November 20, 2014 1405906

Paul Weisner, PE
CE2 Engineers, Inc.
8221 Diamond Hook Drive
Anchorage, AK 99507

RE: GEOTECHNICAL RECOMMENDATIONS FOR PROPOSED PUMPHOUSES, CHEFORNAK, ALASKA

Dear Paul:

Golder Associates Inc. (Golder) is pleased to present this letter report to CE2 Engineers, Inc. (CE2) summarizing our geotechnical exploration and engineering recommendations for the two proposed pumphouses and associated boardwalks in Chefornak, AK (Figure 1).

Our services were performed in general accordance with our proposal to CE2 dated July 23, 2013. Our scope of work consisted of performing a limited, shallow geotechnical probe effort to characterize the near surface soil and thermal conditions. We were requested to develop our geotechnical recommendations for the proposed structures based on the shallow probe findings, our review of existing geotechnical data in the village, and our geotechnical interpretation of site subsurface conditions in the proposed development areas. Based on our discussions, we understand project funding would not accommodate a site-specific exploration program using geotechnical drilling subcontractors and equipment.

1.0 PROJECT UNDERSTANDING

Two new pumphouses are planned as part of water system improvements in Chefornak. The structures will be of a similar design, with dimensions of approximately 24 by 28 feet. We understand that one pumphouse building (Pumphouse #1) will replace an existing pumphouse near the northeast end of the community. The second pumphouse building (Pumphouse #2) will be located on an undeveloped site near the southeast end of the community. The approximate locations for both structures are noted in Figure 1. We understand the existing pumphouse building and pile foundation will be removed in preparation for the new facility. An elevated pipe rack will connect Pumphouse #2 with two existing water wells located south of the proposed site.

2.0 HISTORICAL REVIEW / GENERALIZED SUBSURFACE CONDITIONS

Our scope and geotechnical engineering evaluation and recommendations were based, in part, on our review and interpretation of subsurface conditions based on historic geotechnical data from our in-house files. We have reviewed geotechnical data for the following Chefornak projects:

- Village Store Foundation Rehabilitation, 1995
- Water Treatment Facility Geotechnical Services, 1998
- Thaw Probes for Subdivision, 1999
- Chefornak Power Plant, 2003
- UUI Communication Tower Geotechnical Services, 2005/2006
- Chefornak K-12 School Addition and Fuel Storage, 2009

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Based on a review of the above reports and our geotechnical experience in Chefornak, it is our opinion the generalized subsurface conditions in the community consist of an organic mat overlying fine-grained organic and inorganic silt. The organic mat can be disturbed or absent in developed areas. Discontinuous volcanic flow consisting of vesicular basalt has been encountered below the silt at variable depths throughout the village, but generally between 20 and 30 feet below ground surface (bgs). However, basalt cobbles and boulders have been encountered closer to ground surface in areas throughout the community. The basalt contact with the overlying silty soil has been interpreted as isolated layers or zones (float), frost fractured or degraded material, and as relatively continuous rock of undetermined thickness and extent. Water well logs in the village indicate the basalt layers can be discontinuous and interlayered with silt and other materials. Pile foundation construction as-built records indicate isolated or discontinuous layer or rock fragments can be encountered at relatively shallow depths. Degrading permafrost is present in the community, particularly near larger surface water bodies, in damaged tundra areas, and under areas with larger snow drifts.

3.0 SUBSURFACE EXPLORATION

The field exploration was conducted between 20 and 22, 2014 by Golder engineer Peter Calvin, PE. While in the field we coordinated with Paul Weisner, PE of CE2. The proposed pumphouse sites were provided by CE2. As noted previously, a site-specific exploration program using geotechnical drilling subcontractors and equipment was not authorized due to project funding constraints. CE2 requested that Golder conduct a shallow subsurface exploration program that primarily focused on the following tasks:

- Advancing hand probes within the proposed pumphouse sites and proposed boardwalk route to determine the general depth of thaw.
- Determine shallow subsurface geotechnical conditions by advancing shallow boreholes with a hand operated power drill.

The hand probes consisted of advancing a steel rod, approximately ½-inch in diameter, into the ground surface, typically until refusal. The hand probes were advanced in late summer, when probe depths generally indicate the near-maximum depth of seasonal thaw. Probe resistance was inferred to represent the approximate depth of seasonal thaw for this season. Hand probe refusal may have occurred in relict seasonally frozen ground or possibly the top of the underling permafrost generally encountered in the Chefornak area. Without a more detailed geotechnical drilling effort, it is not possible to distinguish between relict seasonal frost and permafrost. Additionally, the actual depth of thaw will vary between years and the time of the season. The approximate hand probe locations are presented in Figures 2 and 3.

The shallow subsurface soil conditions were investigated using a Hilti TE 70-ATC Combi-hammer drill with a 1.5-inch diameter bit. The drill bit was advanced to the safe working limits of the equipment depending on the subsurface conditions, generally between four to six feet below ground surface. As each exploration was advanced, a Golder representative visually logged the surface and subsurface soil and thermal conditions.

Representative soil samples were collected during the exploration. The recovered soil samples were visually classified in the field according to the Unified Soil Classification System (USCS), which is summarized in Figure A-1 of Appendix A. Recovered soil samples were retained in double sealed polyethylene bags to preserve their natural moisture content. Frozen soil characteristics, where applicable, were described according to Golder's Frozen Soil Classification standards as summarized in Figure A-2. Select soil samples were tested for soil index properties following ASTM recommended laboratory testing procedures and included soil moisture content, particle size distribution, pore water salinity, organic content, and Atterberg. A summary of these tests can be found in Appendix B.

This information in conjunction with our in-house geotechnical data and experience served as the bases for our geotechnical assessment and engineering recommendations.

4.0 SITE AND SUBSURFACE CONDTIONS

The proposed Pumphouse #1 site as shown in Figure 2 is located on the northeast end of town at the site of an existing pumphouse. The existing pumphouse is built on pile foundation that based on a visual inspection appears to be performing as intended. The site is bordered on the North, East, and West sides by an elevated boardwalk. Based on conversations with local residents, snow drifts accumulate along the base of the boardwalks during the winter. The effects of the snow drifting were evident in the ponded water at the base of the boardwalk and the damaged guardrails and support members. Additionally the backwash from the pumphouse is currently discharged though an approximately 5-foot long hose extending from the east side of the building. This discharge likely contributes to the standing water surrounding the building.

