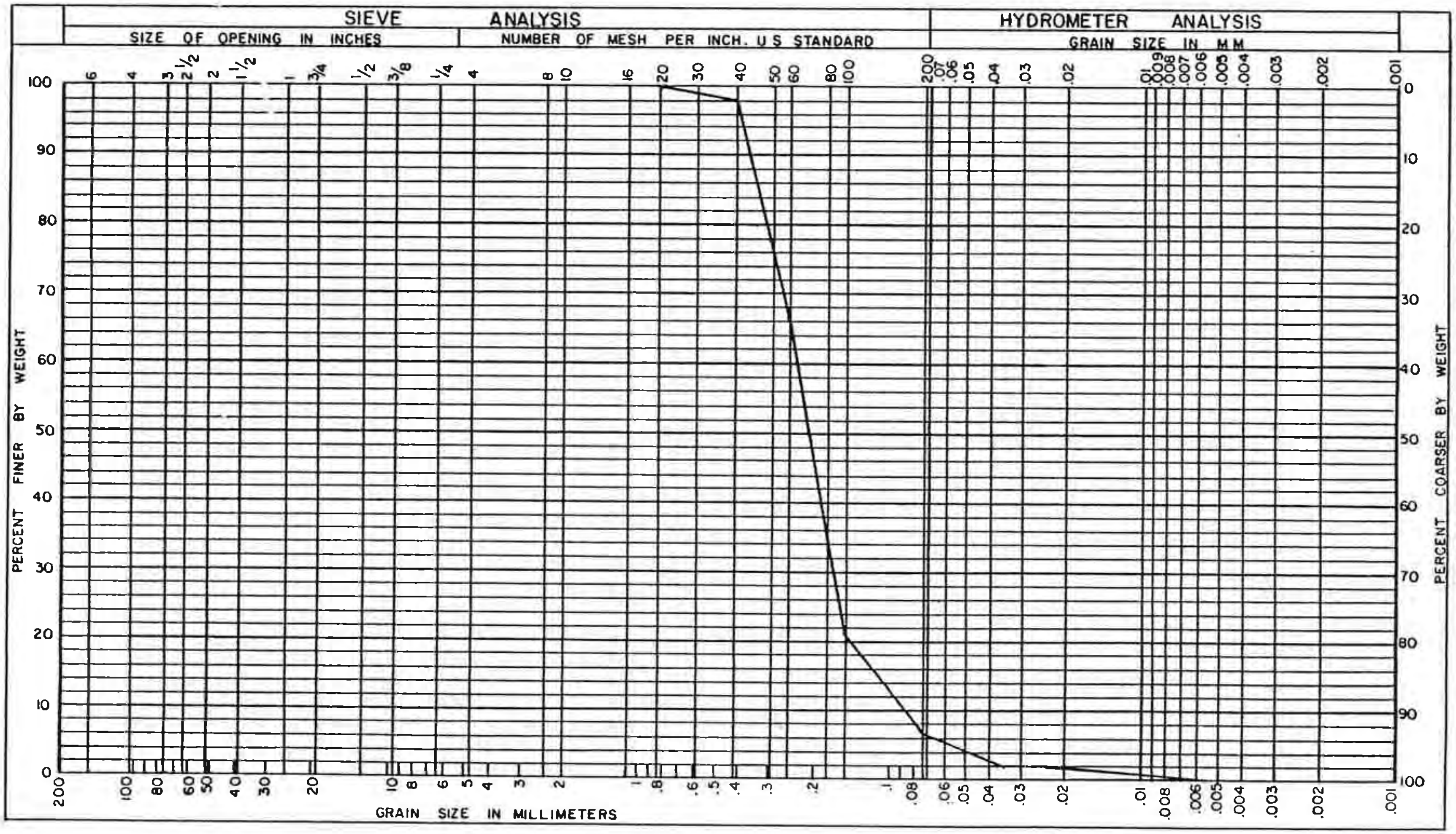
Alaska Testlab

4040 "B" Street Anchorage, Alaska 99503 Phone (907) 278-1551

Sheet 2 of 10
W. O. No. D51343
Date Aug.29,1984
Technician

Textural Class Sand
Frost Class NFS Unified Class SP-SM
Plastic Properties
Date Received

Client Chugach Electric Association
Project Point MacKenzie Substation
Sample Number
Location TB 3, Sa. 4&5, combined, 15' to 21.5'
Sample Taken By



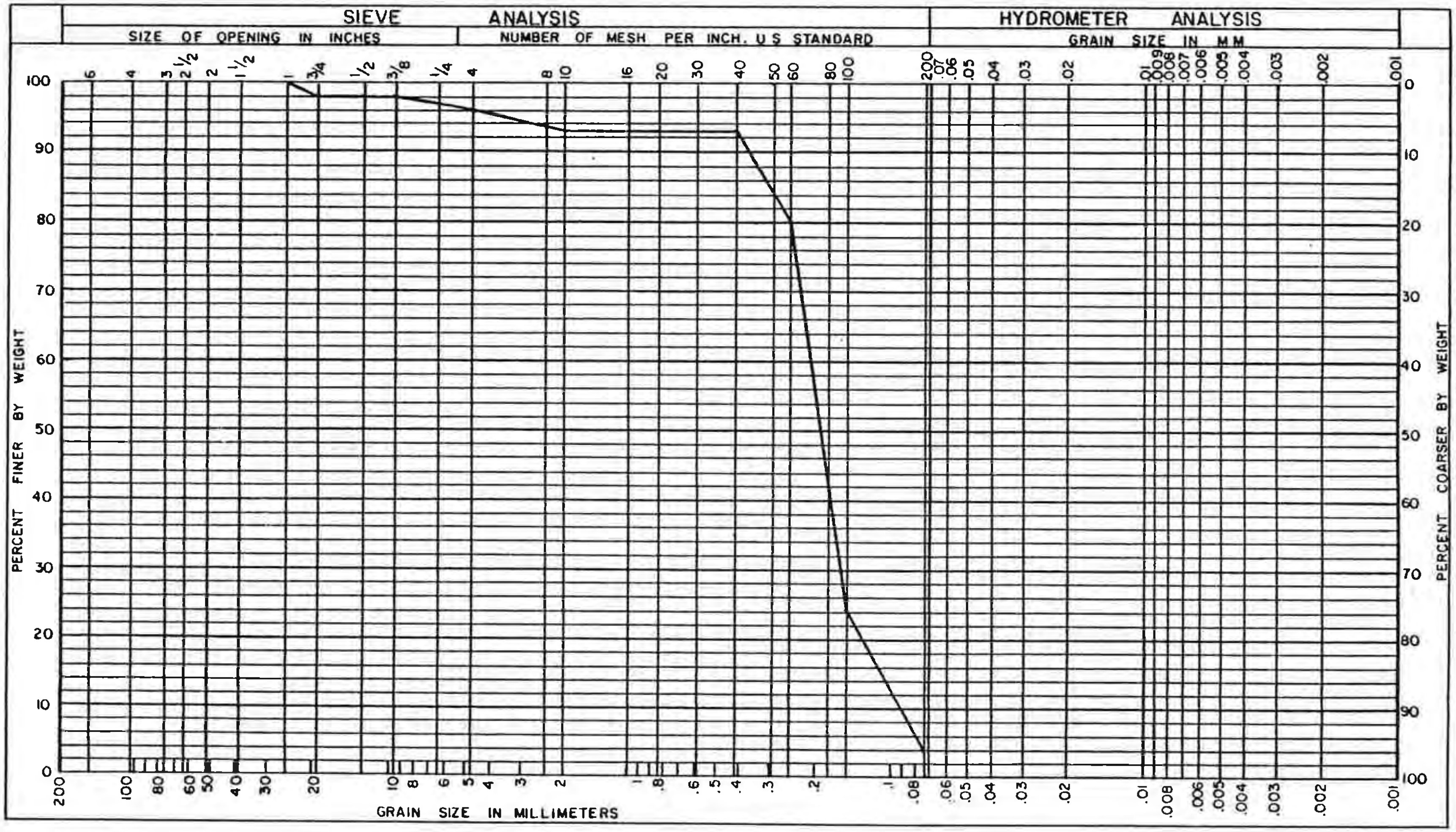
US STD SIEVE	CUM % PASS
3	
2	
1 1/2	
1	
3/4	
1/2	
3/8	
4	
10	
20	100
40	98
100	21
200	7
0.02MM	2

Alaska Testlab
4040 "B" Street Anchorage, Alaska 99503 Phone (907) 278-1551

W. O. No. D51343
Date Aug.29,1984
Technician

Textural Class Sand
Frost Class NFS Unified Class SP
Plastic Properties
Date Received

Client Chugach Electric Association
Project Point MacKenzie Substation
Sample Number
Location TB 3, Sa. 7, 30.0' to 31.5'
Sample Taken By



US STD SIEVE	CUM % PASS
3	
2	
1 1/2	
1	100
3/4	98
1/2	98
3/8	98
4	97
10	93
20	93
40	92
100	24
200	4
0.02MM	

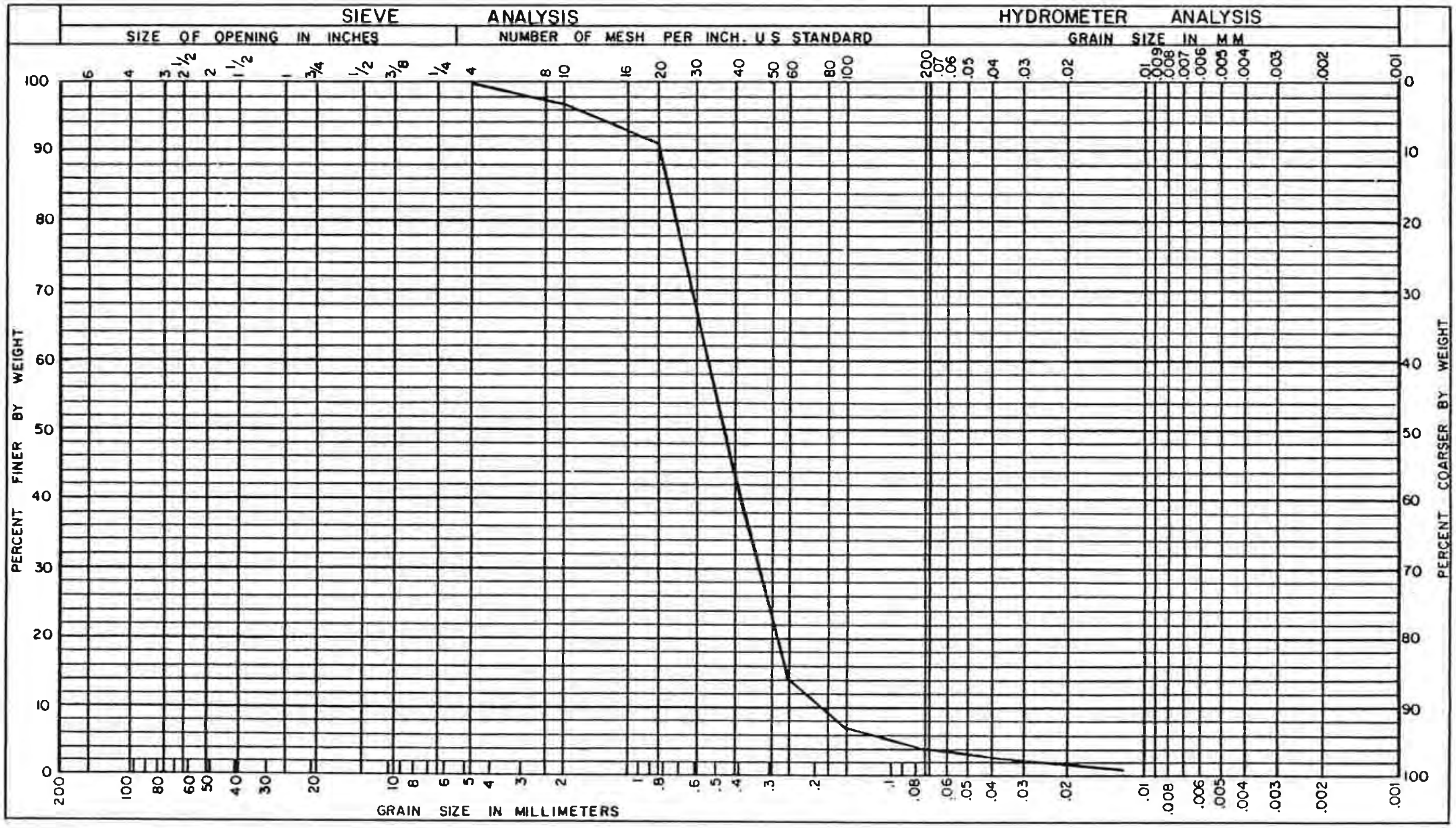
Alaska Testlab

4040 "B" Street Anchorage, Alaska 99503 Phone (907) 278-1551

Sheet 4 of 10
W. O. No. D51343
Date Aug.29,1984
Technician

Textural Class Sand
Frost Class NFS Unified Class SP
Plastic Properties
Date Received