The subsurface soil stratigraphy was generally consistent across the site consisting of vegetative mat overlying approximately 1.5 to 3 feet of organic rich silt. The underlying the organic rich silt was gray mineral silt to the depth of exploration at approximately 6 feet. The thermal conditions at Pumphouse #1 were found to vary widely with frozen soils, generally interpreted as the top of the permafrost, ranging from 2 feet to over 8 feet below the ground surface, the limits of the exploration. Additionally pockets of relic seasonal frost were found between 5 and 6 feet below ground surface. In general, the depth to permafrost increased away from the pumphouse toward the boardwalk. The variable thermal conditions are likely influenced by the dumping of the backwash water from the pumphouse, snow drifting from the adjacent boardwalks, and the proximity to the Kinia River.

The proposed Pumphouse #2 site shown in Figure 3 is located on the southeast end of town on an undeveloped lot. The subsurface thermal conditions at Pumphouse #2 were found to be more consistent, likely due to the lack of development in the area. The subsurface soil stratigraphy generally consisted of a vegetative mat overlying a peat with organic silt to a depth of 1.5 feet bgs. Underlying the organic rich material, mineral silt with pockets of ice rich peat and organic silt was found to the depth of exploration, approximately 6 feet bgs. Based on hand probes the depth of thaw was generally found to be between 2 and 3 feet below ground surface. Areas of deeper thaw, up to 6 feet in depth were found adjacent to the boardwalk on the north side of the site.

Investigation of the potential boardwalk route leading from Pumphouse #2 to the existing water wells shows the majority of the route to have relatively consistent conditions. The subsurface soil stratigraphy generally consisted of a vegetative mat overlying a peat with organic silt to a depth of 1.5 feet bgs. Based on hand probes the depth of thaw was generally found to be between 2 and 3 feet below ground surface. One section of the boardwalk alignment that differed was an approximately 150-foot wide drainage located south of the existing residential houses. The depth of thaw within this area was found to be greater than 8 feet, the limits of our exploration. This increased thaw is likely the result of the ponded water in the area.

Based on our geotechnical experience in Chefornak and a review of historical geotechnical data, our findings are considered generally similar to shallow subsurface conditions encountered in other geotechnical explorations in the village.

5.0 DISCUSSION

The proposed developments are located in areas with organic silt and silt. Basalt rock is expected below the silt. Permafrost is present throughout the Chefornak area but in areas it may be degraded to considerable depth. The subsurface soil and thermal states are undetermined at the proposed development sites.

Based on discussions with CE2, we understand construction equipment mobilization costs are a budgetary concern. Owning to the uncertainty related to subsurface soil and ground temperatures, an adfreeze pile foundation system has long-term performance risk. To reduce foundation performance risk, we recommend seating the foundations on basalt rock to an end bearing condition. It is important to note that isolated basalt cobbles, boulders, and relatively thin basalt flows have been encountered in the

village. These subsurface conditions are generally not considered suitable for pile end bearing Thus foundation construction methods, such as down the hole hammer, air drilling and other methods, are recommended to verify the suitability of the encountered rock for pile end bearing conditions.

Embedment depth to firm basalt rock is uncertain, thus we recommend installing tensioned grouted anchors in each pile to resist seasonal frost jacking on the piles. Depending on the foundation structural loads, smaller dimensioned open-end pipe piles may be suitable for the planned development. These piles may be installed with lower mobilization construction equipment by driving, but predrilling may be required to achieve end bearing on basalt rock. Tension grouted anchors may not be necessary if the piles are embedded sufficiently deep through material suitable for developing adequate resistance to seasonal frost heave forces. However, owning to the uncertainly of subsurface soil and thermal states, tension grouted anchors for each pile are advised.

6.0 RECOMMENDATIONS

6.1 Pumphouse Foundations

Based on the observed and inferred geologic and thermal conditions at the proposed pumphouse sites the use of a conventional shallow or mat foundation system could result in unacceptable differential settlement. Therefore, the recommended option for the proposed pumphouse foundations is drilled micro-piles seated on basalt with anchors advanced inside the micro-piles and grouted into the underlying basalt rock. Grouted micro-pile foundation systems have been successfully installed in areas of Alaska where challenging soil conditions overly relatively shallow bedrock. Site improvement such as buried utilities and facility access that will be constructed at shallower depths should consider the potential for differential settlement.

While the pile axial loads have not been finalized, CE2 has indicated they are to be in the range of 7-kips to 14-kips (per pile) for geotechnical design purposes. For frost uplift, we have used design stress of 40-pounds per square inch (psi) along the pile perimeter though the expected active layer. At these design loads, the frost uplift will exceed the axial compressive design loads.

The following recommendations include Golder's interpretation of the site geology and ground thermal conditions based on the geotechnical exploration data, prior experience in Chefornak and engineering judgment of permafrost conditions at this site, with consideration of preliminary structure geometry and proposed facility location on the site for the analysis.

6.1.1 Grouted Micro-Pile Foundation Design

The permafrost at Pumphouse #1 is considered "warm" and possibly degraded. Continued warming and thawing of the permafrost will result in unacceptable differential settlement if end bearing conditions are not developed or extensive passive subgrade cooling systems are not used. To accommodate the site conditions the building should be supported on mirco-piles installed to an end bearing condition into the underlying bedrock. The micro-pile founded on bedrock will transfer the structural axial loads to the bedrock. End bearing on suitable rock will also accommodate additional down-drag forces on the piles as the surrounding permafrost warm or thaws. Anchoring the micro-pile with a rock anchor grouted into the underlying bedrock will resist the anticipated seasonal frost uplift forces. This type of foundation system has distinct advantages over other deep foundations in that it can be installed in areas with variable thermal and groundwater conditions and in areas where float rock and boulders may be encountered.

The recommended micro-pile foundation would consist of a minimum 6-inch diameter pipe pile seated on firm, competent bead rock. Schedule 40 or greater pipe piles are recommended with pile wall thickness determined by the structural engineer. Piles may be installed driven with or without predrilling vertically to end bearing in bedrock at the site. If seated on competent bedrock, axial capacities approaching the structural limits of the steel may be possible. It is assumed that the axial capacities and structural limits of the steel piles will be in excess of the actual design loads with a factor of safety of at least three. Golder

should review the design teams structural capacity needs for the foundation member for confirmation of bedrock support prior to issuing final design documents.

Drive shoes can be used with the piles, however the drive shoes must be of suitable dimension to allow for installed grouted anchors through pile annular space.

To resist the anticipated frost uplift forces the recommended foundation system would include all-thread rods grouted into the basalt rock through the tip of the seated micro-piles. The grouted anchor rods should be 0.75-inch diameter or as recommended by the structural engineer. The design is based on the grouted rods developing at least 80 pounds per square inch (psi) allowable strength along the grout / rock interface. The grouted anchor rods should be installed and grouted at least six (6) feet into competent basalt rock.

6.1.2 Lateral Loading of Grouted Micro-Piles

For preliminary design considerations, Ensoft's software LPile version 2012.6.37 was used to estimate lateral capacity for a range of pile head deflections. The lateral capacity of the piles should be further analyzed as the project develops with coordination from the structural and civil engineers. The pile capacity response to applied lateral loads will be affected by pile stickup height from the ground surface. Structural framing and bracing will further define the pile head conditions as free head, fixed head, or partially fixed head.

Table 1 shows the estimated deflection with applied lateral load for a vertically installed 6-inch diameter, schedule 40 pipe pile with 3-foot stickup through unfrozen silt. For example, a free head pile installed at the site with pile stickup height of three feet would develop a 1-inch deflection at the pile cap with an estimated applied lateral load of 1.4 kips. These preliminary calculations are presented for planning that will require confirmation as the lateral loads and pile geometry are determined. Pile stresses under these lateral loads should be reviewed by the structural engineer.

Table 1: Estimated Lateral Deflection with Applied Lateral Load: 6-inch diameter Grouted Pipe Pile

Pile Stickup	Pile Head Condition	0.25 inch Deflection	0.5 inch Deflection	1 inch Deflection
3 feet	Free Head	0.5 kips	0.85 kips	1.4 kips

6.1.3 Bedrock Contingency Plan

Due to the lack of a site specific geotechnical drilling program the actual depth to competent bedrock is undetermined. Based on previous geotechnical investigations in the area, the depth of competent bedrock is estimated to be between 20 and 30 feet bgs although a large amount of variability can be expected in the area. In the event of the absence of competent bedrock at a depth of less than 50 feet bgs the 6-inch pile can be employed to support the expected loads if additional measures are taken.

Due to the "warm" permafrost at the site and potential for continued thawing the drilled/driven 6-inch pile option will require passive cooling to develop and maintain adequate capacity for the expected axial loads. Driven pile depths of 50 feet, with passive cooling systems installed on the order of 15 to 20 feet into the underlying permafrost will necessary. Without the use of passive cooling to maintain sufficient ad-freeze bond strength the 6-inch pile depth will required pile embedment depths greater than 50 feet with final embedment based on site-specific site conditions.

If this contingency plan is employed we should be notified so that we may review our conclusions and recommendations and provide a detailed thermal analysis detailing the passive cooling geometry and location.

6.1.4 Seismic Design Criteria

Based on site conditions observed, the proposed building locations should meet seismic site class "D" criteria as defined in the International Building Code Seismic site class "D" is defined as, "Stiff soil with an average Standard Penetration Test (SPT) "N" value between 15 and 50 in the upper 100 feet". It is important to note that relative densities based on SPT "N" values were not determined in this area under our scope of services. However, soils with SPT "N" values between 15 and 50 can be reasonably expected at these sites, particularly with depth. It is also reasonable to expect permafrost or basalt rock to be encountered within 100 feet below of the surface in areas around Chefornak.

The criteria are based on mapped spectral response acceleration for short periods (S_s) of 0.2g and mapped spectral response accelerations for a 1 second period (S_1) of 0.13g.

Site coefficient factors F_a and F_v of 1.6 and 2.2, respectively, are considered appropriate to determine seismic characteristics for Site Class "D". Based on these values, the design spectral response acceleration for short period and 1-second period for Site Class "D" can be determined using the following equations:

$SD_s = 2/3\ F_a S_s$ and $SD_1 = 2/3\ F_v S_1$

$SD_s = 0.21g$ and $SD_1 = 0.19g$

Liquefaction of saturated finer-grained soil, in particular fine to medium grained sands, may occur during seismic events. However, based on our site findings and our general knowledge of the area geology, the risk of liquefaction is considered low unless saturated fine grained sand soils are present. If the project warrants a more refined liquefaction analysis, site-specific geotechnical test borings will be necessary as part of the liquefaction assessment.

6.1.5 Grouted Micro-Pile Installation

While bedrock was not observed during this investigation previous explorations have noted discontinuous volcanic flow consisting of vesicular basalt below the silt at variable depths, but generally between 20 and 30 feet below ground surface. However, 40 foot depths, possibly deeper, to suitable end bearing basalt is possible in the area. Due to the lack of a site specific geotechnical drilling program the soil bedrock contact elevation should be expected to vary by up to 10 feet for planning purposes. Additional micro-pile length will be needed if greater variability in depth to bedrock is present. We do not advise reliance on skin friction or adfreeze bond shear strength for long-term pile axial capacity unless a site specific deeper geotechnical site investigation is conducted.

We have assumed the micro-pile can be advanced to a firm, non-yielding state on competent bedrock though the overlying silt and potential float rock with and air-track or similar drill. The annulus would then be cleaned of soil and rock fragments to the bedrock seating surface. After the pile interior annulus is properly cleaned we recommend minimum 0.75-inch diameter 75-ksi all-thread anchor rods be installed below the pile tip. The anchor rods would be installed through a nominal 3-inch diameter, or larger, bore hole advanced inside the pile through the underlying rock.

The anchor will require tremmie-placed grout from the bottom of the anchor upwards with a fast setting Portland cement or Fondue cement grout. If desired by the contractor, fine grained silica sand may be added to the grout mixture at no more than 1 sand : 3 gout by mass ratio. We recommend continuous grouting through the rock section without pre-tensioning. We have assumed that the anchor rods set at least 6 feet into competent hard basalt would be installed through each pile. The basalt through the grouted anchor section may be fractured or have significant voids thus grout loss may occur.

Grout sequencing of the anchors and piles warrants consideration. The anchors should be grouted their full rock embedment lengths with the grout extending no more than one (1) foot above the pile tip <u>if the</u>

tension testing uses the pile as the reaction base. If the tension testing reaction base is independent of the pile, a thicker grout section inside the pile annular space is permitted.