Client Chugach Electric Association
Project Point MacKenzie Substation
Sample Number
Location TB 4, Sa. 2, 5.0' to 6.5'
Sample Taken By



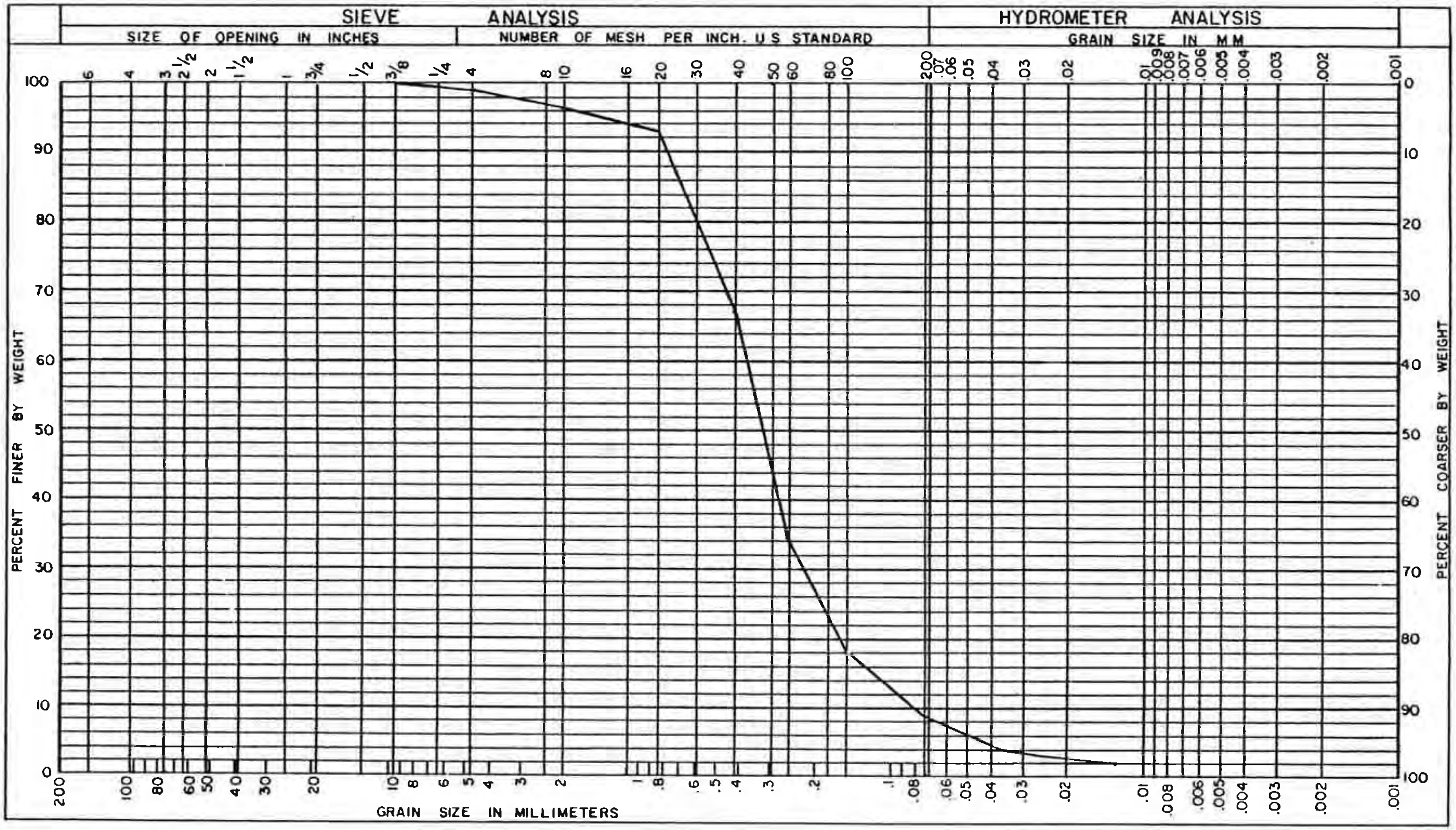
US STD SIEVE	CUM % PASS
3	
2	
1 1/2	
1	
3/4	
1/2	
3/8	
4	100
10	97
20	91
40	44
100	7
200	4
0.02MM	2

Alaska Testlab
4040 "B" Street Anchorage, Alaska 99503 Phone (907) 278-1551

Sheet 5 of 10
W. O. No. D51343
Date Aug. 29, 1984
Technician

Textural Class Sand
Frost Class NFS Unified Class SP-SM
Plastic Properties
Date Received

Client Chugach Electric Association
Project Point MacKenzie Substation
Sample Number
Location TB 5, Sa. 1&2, combined, 2.5' to 6.5'
Sample Taken By



US STD SIEVE	CUM % PASS
3	
2	
1 1/2	
1	
3/4	
1/2	
3/8	100
4	99
10	96
20	93
40	67
100	18
200	9
0.02MM	3

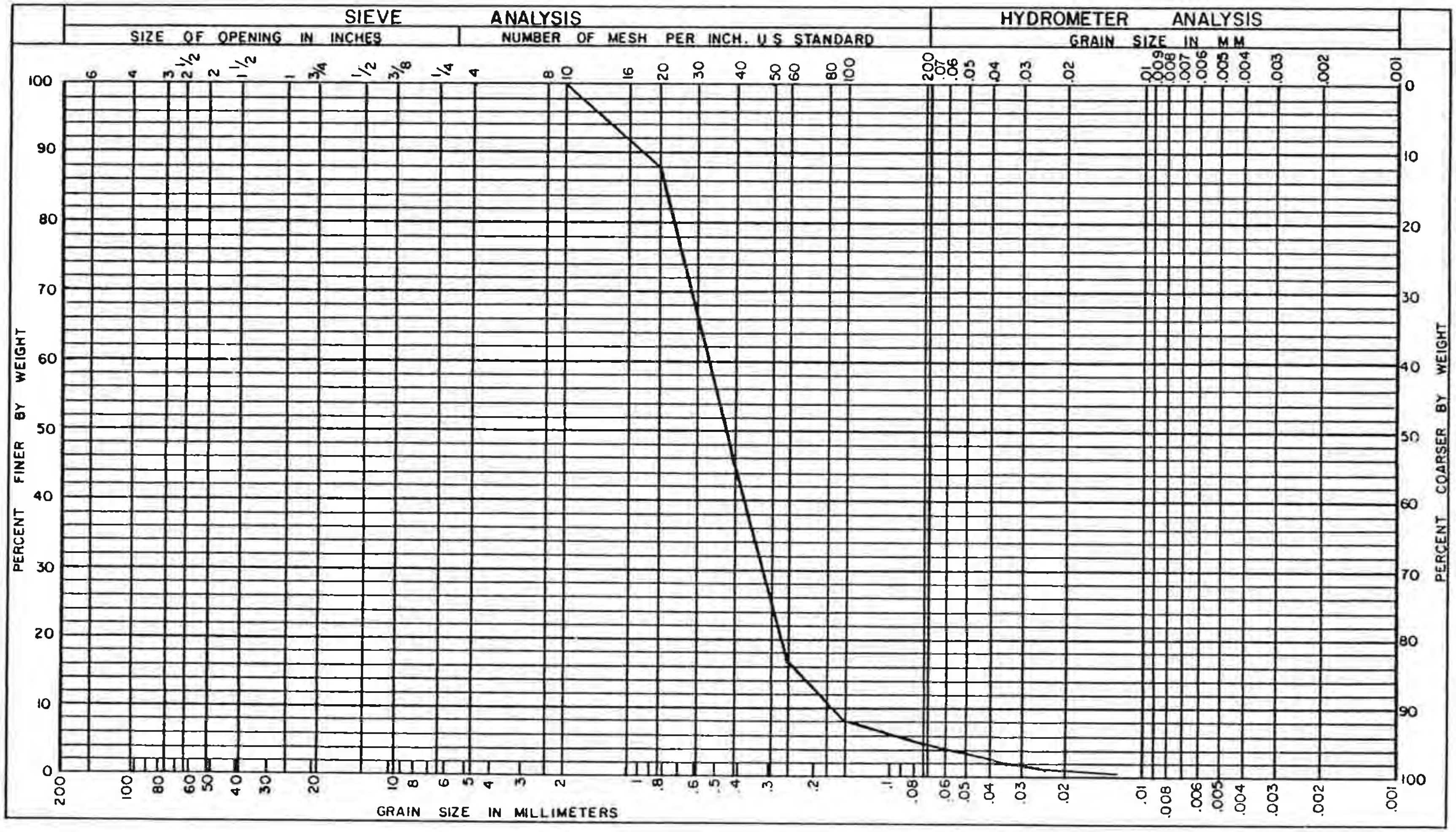
Alaska Testlab

4040 "B" Street Anchorage, Alaska 99503 Phone (907) 278-1551

W. O. No. D51343
Date Aug.29,1984
Technician

Textural Class Sand
Frost Class NFS Unified Class SP-SM
Plastic Properties
Date Received

Client Chugach Electric Association
Project Point MacKenzie Substation
Sample Number
Location TB 5, Sa. 4, 15.0' to 16.5'
Sample Taken By



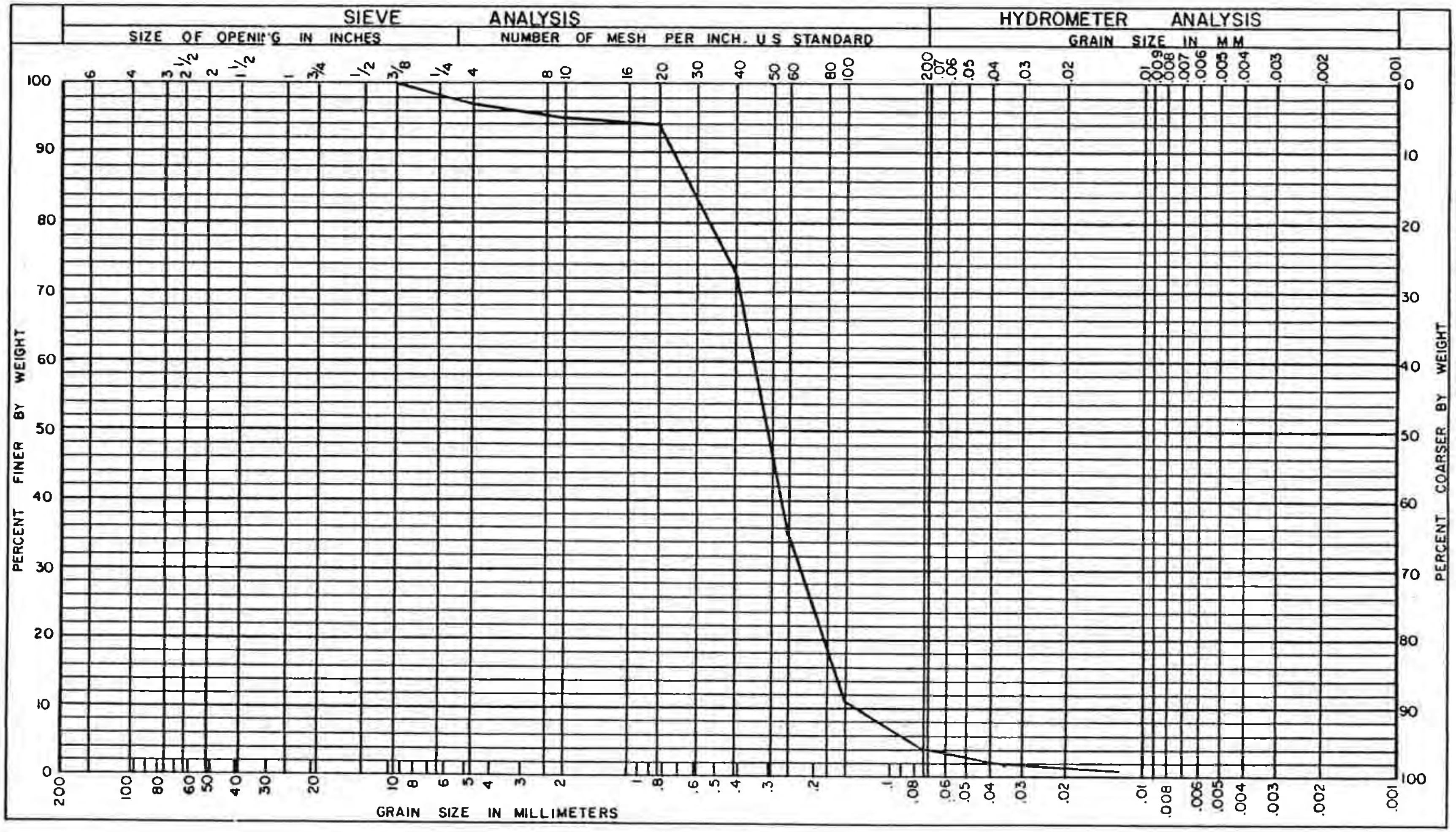
US STD SIEVE	CUM % PASS
3	
2	
1 1/2	
1	
3/4	
1/2	
3/8	
4	
10	100
20	88
40	45
100	8
200	5
0.02MM	1

Alaska Testlab
4040 "B" Street Anchorage, Alaska 99503 Phone (907) 278-1551

Sheet 7 of 10
W. O. No. D51343
Date Aug.29,1984
Technician

Textural Class Sand
Frost Class NFS Unified Class SP
Plastic Properties
Date Received

Client Chugach Electric Association
Project Point MacKenzie Substation
Sample Number
Location TB 6, Sa. 1, 2.5' to 4.0'
Sample Taken By



US STD SIEVE	CUM % PASS
3	
2	
1 1/2	
1	
3/4	
1/2	
3/8	100
4	97
10	95
20	94
40	73
100	11
200	4
0.02MM	1

APPENDIX B

SOIL BORING LOGS AND SPLIT-SPOON SAMPLE PHOTOGRAPHS (NGE-TFT 2019)

Northern Geotechnical Engineering Inc.
d.b.a. Terra Firma Testing
11301 Olive Lane
Anchorage, Alaska 99515
Telephone: 907-344-5934

EXPLORATION B1

NGE-TFT PROJECT NAME: CEA Pt. Mac Substation 230KV Yard Repairs
NGE-TFT PROJECT NUMBER: 5298-19
PROJECT LOCATION: Point MacKenzie, Alaska
EXPLORATION CONTRACTOR: Discovery Drilling, Inc.
EXPLORATION EQUIPMENT: Truck-mounted CME 85
EXPLORATION METHOD: Hollow Stem Auger
SAMPLING METHOD: SPT w/ 140lb autohammer
LOGGED BY: A. Smith
DATE/TIME STARTED: 3/20/2019 @ 9:50:00 AM
DATE/TIME COMPLETED: 3/20/2019 @ 12:00:00 PM
EXPLORATION LOCATION: See report Figure 3
GROUND ELEVATION: Not Known
▽ **GROUNDWATER (ATD):** N/E
▼ **GROUNDWATER ():** N/A
EXPLORATION COMPLETION: Backfilled with cuttings
WEATHER CONDITIONS: Overcast, wind 10-15 mph, 32°F

DEPTH (ft)	MATERIAL DESCRIPTION	FIELD SAMPLE ID	RECOVERY (in)	FIELD BLOWS	$(N_1)_{60}$	LAB SAMPLE ID	LAB RESULTS
0	**SILTY SAND** (SM), olive brown	S1	19	17 19 31	41	S1	S1 MC = 24.5%
		S2	20	16 46 34	66	S2	S2 MC = 10.3% 0.0% gravel, 72.5% sand, 27.5% silt P0.02 = 16.0% FC = F3
5	**COAL**						
	POORLY GRADED SAND (SP), olive gray, damp	S3A	18	12 14 13	38	S3A	S3A MC = 15.1%
	COAL						
	POORLY GRADED SAND (SP), medium dense, olive brown, damp, gravel up to 0.5" in diameter	S4A	15	11 15 12	31	S4A	S4A MC = 5.4%
10	**SILTY SAND** (SM), medium dense, olive brown, damp, non plastic	S5	13	6 7 9	13	S5	S5 MC = 21.6% P200 = 40.9%
15	**POORLY GRADED SAND WITH SILT** (SP-SM), dense, olive gray, damp, gravel up to 0.5" in diameter, some reddish brown staining below 20'	S6	17	14 22 32	50	S6	S6 MC = 3.8% 9.3% gravel, 83.3% sand, 7.4% silt
20	**SILT** (ML), stiff, olive brown, damp, non plastic	S7	18	14 22 20	44	S7	S7 MC = 11.0% P200 = 15.5%
25		S8	18	16 20 23	45	S8	S8 MC = 16.0% LL= NP PL=NP PI=NP

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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Anchorage, Alaska 99515
Telephone: 907-344-5934

EXPLORATION B1

NGE-TFT PROJECT NAME: CEA Pt. Mac Substation 230KV Yard Repairs
NGE-TFT PROJECT NUMBER: 5298-19
PROJECT LOCATION: Point MacKenzie, Alaska
EXPLORATION CONTRACTOR: Discovery Drilling, Inc.
EXPLORATION EQUIPMENT: Truck-mounted CME 85
EXPLORATION METHOD: Hollow Stem Auger
SAMPLING METHOD: SPT w/ 140lb autohammer
LOGGED BY: A. Smith
DATE/TIME STARTED: 3/20/2019 @ 9:50:00 AM
DATE/TIME COMPLETED: 3/20/2019 @ 12:00:00 PM
EXPLORATION LOCATION: See report Figure 3
GROUND ELEVATION: Not Known
GROUNDWATER (ATD): N/E
GROUNDWATER (): N/A
EXPLORATION COMPLETION: Backfilled with cuttings
WEATHER CONDITIONS: Overcast, wind 10-15 mph, 32°F

DEPTH (ft)	MATERIAL DESCRIPTION	FIELD SAMPLE ID	RECOVERY (in)	FIELD BLOWS	$(N_1)_{60}$	LAB SAMPLE ID	LAB RESULTS
	SILT (ML), stiff, olive brown, damp, non plastic *(continued)*						
30		S9	16	11 24 23	49	S9	S9 MC = 9.0%
	SILTY SAND (SM), dense, olive gray, damp						
35		S10	19	16 29 28	63	S10	S10 MC = 9.4% P200 = 45.3%
40		S11	16	15 22 26	53	S11	S11 MC = 4.7% P200 = 21.0%
45							
	POORLY GRADED SAND (SP), dense, olive brown, damp						
50		S12	15	15 22 33	36	S12	S12 MC = 2.0%

Bottom of borehole at 51.5 ft bgs.

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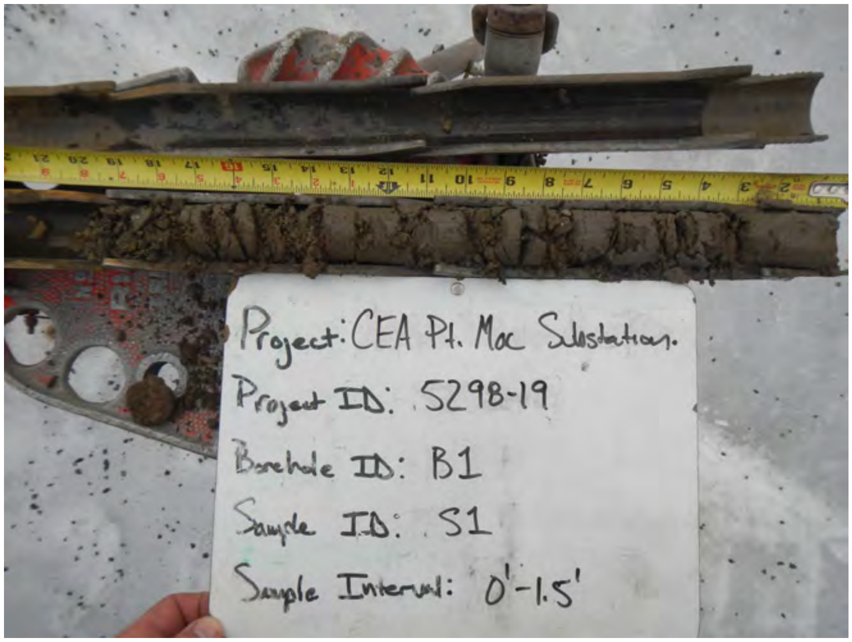
11301 Olive Lane
Anchorage, Alaska 99515
Telephone: 907-344-5934

PHOTO LOG
EXPLORATION B1

CLIENT Electric Power Systems, Inc. **PROJECT NAME** CEA Pt. Mac Substation

PROJECT NUMBER 5298-19 **PROJECT LOCATION** Point MacKenzie



Exploration B1 Sample S1
Sample Interval 0 - 1.5 ft bgs

Exploration B1 Sample S2
Sample Interval 2.5 - 4 ft bgs

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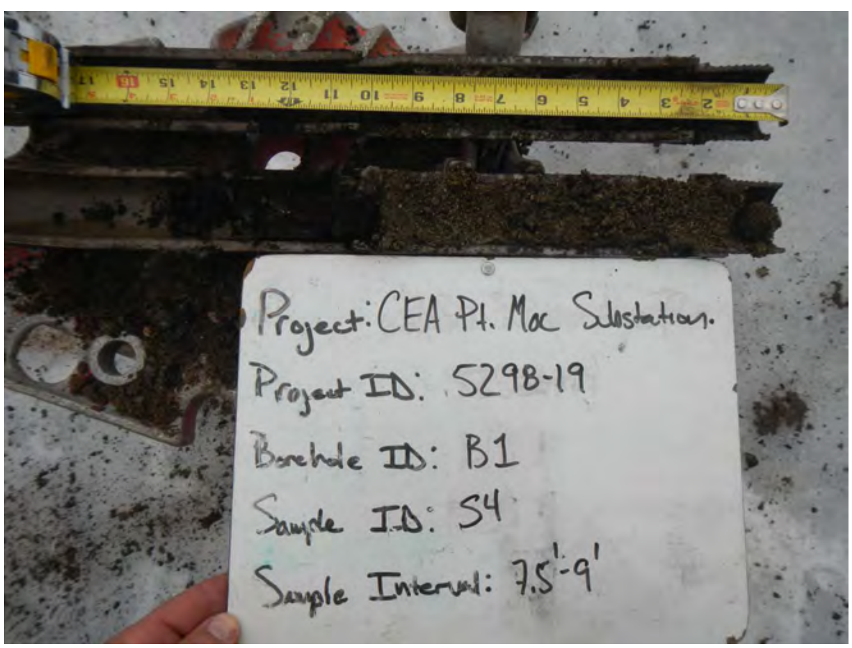
11301 Olive Lane
Anchorage, Alaska 99515
Telephone: 907-344-5934

PHOTO LOG
EXPLORATION B1

CLIENT Electric Power Systems, Inc. **PROJECT NAME** CEA Pt. Mac Substation

PROJECT NUMBER 5298-19 **PROJECT LOCATION** Point MacKenzie

Exploration B1 Sample S3
Sample Interval 5 - 6.5 ft bgs



Exploration B1 Sample S4
Sample Interval 7.5 - 9 ft bgs

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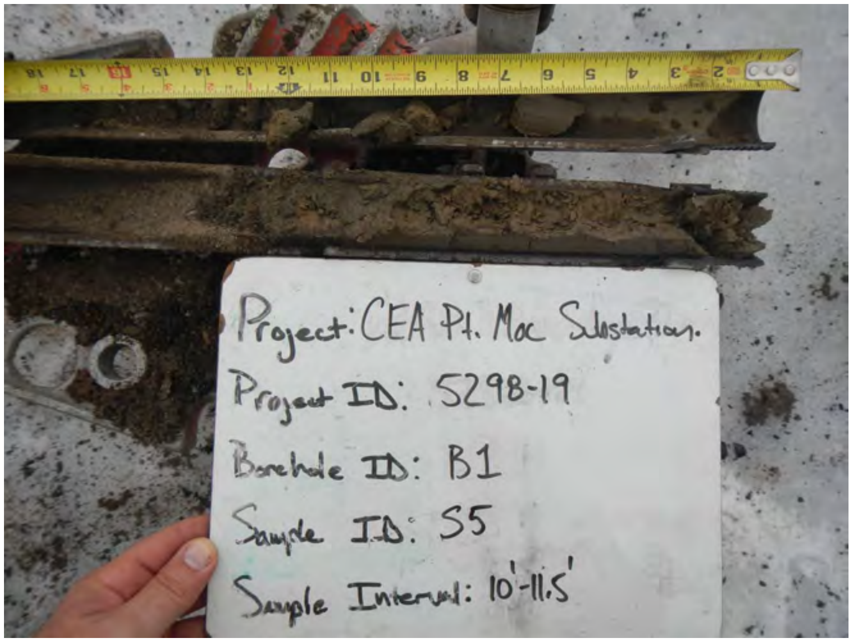
Anchorage, Alaska 99515
Telephone: 907-344-5934