After the rock section grout has cured, the anchor rods should be tensioned tested. We have assumed tensions will be conducted against the top plate. We recommend tensioning each rock section grouted anchor to 50-kips then backing down the lock off load of 15–kips. The Post Tensioning Institute's recommended practices for proof testing grouted anchors are recommended. Deflection and load measurements should be collected during anchor tensioning as part of the construction installation records.

We have assumed winter construction (frozen ground) for installing the grouted micro-pile foundation. We have based our recommendation on winter construction due to the challenging site conditions and to reduce adverse impacts on the existing tundra surface. If summer construction is under consideration, we recommend using temporary construction mats or an insulated permanent fill pad to protect the tundra surface. We should be notified as soon as possible if construction during thawed surface soil conditions is under consideration.

6.2 Boardwalk Foundations

Based on conversations with CE2 we understand that the preferred foundation for the boardwalk connecting Pumphouse #2 to the existing wells will be helical anchors. Helical anchors need to be sufficiently embedded to resist axial compression, frost heave forces, and lateral forces. Axial compression loads consist of the boardwalk's self-weight and traffic loads. The design axial compression helical anchor loads, used for analysis in this report, are 2.7 kips allowable load per helical anchor. The frost heave force is assumed to be 40 pounds per square inch (psi) uplift force acting over the riser's embedment within the active layer. For a minimum recommended riser diameter of 3.5 inches and an active layer of 4 feet (conservative estimate for the project area), the design frost force is 21 kips. We have assumed a 1,000 pound lateral load applied three feet above the ground surface for the purposes of this analysis.

6.2.1 Helical Anchor Foundation

The helical anchors should consist of a 3.5-inch diameter schedule 80 riser shaft with twin 10-inch diameter helices. The lower helix should be spaced approximately 1-foot from the tip of the riser shaft (manufacturer supplied with an angle cut at the tip). The upper helix should be spaced approximately three helix diameters (30 inches) from the lower helix. The helix pitch and tip cut angle should be as recommended by the manufacturer, but we advise a 3-inch helix pitch. The maximum installation torque should be 13,000 foot-pounds or as recommended by the manufacturer.

Due to the varied site conditions, two installation procedures are required based on the varying thermal conditions within the project area. Embedment criteria is based on the embedment to the uppermost helix, the depth to permafrost (where present), and installation torque.

In permafrost areas, the minimum embedment, to the uppermost helix, should be installed to at least 8 feet below the ground surface or at least 3 feet below the surface of the permafrost, whichever is deeper. For example, if during installation the torque increases at approximately 6 feet below ground surface, indicating permafrost contact, the uppermost helix should be embedded to a depth of 9 feet. Permafrost contact at helical anchor locations should be confirmed during installation by the contractor with adjacent thaw depth probes.

In areas of degraded permafrost (deep surface thaw), the minimum embedment to the uppermost helix should be 15 feet below ground surface, or the average installation torque should exceed the minimum installation torque over the final 3 feet of helical pile installation, whichever is deeper. The minimum installation torque recommendation is 1,500 foot-pounds. For example, if a helical anchor is installed to a depth of 15 feet with an averaged installation torque of 1,200 foot-pounds, installation should continue until the minimum torque requirement is met.

6.2.2 Lateral Loading on Anchors

The surface organic soils will provide virtually no lateral support for the helical anchors. The mineral silt will provide some lateral resistance. For the purposes of this analysis, a 1,000 pound lateral load was applied 3 feet above the ground surface with 3 feet of unfrozen peat overlying unfrozen mineral silt to the full depth of the anchor, a conservative case based on existing subsurface data from the Chefornak area. Under these conditions, the helical anchor will deflect approximately 1-inch at the ground surface. If the 1,000 pound load is applied at 6 feet above the ground surface, the deflection at ground surface will increase to approximately 1.5 inches. The project structural engineer should determine areas of the boardwalk that may require lateral bracing to minimize the lateral deflection where needed.

The above analyses assume that the soil surrounding the helical is flush to the riser surface. During helical anchor installation, the helical must be adjusted to maintain vertical installation. The adjustments made during installation can disturb the soil adjacent to the riser shaft creating a gap. Over time the soil will compress and the gap will close; however, the initial lateral anchor deflection will be increased by the gap surrounding the anchor riser shaft.

We recommend the space between the ground surface and the helical anchor connections or boardwalk timber members be 6 inches to allow frost associated movements in the active layer. It is also recommended that maximum laterally unsupported riser height above the ground surface be no greater than 3 feet, due to potential lateral loading concerns. Anchors in areas with expected lateral loading or with greater than 3 feet between the ground surface and the boardwalk may need lateral bracing or larger dimensioned helical anchor riser sections for lateral stability, as determined by the project structural engineer.

6.2.3 Helical Anchor Installation

Helical anchors should be installed in a continuous manner and advanced at a rate equaling the pitch of the helix for each rotation. The anchor should be “screwed” into the subsurface soil without “auguring.” The twin helices should be spaced such that the leading edge of one helix meets with the trailing edge of the previous helix.

A pressure gauge should be attached in line with the hydraulic system to the torque drive head installing the anchors. The pressure measurement should be recorded during installation and converted to torque using the pressure torque calibration for the drive head. The drive head should be calibrated prior to construction activities to determine the pressure torque ratio/curve.

We recommend anchor installation in the late fall or early winter, when the soil temperature will be warmer and therefore less torque may be required for installation. If the anchors are installed during the winter, the amount of torque, required to install the anchors through the frozen surficial soils or permafrost, may be increased and possibly exceed the maximum allowable installation torque recommended by the manufacturer.

At no time should the torque exceed the maximum torque recommended by the manufacturer and in extreme cold, the allowable torque might need to be down-rated. If the maximum installation torque is met during installation, a pilot hole drilled to accommodate the shaft through the frozen surface or permafrost soils allows for installation in areas of hard frozen conditions. Predrilled pilot holes greater than the shaft diameter, to allow penetration through the surface frost, may be required to install the helical anchors without exceeding the maximum allowable torque during the later winter months. Pre-drilling to accommodate installation through the surface frost should be limited to a maximum depth of 3 feet. If predrilling is required, we should be notified to confirm that the contractor’s installation methods are consistent with the intent of our recommendations.

If the required torque is not achieved at the initial embedment depth, shaft riser extensions should be added and the helical anchor installed to a deeper depth until the required minimum torque criteria is reached. If the upper most helix is installed to a depth of 20 feet and the required torque has not been

achieved, the anchor should be load tested to 1.5 times the required load with the limited permanent (inelastic) vertical movement of 0.25 inches. The testing should be monitored by a qualified engineer. If this case occurs during construction, we should be notified to review our design recommendations and provide further guidance. It is likely that another helical anchor may be installed adjacent to the existing anchor at a minimum distance of 3 helix diameters (largest helix).