PHOTO LOG
EXPLORATION B1

CLIENT Electric Power Systems, Inc. **PROJECT NAME** CEA Pt. Mac Substation

PROJECT NUMBER 5298-19 **PROJECT LOCATION** Point MacKenzie



Exploration B1 Sample S5
Sample Interval 10 - 11.5 ft bgs

Exploration B1 Sample S6
Sample Interval 15 - 16.5 ft bgs

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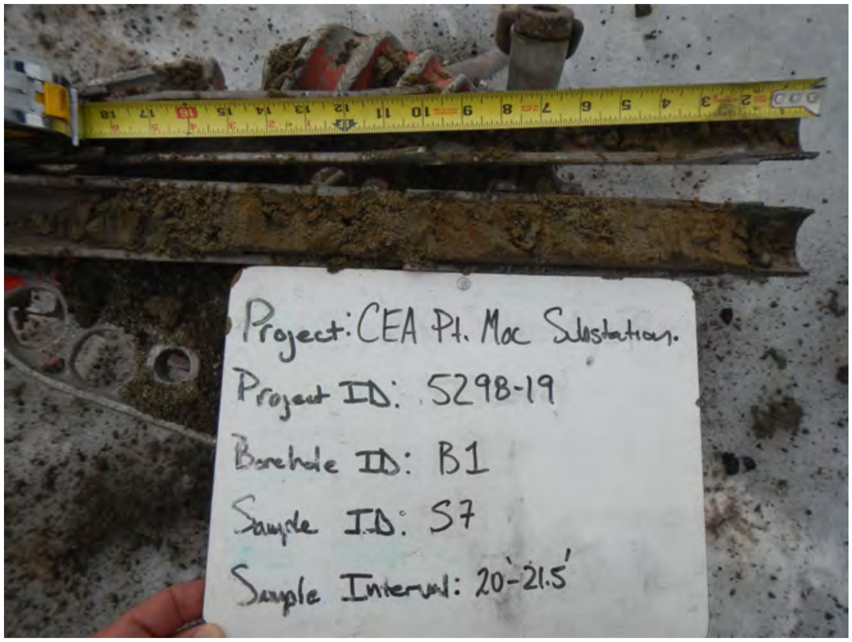
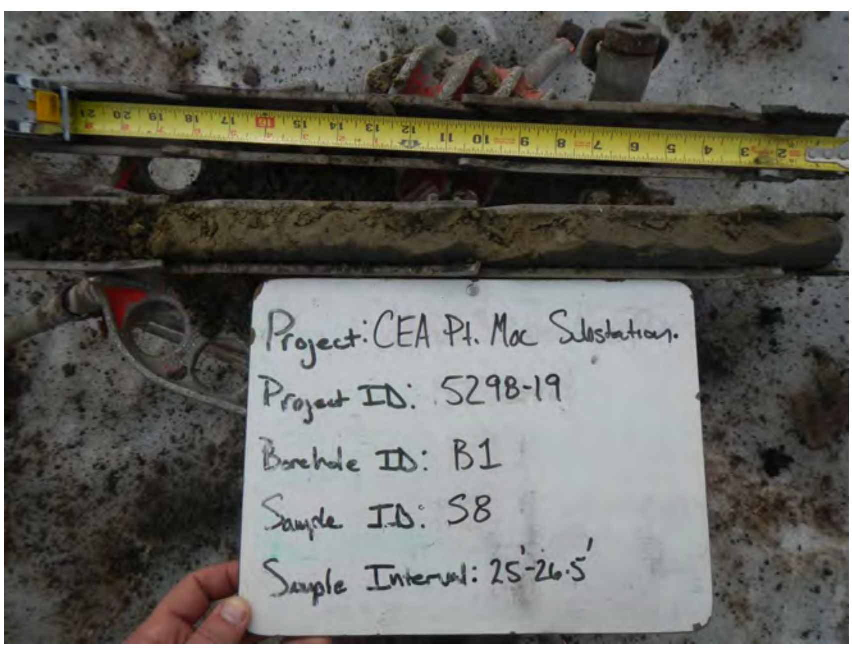
Anchorage, Alaska 99515
Telephone: 907-344-5934

PHOTO LOG
EXPLORATION B1

CLIENT Electric Power Systems, Inc.
PROJECT NAME CEA Pt. Mac Substation
PROJECT NUMBER 5298-19
PROJECT LOCATION Point MacKenzie



Exploration B1 Sample S7
Sample Interval 20 - 21.5 ft bgs



Exploration B1 Sample S8
Sample Interval 25 - 26.5 ft bgs

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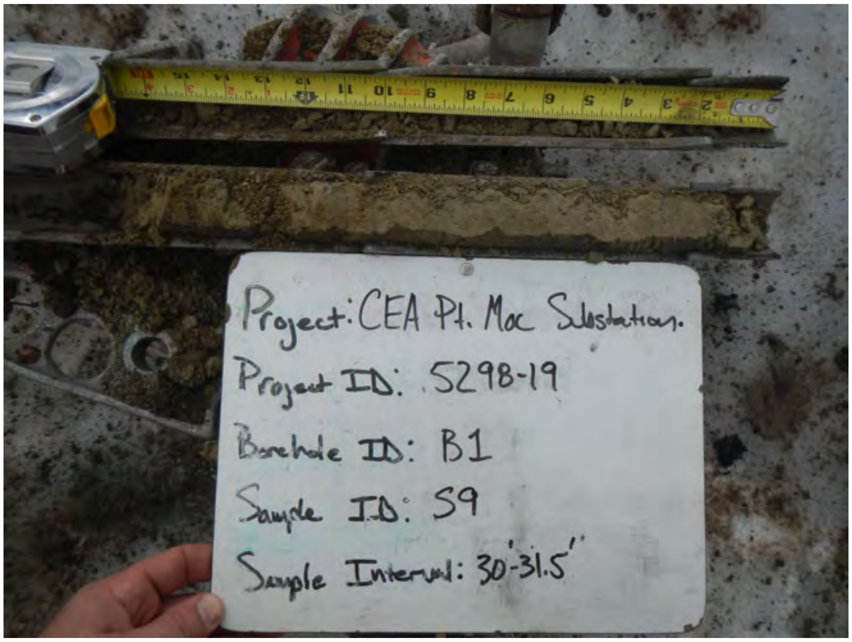
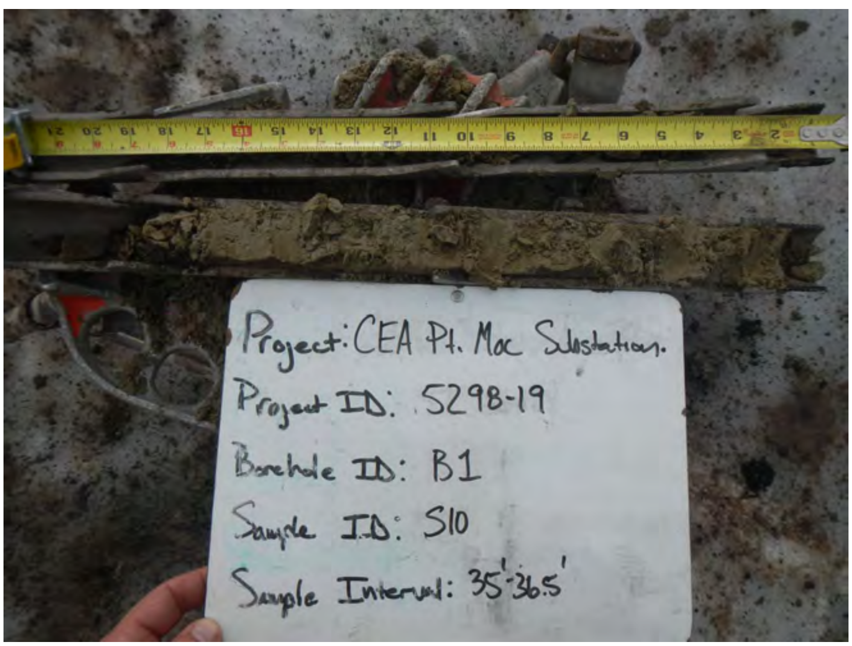
11301 Olive Lane
Anchorage, Alaska 99515
Telephone: 907-344-5934

PHOTO LOG
EXPLORATION B1

CLIENT Electric Power Systems, Inc.
PROJECT NAME CEA Pt. Mac Substation
PROJECT NUMBER 5298-19
PROJECT LOCATION Point MacKenzie



Exploration B1 Sample S9
Sample Interval 30 - 31.5 ft bgs



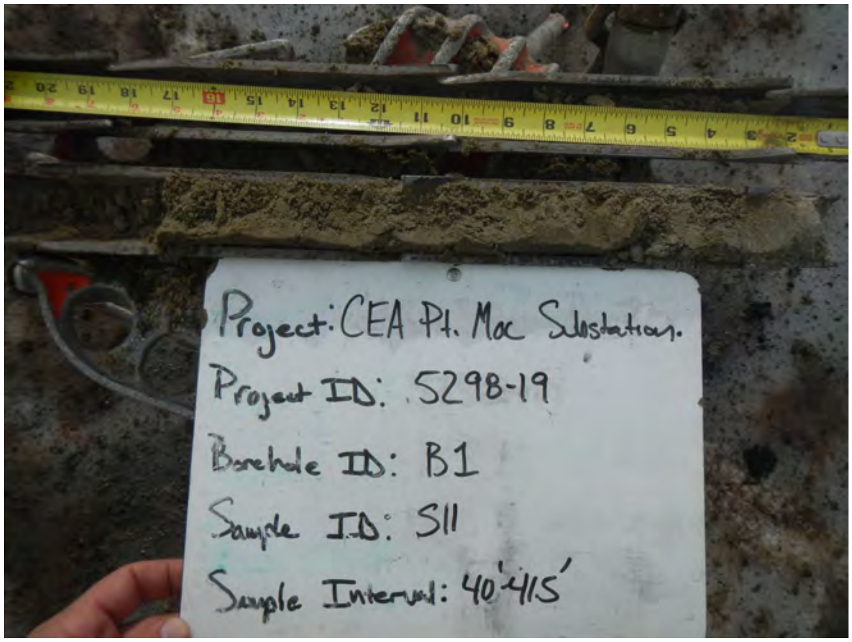
Exploration B1 Sample S10
Sample Interval 35 - 36.5 ft bgs

Northern Geotechnical Engineering Inc. d.b.a. Terra Firma Testing
11301 Olive Lane
Anchorage, Alaska 99515
Telephone: 907-344-5934

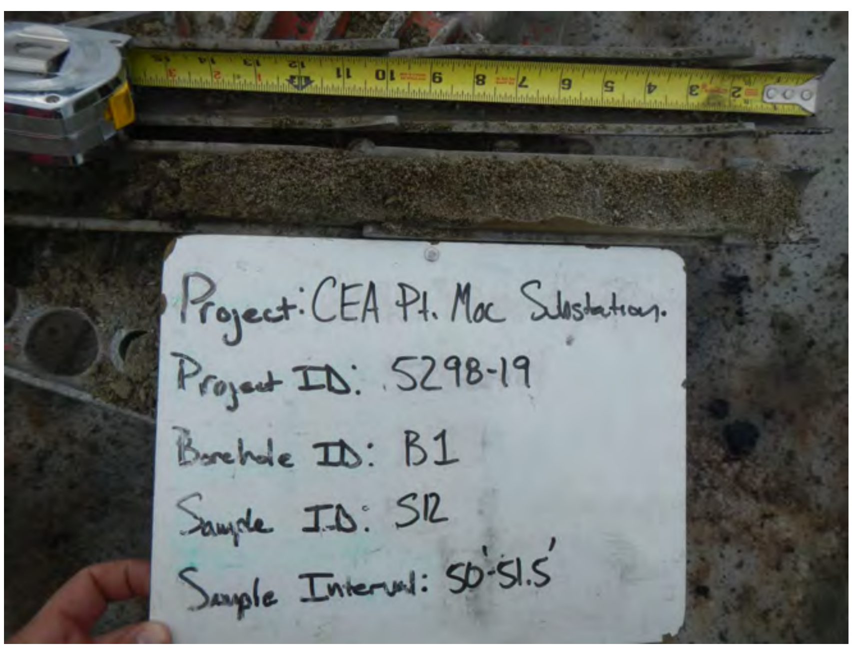
PHOTO LOG
EXPLORATION B1

CLIENT Electric Power Systems, Inc. **PROJECT NAME** CEA Pt. Mac Substation

PROJECT NUMBER 5298-19 **PROJECT LOCATION** Point MacKenzie



Exploration B1 Sample S11
Sample Interval 40 - 41.5 ft bgs



Exploration B1 Sample S12
Sample Interval 50 - 51.5 ft bgs

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d.b.a. Terra Firma Testing
11301 Olive Lane
Anchorage, Alaska 99515
Telephone: 907-344-5934

EXPLORATION B2

NGE-TFT PROJECT NAME: CEA Pt. Mac Substation 230KV Yard Repairs
NGE-TFT PROJECT NUMBER: 5298-19
PROJECT LOCATION: Point MacKenzie, Alaska
EXPLORATION CONTRACTOR: Discovery Drilling, Inc.
EXPLORATION EQUIPMENT: Truck-mounted CME 85
EXPLORATION METHOD: Hollow Stem Auger
SAMPLING METHOD: SPT w/ 140lb autohammer
LOGGED BY: A. Smith
DATE/TIME STARTED: 3/20/2019 @ 12:30:00 PM
DATE/TIME COMPLETED: 3/20/2019 @ 2:30:00 PM
EXPLORATION LOCATION: See report Figure 3
GROUND ELEVATION: Not Known
∇ **GROUNDWATER (ATD):** N/E
▼ **GROUNDWATER ():** N/A
EXPLORATION COMPLETION: Backfilled with cuttings
WEATHER CONDITIONS: Overcast, wind 10-15 mph, 32°F

DEPTH (ft)	MATERIAL DESCRIPTION	FIELD SAMPLE ID	RECOVERY (in)	FIELD BLOWS	$(N_1)_{60}$	LAB SAMPLE ID	LAB RESULTS	REMARKS/NOTES
0	**SILTY SAND** (SM), olive brown	S1	16	14 33 22	45	S1	S1 MC = 9.6% P200 = 16.3%	
5	**POORLY GRADED SAND** (SP), medium dense, olive brown, damp	S2	15	13 13 12	37	S2	S2 MC = 3.5% 0.2% gravel, 94.7% sand, 5.1% silt P0.02 = 4.1% FC = S2	
	COAL							
	SILT (ML), medium stiff, olive brown, damp	S3A	18	5 15 16	26	S3A	S3A MC = 13.0%	
10	**POORLY GRADED SAND WITH GRAVEL** (SP), dense, olive brown, damp, gravel up to 2" in diameter	S4	10	28 45 30 3"	N/A		S4 MC = 3.8%	Some broken rock fragments in cuttings.
15		S5	0	45 30 3"	N/A			No recovery - pushing gravel/cobble ahead of sampler. All sand in cuttings.
20	**SILTY SAND** (SM), dense, olive brown, damp	S6	12	16 23 23	48	S6	S6 MC = 9.0% P200 = 20.8%	
	POORLY GRADED SAND (SP), medium dense, olive gray, damp							
25		S7	16	8 14 14	23	S7	S7 MC = 4.8%	

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)

Northern Geotechnical Engineering Inc.
d.b.a. Terra Firma Testing
11301 Olive Lane
Anchorage, Alaska 99515
Telephone: 907-344-5934

EXPLORATION B2

NGE-TFT PROJECT NAME: CEA Pt. Mac Substation 230KV Yard Repairs
NGE-TFT PROJECT NUMBER: 5298-19
PROJECT LOCATION: Point MacKenzie, Alaska
EXPLORATION CONTRACTOR: Discovery Drilling, Inc.
EXPLORATION EQUIPMENT: Truck-mounted CME 85
EXPLORATION METHOD: Hollow Stem Auger
SAMPLING METHOD: SPT w/ 140lb autohammer
LOGGED BY: A. Smith
DATE/TIME STARTED: 3/20/2019 @ 12:30:00 PM
DATE/TIME COMPLETED: 3/20/2019 @ 2:30:00 PM
EXPLORATION LOCATION: See report Figure 3
GROUND ELEVATION: Not Known
▽ **GROUNDWATER (ATD):** N/E
▼ **GROUNDWATER ():** N/A
EXPLORATION COMPLETION: Backfilled with cuttings
WEATHER CONDITIONS: Overcast, wind 10-15 mph, 32°F

DEPTH (ft)	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
			POORLY GRADED SAND (SP), medium dense, olive gray, damp *(continued)*									
30					S8	14	10 13 11	18		S8	S8 MC = 3.4% 0.0% gravel, 95.8% sand, 4.2% silt	
35					S9	16	10 15 16	24		S9	S9 MC = 4.7%	
40					S10	16	10 12 16	18		S10	S10 MC = 2.9% P200 = 1.8%	

Bottom of borehole at 41.5 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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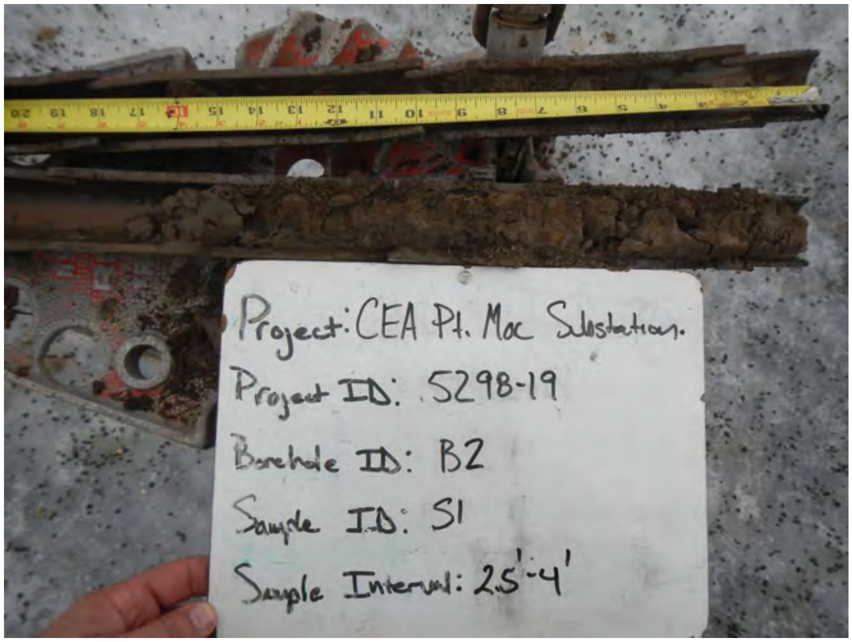
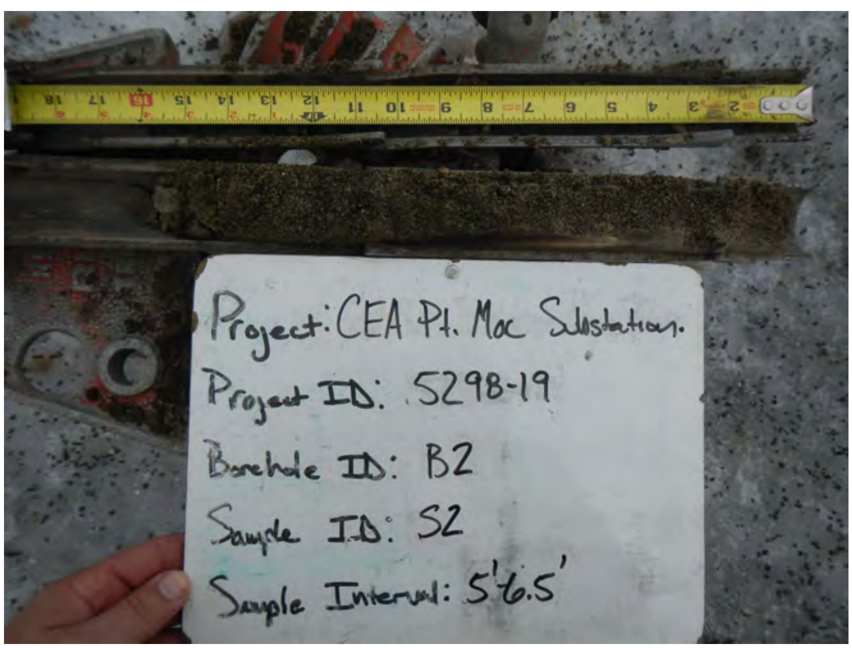
Anchorage, Alaska 99515
Telephone: 907-344-5934

PHOTO LOG
EXPLORATION B2

CLIENT Electric Power Systems, Inc. **PROJECT NAME** CEA Pt. Mac Substation
PROJECT NUMBER 5298-19 **PROJECT LOCATION** Point MacKenzie