To prevent large frost heave forces from developing along the couplers (connecting joints), couplers should not be located in the uppermost 5 feet below, and the 6 inches above ground surface.

The plans and specifications should be reviewed by Golder to verify that they are in accordance with the intent of our recommendations. The helical anchors should be installed in accordance with the manufacturer's recommendations. A qualified inspector, or contractor representative, should be on site during installation to record installation torque per foot of embedment, embedment depth, permafrost depth, date and duration of installation. Golder should be retained during construction to review as-built records to ensure the helical anchors are installed to the intent of our recommendations.

7.0 USE OF THIS REPORT

This report has been prepared exclusively for CE2 for the proposed pumphouse and boardwalk project in Chefornak, Alaska. If there are significant changes in the nature, design, or location of the proposed facility, we should be notified so that we may review our data findings in light of the proposed changes and provide a written modification or verification of the changes.

There are possible variations in subsurface conditions between explorations and also with time. Therefore, observation and testing by a qualified geotechnical engineer should be included during design and construction to provide corrective recommendations adapted to the conditions revealed during the work. In addition, a contingency for unanticipated conditions should be included in the construction budget and schedule. The work program followed the standard of care expected of geotechnical professionals undertaking similar work in the State of Alaska under similar conditions. No warranty expressed or implied is made.

It has been a pleasure to assist you with this project. Please contact us if you have any questions or require additional information.

GOLDER ASSOCIATES INC.

Peter Calvin, PE
Project Geotechnical Engineer

Richard A. Mitchells, PE
Associate and Manager Alaska Operations

Attachments:	Figure 1	Vicinity Map
	Figure 2	Thaw Probe and Test Hole Location Map, Proposed Pumphouse #1
	Figure 3	Thaw Probe and Test Hole Location Map, Proposed Pumphouse #2
	Appendix A	Soil Classification Information
	Appendix B	Laboratory Data

PAC/RAM/mlp

FIGURES

REFERENCE

AERIAL IMAGERY PROVIDED BY CE2 ENGINEERS, INC. ON SEPTEMBER 2, 2014.

CLIENT CE2 ENGINEERS, INC.		PROJECT CHEFORNAK PUMPHOUSE CHEFORNAK, ALASKA	
CONSULTANT	YYYY-MM-DD	2014-11-20	TITLE **VICINITY MAP**
	PREPARED	APG	
	DESIGN	-	
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Golder Associates

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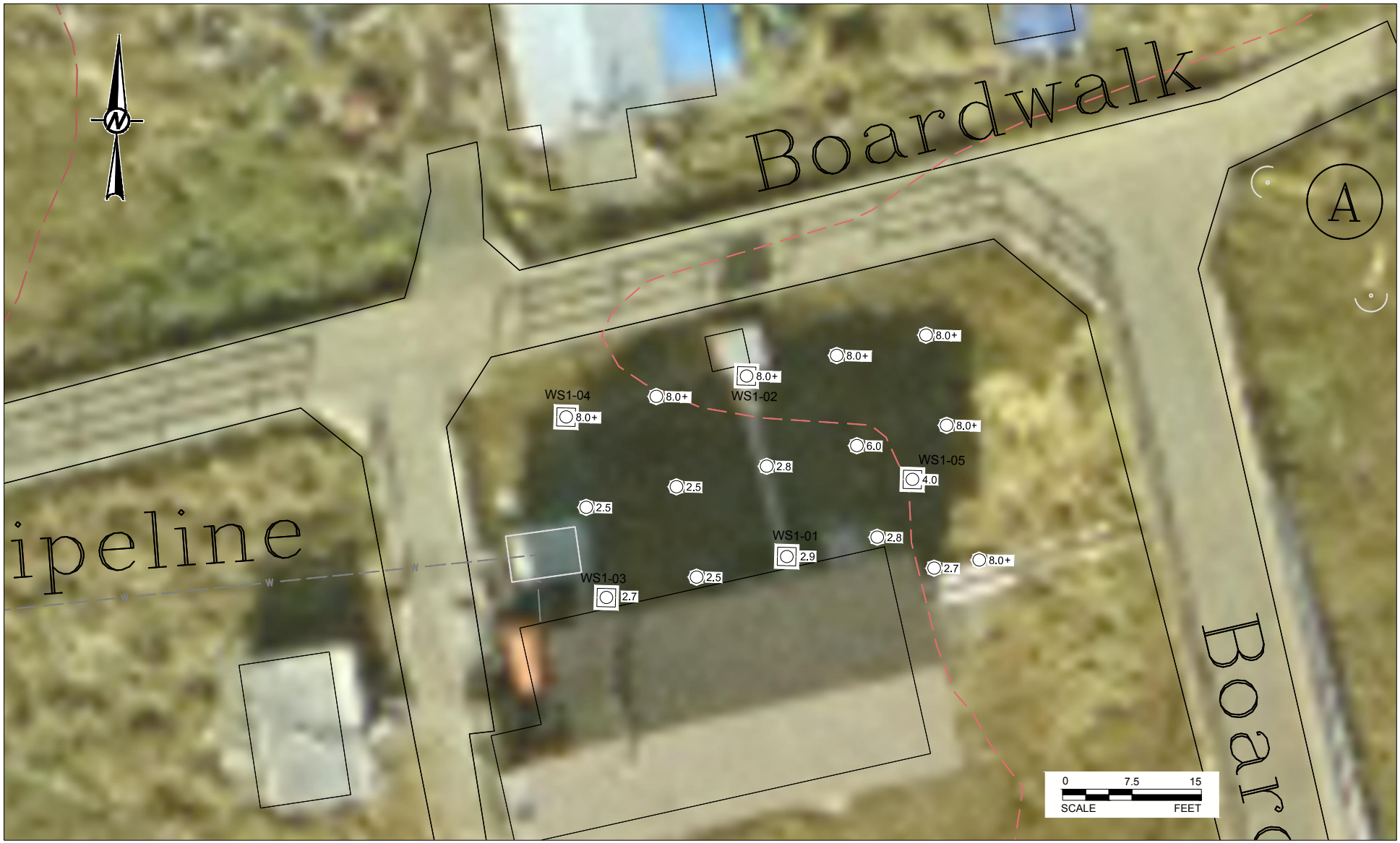

LEGEND

2.7 THAW PROBE LOCATION AND DEPTH IN FEET

WS1-01 TEST HOLE LOCATION

REFERENCE

AERIAL IMAGERY AND BASE MAP PROVIDED BY CE2 ENGINEERS, INC. ON SEPTEMBER 2, 2014.