Exploration B2 Sample S1
Sample Interval 2.5 - 4 ft bgs

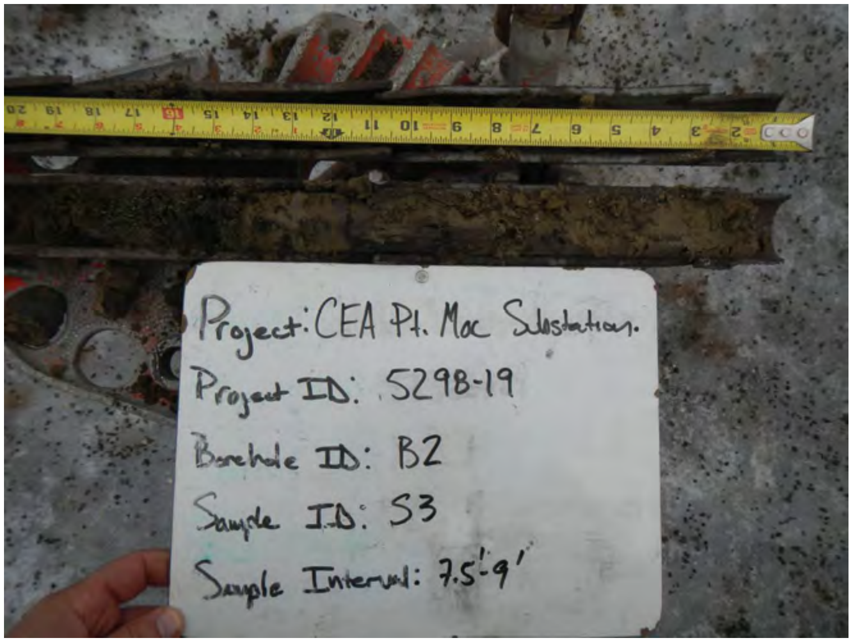


Exploration B2 Sample S2
Sample Interval 5 - 6.5 ft bgs

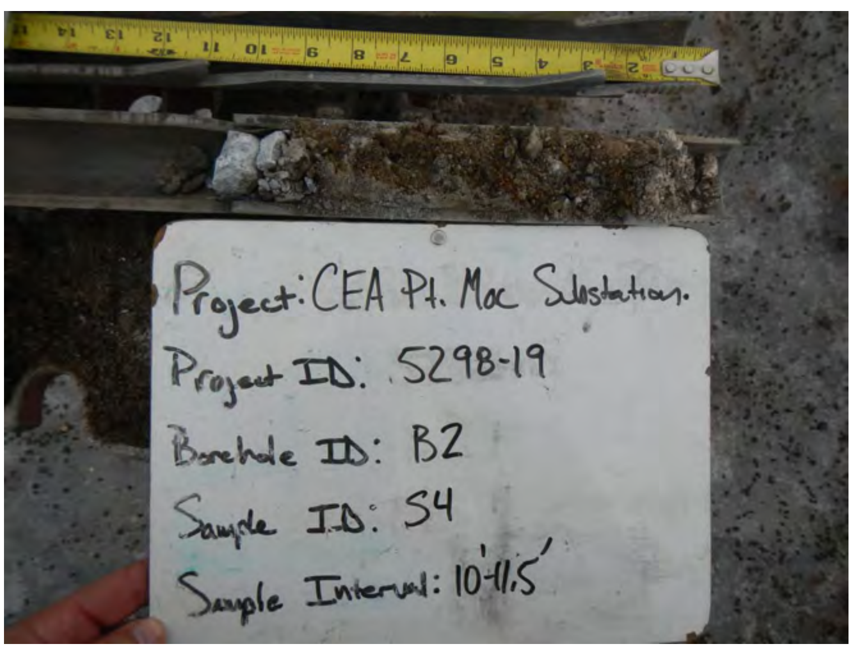
PHOTO LOG
EXPLORATION B2

CLIENT Electric Power Systems, Inc. **PROJECT NAME** CEA Pt. Mac Substation

PROJECT NUMBER 5298-19 **PROJECT LOCATION** Point MacKenzie



Exploration B2 Sample S3
Sample Interval 7.5 - 9 ft bgs



Exploration B2 Sample S4
Sample Interval 10 - 11.5 ft bgs

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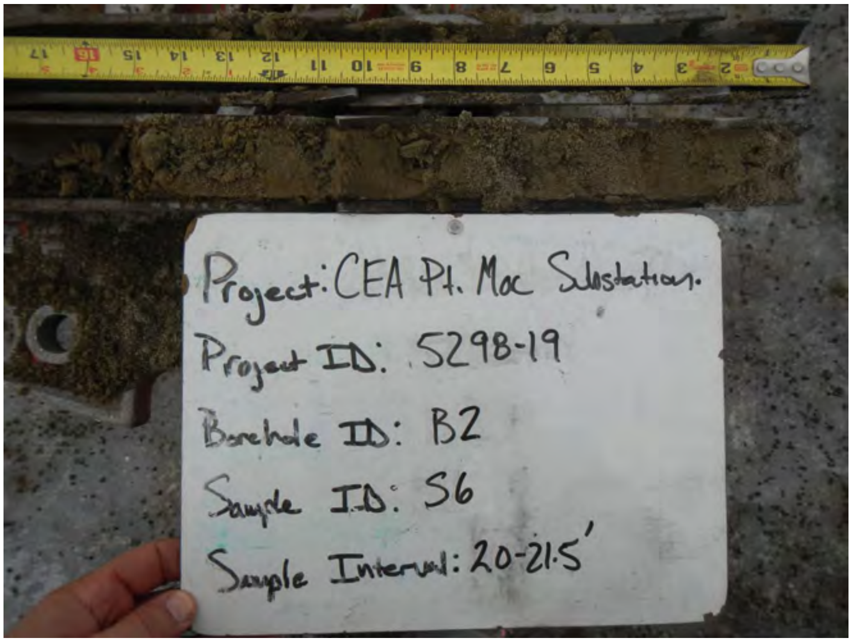
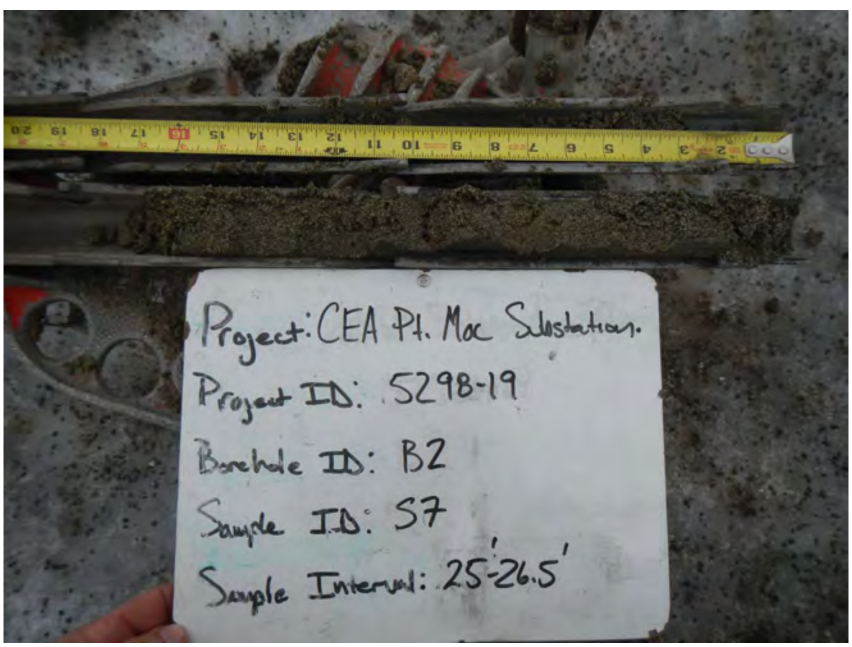
PHOTO LOG
EXPLORATION B2

CLIENT Electric Power Systems, Inc.
PROJECT NAME CEA Pt. Mac Substation
PROJECT NUMBER 5298-19
PROJECT LOCATION Point MacKenzie



Exploration B2 Sample S6
Sample Interval 20 - 21.5 ft bgs



Exploration B2 Sample S7
Sample Interval 25 - 26.5 ft bgs

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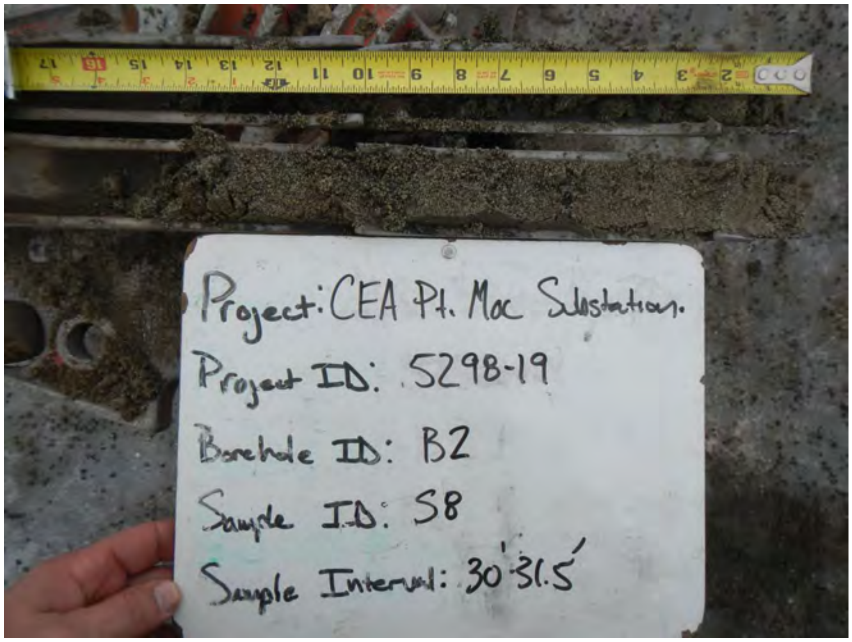
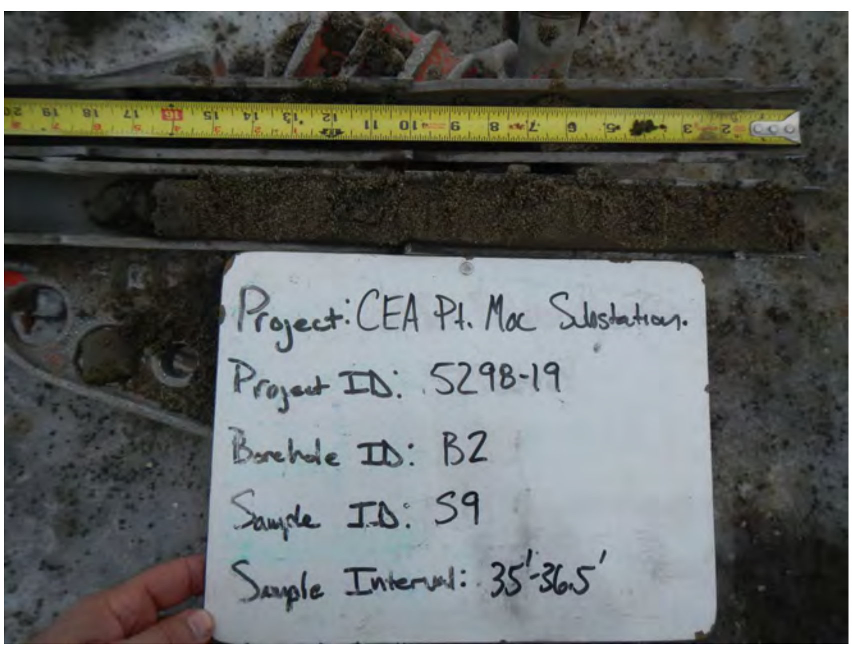
PHOTO LOG
EXPLORATION B2

CLIENT Electric Power Systems, Inc. **PROJECT NAME** CEA Pt. Mac Substation

PROJECT NUMBER 5298-19 **PROJECT LOCATION** Point MacKenzie



Exploration B2 Sample S8
Sample Interval 30 - 31.5 ft bgs



Exploration B2 Sample S9
Sample Interval 35 - 36.5 ft bgs

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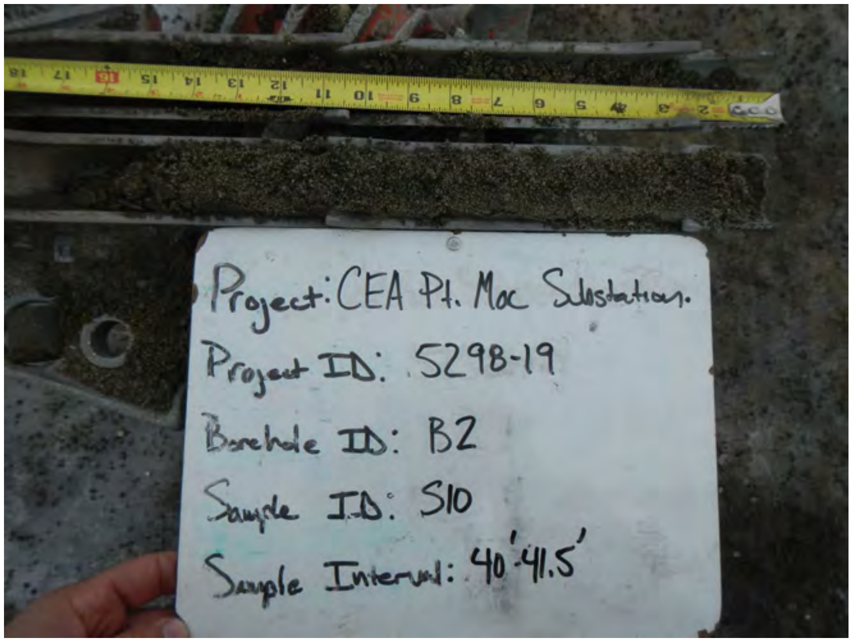
PHOTO LOG
EXPLORATION B2

CLIENT Electric Power Systems, Inc. **PROJECT NAME** CEA Pt. Mac Substation

PROJECT NUMBER 5298-19 **PROJECT LOCATION** Point MacKenzie



Exploration B2 Sample S10
Sample Interval 40 - 41.5 ft bgs

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EXPLORATION LEGEND

CLIENT Electric Power Systems, Inc.

NGE-TFT PROJECT NAME CEA Pt. MacKenzie Substation

NGE-TFT PROJECT NUMBER 5298-19

PROJECT LOCATION Point MacKenzie, Alaska

LITHOLOGIC SYMBOLS (Unified Soil Classification System)

COAL: Coal

ML: USCS Silt

SM: USCS Silty Sand

SP: USCS Poorly-graded Sand

SP-SM: USCS Poorly-graded Sand with Silt

SAMPLER SYMBOLS

Standard Penetration Test

WELL CONSTRUCTION SYMBOLS

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

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

∇ Water Level at Time Drilling, or as Shown

$\blacktriangledown$ Water Level After 24 Hours, or as Shown

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SOIL CLASSIFICATION CHART

CLIENT Electric Power Systems, Inc.

PROJECT NAME CEA Pt. MacKenzie Substation

NGE-TFT PROJECT NUMBER 5298-19

PROJECT LOCATION Point MacKenzie, Alaska

MAJOR DIVISIONS			SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
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.