CLIENT
CE2 ENGINEERS, INC.

CONSULTANT

Golder Associates

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PROJECT
CHEFORNAK PUMPHOUSE

CHEFORNAK, ALASKA

TITLE
THAW PROBE AND TEST HOLE LOCATION MAP - PROPOSED PUMPHOUSE #1

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2.7 ○ THAW PROBE LOCATION AND DEPTH IN FEET

WS1-12 □ TEST HOLE LOCATION

REFERENCE

AERIAL IMAGERY AND BASE MAP PROVIDED BY CE2 ENGINEERS, INC. ON SEPTEMBER 2, 2014.

CLIENT
CE2 ENGINEERS, INC.

CONSULTANT

Golder Associates

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CHEFORNAK, ALASKA

TITLE
THAW PROBE AND TEST HOLE LOCATION MAP - PROPOSED PUMPHOUSE #2

PROJECT No.	CONTROL	Rev.	FIGURE
1405906		----	3

IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ANSI A
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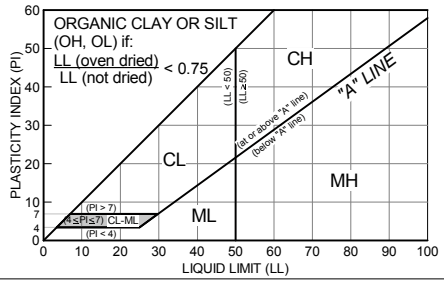
APPENDIX A
SOIL CLASSIFICATION INFORMATION

UNIFIED SOIL CLASSIFICATION (adapted from ASTM D2487)

MATERIAL TYPES	CRITERIA FOR ASSIGNING SOIL GROUP NAMES AND GROUP SYMBOLS USING LABORATORY TESTS			GROUP SYMBOL	SOIL GROUP NAMES & LEGEND	
COARSE-GRAINED SOILS >50% RETAINED ON NO. 200 SIEVE	GRAVELS >50% OF COARSE FRACTION RETAINED ON NO 4. SIEVE	CLEAN GRAVELS <5% FINES	$C_U \geq 4$ AND $1 \leq C_C \leq 3$	GW	WELL-GRADED GRAVEL	If soil contains ≥15% sand, add "with sand"
			$C_U < 4$ AND/OR [$C_C < 1$ OR $C_C > 3$]	GP	POORLY GRADED GRAVEL	
		GRAVELS WITH FINES >12% FINES	FINES CLASSIFY AS ML OR MH	GM	SILTY GRAVEL	
			FINES CLASSIFY AS CL OR CH	GC	CLAYEY GRAVEL	
	SANDS ≥50% OF COARSE FRACTION PASSES ON NO 4. SIEVE	CLEAN SANDS <5% FINES	$C_U \geq 6$ AND $1 \leq C_C \leq 3$	SW	WELL-GRADED SAND	If soil contains ≥15% gravel, add "with gravel"
			$C_U < 6$ AND/OR [$C_C < 1$ OR $C_C > 3$]	SP	POORLY GRADED SAND	
		SANDS AND FINES >12% FINES	FINES CLASSIFY AS ML OR MH	SM	SILTY SAND	
			FINES CLASSIFY AS CL OR CH	SC	CLAYEY SAND	
FINE-GRAINED SOILS >50% PASSES NO. 200 SIEVE	SILTS AND CLAYS LIQUID LIMIT <50	PLASTICITY CHART		CL	LEAN CLAY	If soil contains coarse-grained soil from 15% to 29%, add "with sand" or "with gravel" for whichever type is prominent, or for ≥30%, add "sandy" or "gravelly"
				ML	SILT	
				OL	ORGANIC CLAY OR SILT	
	SILTS AND CLAYS LIQUID LIMIT ≥50			CH	FAT CLAY	
				MH	ELASTIC SILT	
				OH	ORGANIC CLAY OR SILT	
HIGHLY ORGANIC SOILS		PRIMARILY ORGANIC MATTER, DARK IN COLOR, AND ORGANIC ODOR		PT	PEAT	



NOTES: $C_U = \frac{D_{60}}{D_{10}}$ $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$

Gravels or sands with 5% to 12% fines require dual symbols (GW-GM, GW-GC, GP-GM, GP-GC, SW-SM, SW-SC, SP-SM, SP-SC) and add "with clay" or "with silt" to group name. If fines classify as CL-ML for GM or SM, use dual symbol GC-GM or SC-SM. $D_{(X\%)}$ is soil particle diameter where X% is % finer. *Optional Abbeviations:* Lower case "s" after USCS group symbol denotes either "sandy" or "with sand" while "g" denotes either "gravelly" or "with gravel"

RELATIVE DENSITY / CONSISTENCY ESTIMATE USING STANDARD PENETRATION TEST (SPT) VALUES

(adapted from Terzaghi and Peck 1967)

COHESIONLESS SOILS [a]		COHESIVE SOILS[b]		
RELATIVE DENSITY	$(N_1)_{60}$ (blows/ft)[c]	CONSISTENCY	$(N_1)_{60}$ (blows/ft)[c]	UNCONFINED COMPRESSIVE STRENGTH (TSF)[d]
VERY LOOSE	0 - 4	VERY SOFT	0 - 2	0 - 0.25
LOOSE	4 - 10	SOFT	2 - 4	0.25 - 0.50
COMPACT	10 - 30	FIRM	4 - 8	0.50 - 1.0
DENSE	30 - 50	STIFF	8 - 15	1.0 - 2.0
VERY DENSE	OVER 50	VERY STIFF	15 - 30	2.0 - 4.0
		HARD	OVER 30	OVER 4.0

(a) Soils consisting of gravel, sand, and silt, either separately or in combination possessing no characteristics of plasticity, and exhibiting drained behavior.
(b) Soils possessing the characteristics of plasticity, and exhibiting undrained behavior.
(c) Refer to ASTM D1586 for a definition of N value. $(N_1)_{60}$ is the N value corrected for hammer energy and overburden pressure, and is detailed in ASTM D6066. N values may be affected by a number of factors including: material size, sampler size, hammer weight and type, depth, drilling method, and borehole disturbance. **N values are only an approximate guide for frozen soil or cohesive soil.**
(d) Undrained shear strength, s_u= 1/2 unconfined compression strength, U_c. Note that Torvane (TV) measures s_u and pocket penetrometer (PP) measures U_c

CRITERIA FOR DESCRIBING MOISTURE CONDITION

(adapted from ASTM D2488)

DRY	ABSENCE OF MOISTURE, DUSTY, DRY TO THE TOUCH
MOIST	DAMP BUT NO VISIBLE WATER
WET	VISIBLE FREE WATER, USUALLY SOIL IS BELOW WATER TABLE

COMPONENT DEFINITIONS BY GRADATION

COMPONENT	SIZE RANGE
BOULDERS	GREATER THAN 12 in.
COBBLES	12 in. to 3 in.
GRAVEL	3 in. to #4 Sieve (4.76 mm)
COARSE GRAVEL	3 in. to 3/4 in.
FINE GRAVEL	3/4 in. to #4 (4.76 mm)
SAND	#4 (4.76 mm) to #200 (0.074 mm)
COARSE SAND	#4 (4.76 mm) to #10 (2.0 mm)
MEDIUM SAND	#10 (2.0 mm) to #40 (0.42 mm)
FINE SAND	#40 (0.42 mm) to #200 (0.074 mm)
SILT & CLAY (FINES)	SMALLER THAN #200 (0.074 mm)

SAMPLER ABBREVIATIONS

SS	SPT Sampler (2 in. OD, 140 lb hammer)	**C**	Core (Diamond Bit)
HD	Heavy Duty Split Spoon (3 in. OD, 340 lb hammer)	**TW**	Thin Wall (Shelby Tube)
-BL	Brass Liners used in Split Spoon	**TP**	Thin Wall Piston Sampler
CA	Continous Core (Soil in Hollow-Stem Auger)	**MS**	Modified Shelby
GS	Grab Sample from Surface / Testpit	**MC**	Geoprobe Macro-Core
AC	Auger Charge	**RC**	Air Rotary Cuttings
AW	Auger Wash	**AG**	Auger Cuttings

DESCRIPTIVE TERMINOLOGY FOR PERCENTAGES (ASTM D2488)

DESCRIPTIVE TERMS	RANGE OF PROPORTION
TRACE	0 - 5%
FEW	5 - 10%
LITTLE	10 - 25%
SOME	30 - 45%
MOSTLY	50 - 100%

LABORATORY TEST AND NOTES ABBREVIATIONS / SYMBOLS

Con	Consolidation	**PID**	Photoionization Detector	**TXCD**	Triaxial, Consolidated Drained
Dd	Dry Density	**PM**	Modified Proctor (D1557)	**TXCU**	Triaxial, Consolidated Undrained
K	Thermal Conductivity	**PP**	Pocket Penetrometer (Field)	**TXUU**	Triaxial, Unconsolidated Undrained
MA	Sieve and Hydrometer	**PTLD**	Point Load	**W_C**	Liquid Limit (LL)
NP	Non-plastic	**SA**	Sieve Analysis	**W_P**	Plastic Limit (PL)
OLI	Organic Loss	**SpG**	Specific Gravity	**Ω**	Soil Resistivity (Res.)
P200	Passing #200 Sieve (D1140)	**TC**	Thaw Consolidation/Strain	▼	Water Level
pH	Soil pH	**TV**	Torvane (Field)	▽	Water Level While Drilling

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SOIL CLASSIFICATION / LEGEND

Figure A-1

FROZEN SOIL CLASSIFICATION (ASTM D4083)

1. DESCRIBE SOIL INDEPENDENT OF FROZEN STATE	CLASSIFY SOIL BY THE UNIFIED SOIL CLASSIFICATION SYSTEM				
2. MODIFY SOIL DESCRIPTION BY DESCRIPTION OF FROZEN SOIL	MAJOR GROUP		SUBGROUP		
	DESCRIPTION	DESIGNATION	DESCRIPTION		DESIGNATION
	Segregated ice not visible by eye	N	Poorly bonded of friable		Nf
			Well bonded	No excess ice	Nbn
				Excess ice	Nbe
	Segregated ice visible by eye (ice less than 25 mm thick)	V	Individual ice crystals or inclusions		Vx
			Ice coatings on particles		Vc
			Random or irregularly oriented ice formations		Vr
			Stratified or distinctly oriented ice formations		Vs
			Uniformly distributed ice		Vu
3. MODIFY SOIL DESCRIPTION BY DESCRIPTION OF SUBSTANTIAL ICE STRATA	Ice greater than 25 mm thick	ICE	Ice with soil inclusions		ICE+soil type
			Ice without soil inclusions		ICE

FROST DESIGN SOIL CLASSIFICATION (1)

FROST GROUP(2)	GENERAL SOIL TYPE	% FINER THAN 0.02 mm BY WEIGHT	TYPICAL USCS SOIL CLASS
NFS(3) [MOA NFS]	(a) Gravels Crushed stone Crushed rock	0 to 1.5	GW, GP
	(b) Sands	0 to 3	SW, SP
PFS(4) [MOA NFS]	(a) Gravels Crushed stone Crushed rock	1.5 to 3	GW, GP
[MOA F2]	(b) Sands	3 to 10	SW, SP
S1 [MOA F1]	Gravelly soils	3 to 6	GW, GP GW-GM, GP-GM, GW-GC, GP-GC
S2 [MOA F2]	Sandy soils	3 to 6	SW, SP SW-SM, SP-SM, SW-SC, SP-SC
F1 [MOA F1]	Gravelly soils	6 to 10	GM, GC, GM-GC, GW-GM, GP-GM, GW-GC, GP-GC
F2 [MOA F2]	(a) Gravelly soils	10 to 20	GW, GP GW-GM, GP-GM, GW-GC, GP-GC
	(b) Sands	6 to 15	SM, SW-SM, SP-SM, SC, SW-SC, SP-SC, SM-SC
F3 [MOA F3]	(a) Gravelly soils	Over 20	GM, GC, GM-GC
	(b) Sands, except very fine silty sands	Over 15	SM, SC, SM-SC
	(c) Clays, PI>12	--	CL, CH
F4 [MOA F4]	(a) Silts	--	ML, MH, ML-CL
	(b) Very fine silty sands	Over 15	SM, SC, SM-SC
	(c) Clays, PI<12	--	CL, ML-CL
	(d) Varved clays or other fine-grained banded sediments	--	CL or CH layered with ML, MH, ML-CL, SM, SC, or SM-SC