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EXPLORATION LOG KEY

CLIENT Electric Power Systems, Inc.
PROJECT NAME CEA Pt. MacKenzie Substation
NGE-TFT PROJECT NUMBER 5298-19
PROJECT LOCATION Point MacKenzie, Alaska

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 CEA Pt. MacKenzie Substation

NGE-TFT PROJECT NUMBER 5298-19

PROJECT LOCATION Point MacKenzie, Alaska

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 C

LABORATORY TEST RESULTS
(NGE-TFT 2019)

Summary of Laboratory Test Results

CEA Pt. MacKenzie Substation

230kV Yard Repairs/Upgrades

Pt. MacKenzie, Alaska

NGE-TFT Project #:5298-19

Exploration ID	**Sample Number**	**Depth Interval** (ft) Top	(ft) Bottom	**Moisture Content** ASTM D2216 (% By Dry Mass)	**Atterberg Limits** ASTM D4318 LL	PL	PI	**Particle Size Analysis** ASTM C136/D422/D6913 (% By Mass) Gravel	Sand	Silt/Clay	**Passing #200** ASTM D1140 (% By Mass)	**Passing 0.02mm** ASTM D422 (% By Mass)	**Frost Class.**	**Unified Soil Classification** ASTM D2487
B1	S1	0.0	1.5	24.5										
B1	S2	2.5	4.0	10.3				0.0	72.5	27.5		16.0	F3	(SM) Silty sand
B1	S3	6.0	6.5	15.1										
B1	S4	8.0	9.0	5.4										
B1	S5	10.0	11.5	21.6							40.9			
B1	S6	15.0	16.5	3.8				9.3	83.3	7.4				(SP-SM) Poorly-graded sand w/ silt
B1	S7	20.0	21.5	11.0							15.5			
B1	S8	25.0	26.5	16.0	NP	NP	NP							
B1	S9	30.0	31.5	9.0										
B1	S10	35.0	36.5	9.4							45.3			
B1	S11	40.0	41.5	4.7							21.0			
B1	S12	50.0	51.5	2.0										
B2	S1	2.5	4.0	9.6							16.3			
B2	S2	5.0	6.5	3.5				0.2	94.7	5.1		4.1	S2	(SP-SM) Poorly-graded sand w/ silt
B2	S3	8.1	9.0	13.0										
B2	S4	10.0	11.3	3.8										
B2	S6	20.0	21.5	9.0							20.8			
B2	S7	25.0	26.5	4.8										
B2	S8	30.0	31.5	3.4				0.0	95.8	4.2				(SP) Poorly-graded sand
B2	S9	35.0	36.5	4.7										
B2	S10	40.0	41.5	2.9							1.8			

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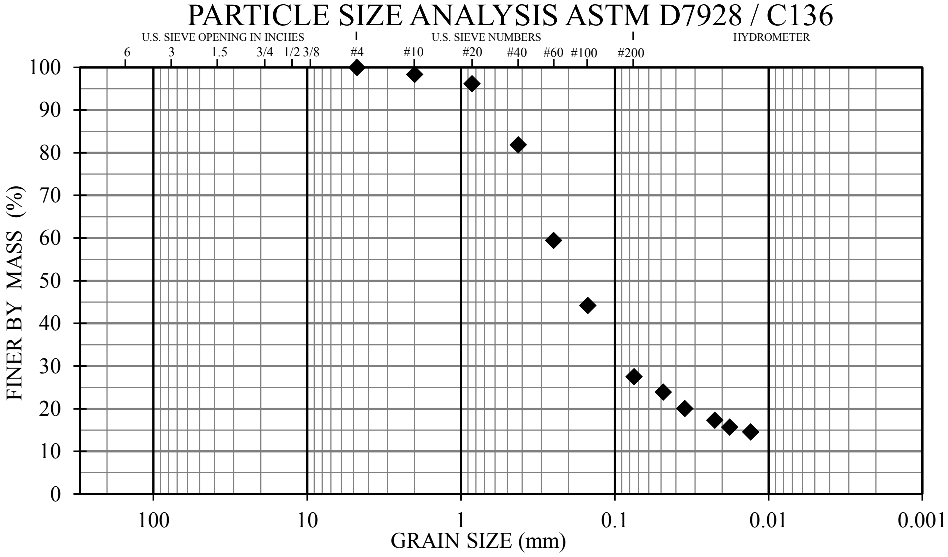
PROJECT CLIENT:	**EPS**
PROJECT NAME:	**CEA Pt. Mac Substation**
PROJECT NO.:	**5298-19**
SAMPLE LOC.:	**B1**
NUMBER/ DEPTH:	**S2 / 2.5 - 4'**
DESCRIPTION:	**Silty sand**
DATE RECEIVED:	**3/21/2019**
TESTED BY:	**JA**
REVIEWED BY:	**ACS**

% GRAVEL	**0.0**	USCS	**SM**
% SAND	**72.5**	USACOE FC	**F3**
% SILT/CLAY	**27.5**	% PASS. 0.02 mm	**16.0**
% MOIST. CONTENT	**10.3**	% PASS. 0.002 mm	**N/A**
UNIFORMITY COEFFICIENT (C_u)		**UNKNOWN**	
COEFFICIENT OF GRADATION (C_c)		**UNKNOWN**	
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	

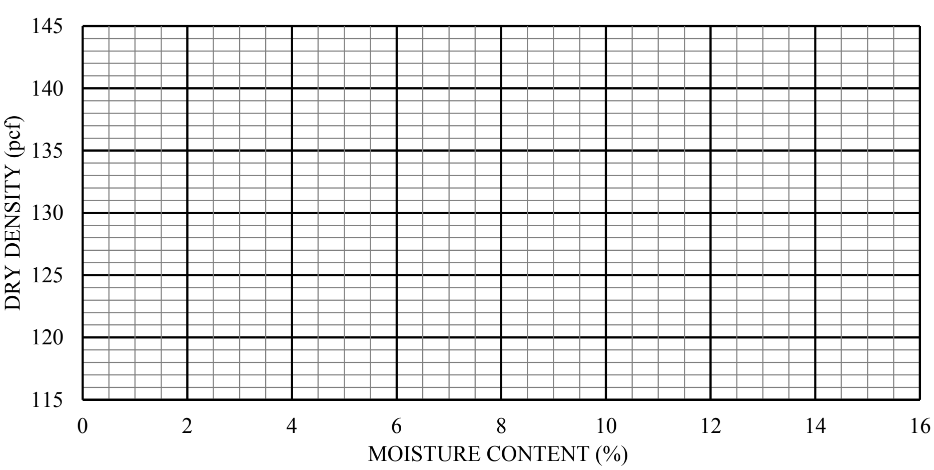
SIEVE ANALYSIS RESULT

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"		
12.70	1/2"		
9.50	3/8"		
4.75	#4	**100**	
2.00	#10	**98**	
0.85	#20	**96**	
0.43	#40	**82**	
0.25	#60	**59**	
0.15	#100	**44**	
0.075	#200	**27.5**	

HYDROMETER RESULT

ELAPSED TIME (MIN)	DIAMETER (mm)	TOTAL % PASSING
0		
1	0.0485	23.9
2	0.0351	20.0
5	0.0224	17.3
8	0.0179	15.7
15	0.0131	14.6
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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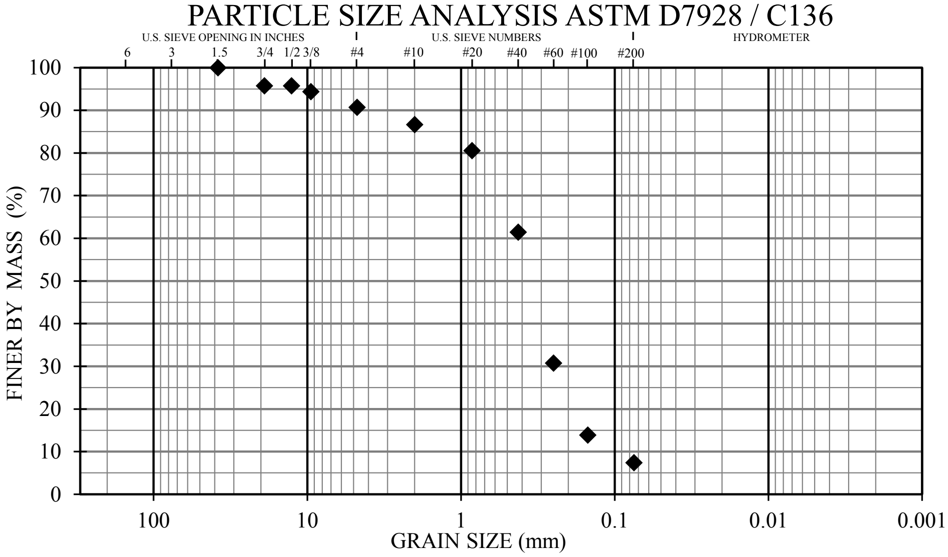
PROJECT CLIENT:	**EPS**
PROJECT NAME:	**CEA Pt. Mac Substation**
PROJECT NO.:	**5298-19**
SAMPLE LOC.:	**B1**
NUMBER/ DEPTH:	**S6 / 15 - 16.5'**
DESCRIPTION:	**Poorly-graded sand w/ silt**
DATE RECEIVED:	**3/21/2019**
TESTED BY:	**JA**
REVIEWED BY:	**ACS**

% GRAVEL	**9.3**	USCS	**SP-SM**
% SAND	**83.3**	USACOE FC	**N/A**
% SILT/CLAY	**7.4**	% PASS. 0.02 mm	**N/A**
% MOIST. CONTENT	**3.8**	% PASS. 0.002 mm	**N/A**

UNIFORMITY COEFFICIENT (C_u)	**4.0**
COEFFICIENT OF GRADATION (C_c)	**1.4**
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	

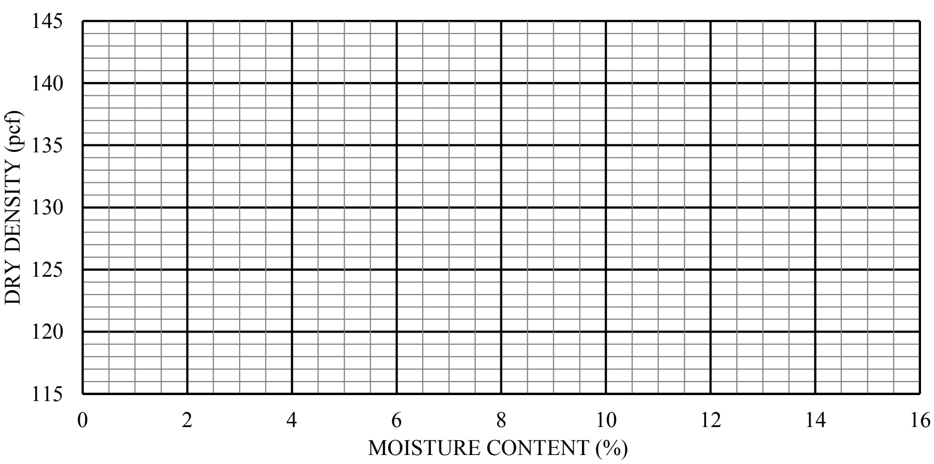
SIEVE ANALYSIS RESULT

SIEVE SIZE (mm)	SIEVE SIZE (U.S.)	TOTAL % PASSING	SPECIFICATION (% PASSING)
152.40	6"		
76.20	3"		
38.10	1.5"	**100**	
19.00	3/4"	**96**	
12.70	1/2"	**96**	
9.50	3/8"	**94**	
4.75	#4	**91**	
2.00	#10	**87**	
0.85	#20	**81**	
0.43	#40	**61**	
0.25	#60	**31**	
0.15	#100	**14**	
0.075	#200	**7.4**	

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**

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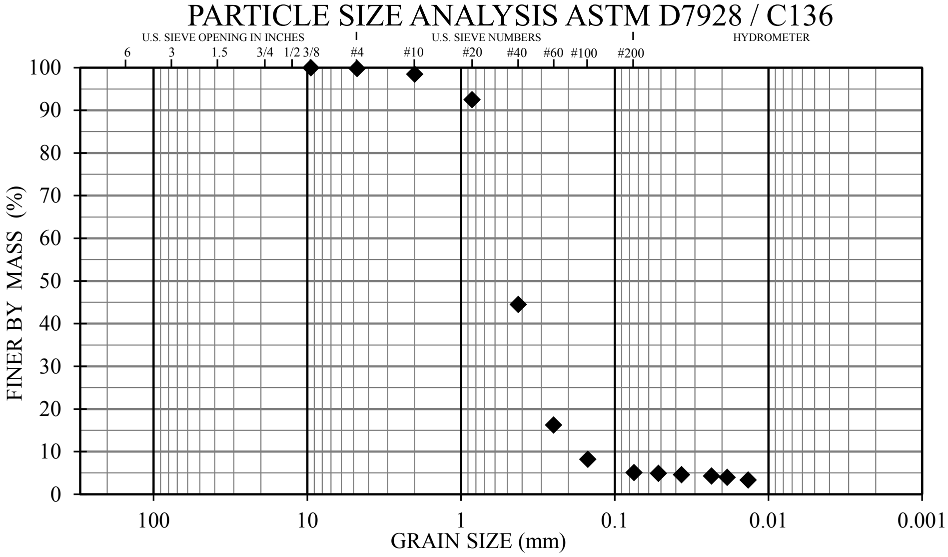
PROJECT CLIENT:	**EPS**
PROJECT NAME:	**CEA Pt. Mac Substation**
PROJECT NO.:	**5298-19**
SAMPLE LOC.:	**B2**
NUMBER/ DEPTH:	**S2 / 5 - 6.5'**
DESCRIPTION:	**Poorly-graded sand w/ silt**
DATE RECEIVED:	**3/21/2019**
TESTED BY:	**JA**
REVIEWED BY:	**ACS**

% GRAVEL	**0.2**	USCS	**SP-SM**
% SAND	**94.7**	USACOE FC	**S2**
% SILT/CLAY	**5.1**	% PASS. 0.02 mm	**4.1**
% MOIST. CONTENT	**3.5**	% PASS. 0.002 mm	**N/A**

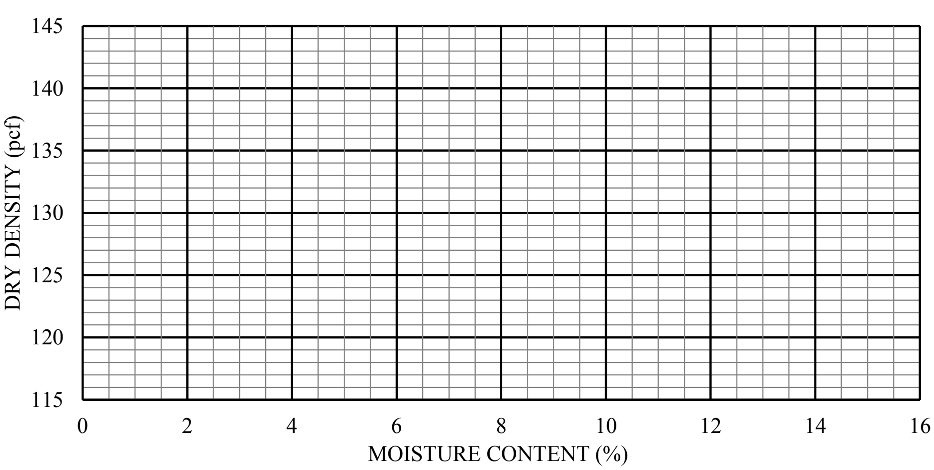
UNIFORMITY COEFFICIENT (C_u)	**3.3**
COEFFICIENT OF GRADATION (C_c)	**1.2**
ASTM D1557 (uncorrected)	**N/A**
ASTM D4718 (corrected)	**N/A**
OPTIMUM MOIST. CONTENT. (corrected)	**N/A**

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"		
19.00	3/4"		
12.70	1/2"		
9.50	3/8"	**100**	
4.75	#4	**100**	
2.00	#10	**98**	
0.85	#20	**92**	
0.43	#40	**45**	
0.25	#60	**16**	
0.15	#100	**8**	
0.075	#200	**5.1**	

HYDROMETER RESULT

ELAPSED TIME (MIN)	DIAMETER (mm)	TOTAL % PASSING
0		
1	0.0521	4.9
2	0.0368	4.6
5	0.0235	4.3
8	0.0185	3.9
15	0.0135	3.3
30		
60		
250		
1440		

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

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NORTHERN GEOTECHNICAL ENGINEERING, INC. / TERRA FIRMA TESTING

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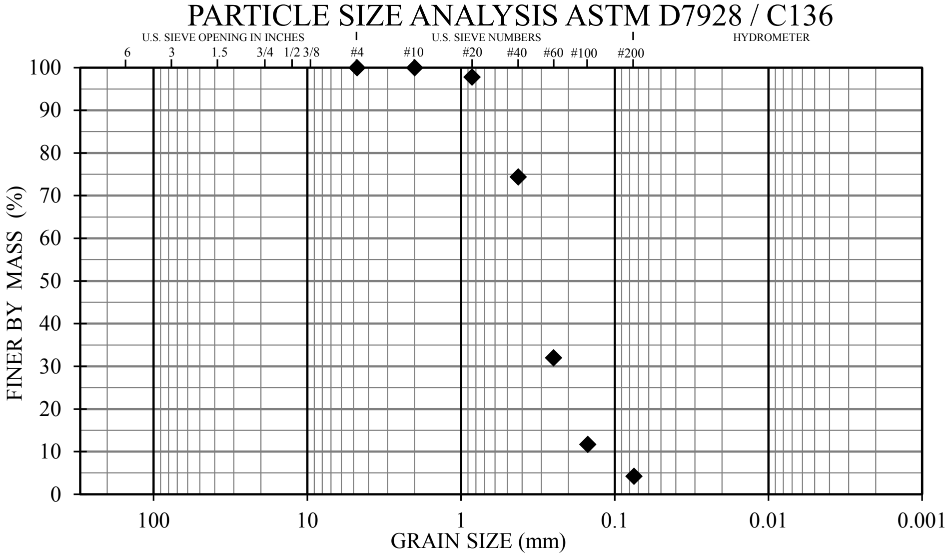
PROJECT CLIENT:	**EPS**
PROJECT NAME:	**CEA Pt. Mac Substation**
PROJECT NO.:	**5298-19**
SAMPLE LOC.:	**B2**
NUMBER/ DEPTH:	**S8 / 30 - 31.5'**
DESCRIPTION:	**Poorly-graded sand**
DATE RECEIVED:	**3/21/2019**
TESTED BY:	**JA**
REVIEWED BY:	**ACS**

% GRAVEL	**0.0**	USCS	**SP**
% SAND	**95.8**	USACOE FC	**N/A**
% SILT/CLAY	**4.2**	% PASS. 0.02 mm	**N/A**
% MOIST. CONTENT	**3.4**	% PASS. 0.002 mm	**N/A**

UNIFORMITY COEFFICIENT (C_u)	**2.7**
COEFFICIENT OF GRADATION (C_c)	**1.2**
ASTM D1557 (uncorrected)	**N/A**
ASTM D4718 (corrected)	**N/A**
OPTIMUM MOIST. CONTENT. (corrected)	**N/A**

PARTICLE SIZE ANALYSIS ASTM D7928 / C136

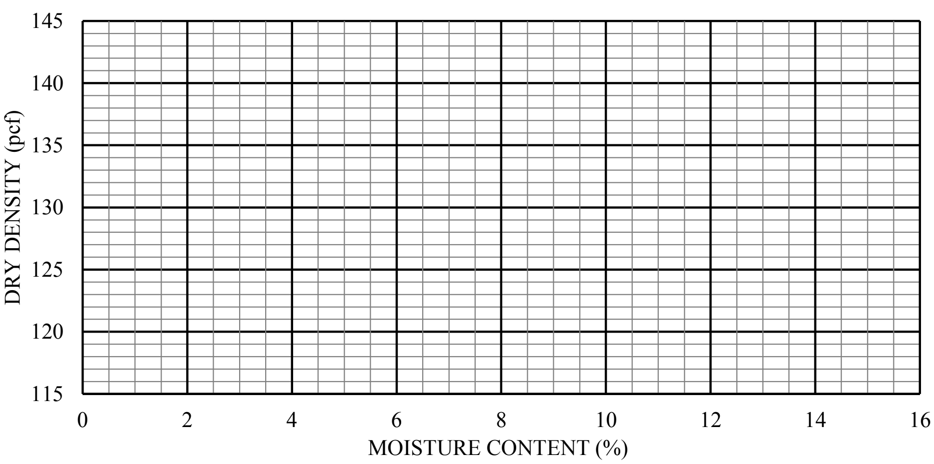


COBBLES	GRAVEL		SAND			SILT or CLAY
	Coarse	Fine	Coarse	Medium	Fine	

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"		
12.70	1/2"		
9.50	3/8"		
4.75	#4	**100**	
2.00	#10	**100**	
0.85	#20	**98**	
0.43	#40	**74**	
0.25	#60	**32**	
0.15	#100	**12**	
0.075	#200	**4.2**	

MOISTURE-DENSITY RELATIONSHIP ASTM D1557



HYDROMETER RESULT

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

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

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APPENDIX D

SEAOC SEISMIC SITE CLASSIFICATION DATA REPORT

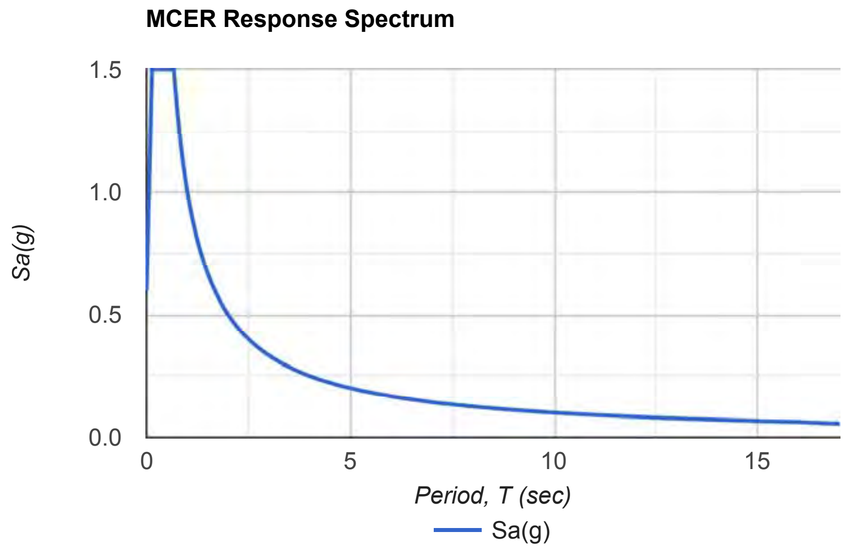
Pt. Mackenzie Substation

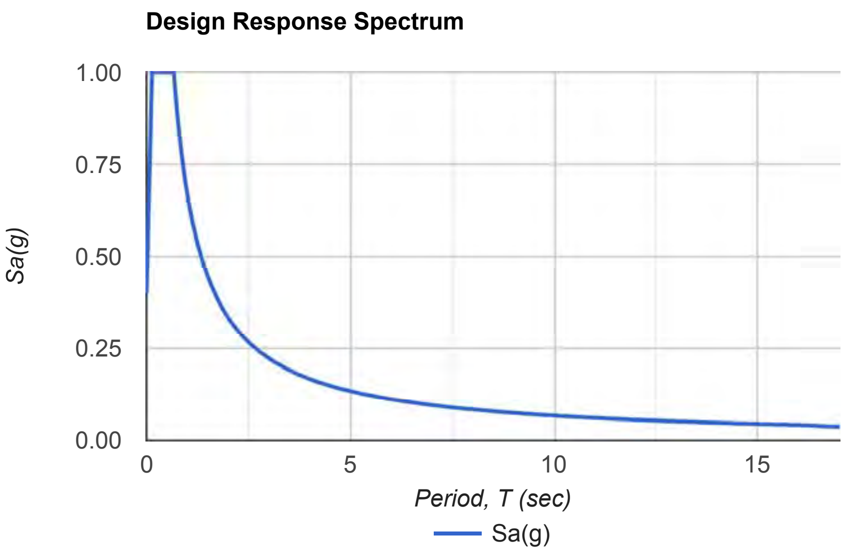
Latitude, Longitude: 61.24950372, -150.03310123

Date	3/29/2019, 2:48:33 PM
Design Code Reference Document	IBC-2015
Risk Category	IV
Site Class	D - Stiff Soil

Type	Value	Description
S_S	1.5	MCE_R ground motion. (for 0.2 second period)
S_1	0.669	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.5	Site-modified spectral acceleration value
S_{M1}	1.004	Site-modified spectral acceleration value
S_{DS}	1	Numeric seismic design value at 0.2 second SA
S_{D1}	0.669	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	1.906	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	1.702	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	1.5	Factored deterministic acceleration value. (0.2 second)
S1RT	0.827	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.796	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.669	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





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