(1) From U.S. Army Corps of Engineers (USACE), EM 1110-3-138, "Pavement Criteria for Seasonal Frost Conditions," April 1984
(2) USACE frost groups directly correspond to frost groups listed in Municipality of Anchorage (MOA) design criteria manual (DCM), 2007; except as noted.
(3) Non-frost susceptible
(4) Possibly frost susceptible, requires lab test for void ratio to determine frost design soil classification. Gravel with void ratio > 0.25 would be NFS; Gravel with void ratio < 0.25 would be S1; Sands with void ratio > 0.30 would be NFS; Sands with void ratio < 0.30 would be S2 or F2

ICE BONDING SYMBOLS

No ice-bonded soil observed

Poorly bonded or friable

Well bonded

DEFINITIONS

Candled Ice is ice which has rotted or otherwise formed into long columnar crystals, very loosely bonded together.

Clear Ice is transparent and contains only a moderate number of air bubbles.

Cloudy Ice is translucent, but essentially sound and non-pervious

Friable denotes a condition in which material is easily broken up under light to moderate pressure.

Granular Ice is composed of coarse, more or less equidimensional, ice crystals weakly bonded together.

Ice Coatings on particles are discernible layers of ice found on or below the larger soil particles in a frozen soil mass. They are sometimes associated with hoarfrost crystals, which have grown into voids produced by the freezing action.

Ice Crystal is a very small individual ice particle visible in the face of a soil mass. Crystals may be present alone or in a combination with other ice formations.

Ice Inclusions are individual ice masses visible in the face of a soil mass. Inclusions may be present alone or in a combination with other ice formations.

Ice Lenses are lenticular ice formations in soil occurring essentially parallel to each other, generally normal to the direction of heat loss and commonly in repeated layers.

Ice Segregation is the growth of ice as distinct lenses, layers, veins and masses in soils, commonly but not always oriented normal to direction of heat loss.

Massive Ice is a large mass of ice, typically nearly pure and relatively homogeneous.

Poorly-bonded signifies that the soil particles are weakly held together by the ice and that the frozen soil consequently has poor resistance to chipping or breaking.

Porous Ice contains numerous voids, usually interconnected and usually resulting from melting at air bubbles or along crystal interfaces from presence of salt or other materials in the water, or from the freezing of saturated snow. Though porous, the mass retains its structural unity.

Thaw-Stable frozen soils do not, on thawing, show loss of strength below normal, long-time thawed values nor produce detrimental settlement.

Thaw-Unstable frozen soils show on thawing, significant loss of strength below normal, long-time thawed values and/or significant settlement, as a direct result of the melting of the excess ice in the soil.

Well-Bonded signifies that the soil particles are strongly held together by the ice and that the frozen soil possesses relatively high resistance to chipping or breaking.

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FROZEN SOIL CLASSIFICATION / LEGEND

Figure A-2

APPENDIX B
LABORATORY DATA

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TABLE B-1: SAMPLE SUMMARY

Client:	CE2 Engineers Inc.	**Project No.:**	1405906		
Project:	Chefornak Pumphouse Sites	**QA/QC By:**	J. Randazzo	**Date:**	9/9/2014
Location:	Chefornak, AK	**Reviewed By:**	H. Weston	**Date:**	9/15/2014

SAMPLING DATA						CLASSIFICATION AND INDEX TEST RESULTS											
SAMPLE LOCATION	SAMPLE NUMBER	DEPTH (ft)		RECOVERY (%)	SAMPLE TYPE	NATURAL MOISTURE CONTENT (%)	LIQUID LIMIT (LL) (%)	PLASTIC LIMIT (PL) (%)	PLASTICITY INDEX (PI) (%)	GRADATION (%)			ORGANIC CONTENT (%)	SALINITY (ppt) [(d) is directly meas.]	DESCRIPTION (USCS)	TESTS / OTHER TESTS	
		TOP	BOTTOM							GRAVEL	SAND	FINES (SILT & CLAY)					
WS1-01	1	1.0	1.5		GS	105							20.7				
WS1-01	2	3.0	3.5		GS	77											
WS1-01	3	5.0	5.5		GS	70											
WS1-03	1	3.5	4.0		GS	67											
WS1-04	1	1.8	2.0		GS	121				0.0	21.6	78.4		2			
WS1-06	1	5.5	6.0		GS	75											
WS1-07	1	1.0	1.5		GS	79											
WS2-01	1	1.5	1.8		GS	350								0			
WS2-01	2	4.0	5.5		GS												
WS2-01	3	6.0	7.0		GS	101											
WS2-02	1	1.0	1.3		GS												
WS2-02	2	3.0	3.5		GS												
WS2-03	1	1.5	2.0		GS	493											
WS2-03	2	5.5	6.0		GS	387											
WS2-04	1	2.3	3.0		GS												
WS2-04	2	5.0	6.0		GS												
WS2-05	1	1.0	1.5		GS	163											
WS2-05	2	3.5	4.0		GS												
WS2-05	3	5.5	6.0		GS	160											
WS2-06	1	2.0	3.0		GS												
WS2-06	2	5.0	6.0		GS												
WS2-07	1	0.5	1.0		GS	439											
WS2-07	2	2.5	3.0		GS	384											
WS2-08	1	2.0	2.2		GS												
WS2-08	2	4.0	4.5		GS												
WS2-08	3	5.0	6.0		GS												
WS2-09	1	2.0	3.0		GS												
WS2-10	1	1.0	1.5		GS	235											
WS2-10	2	3.0	4.0		GS												
WS2-10	3	4.0	5.0		GS	71							14.2				
WS2-11	1	2.0	3.0		GS												
WS2-11	2	4.0	5.0		GS												

1405906-CHEFORNAK PUMPHOUSE.GPJ LIBRARY-ANC(8-5-14).GLB [ANC_SAMPLE_SUMMARY] PCalvin 11/20/14

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SAMPLE LOCATION	SAMPLE NUMBER	DEPTH (ft)		RECOVERY (%)	SAMPLE TYPE	NATURAL MOISTURE CONTENT (%)	LIQUID LIMIT (LL) (%)	PLASTIC LIMIT (PL) (%)	PLASTICITY INDEX (PI) (%)	GRADATION (%)			ORGANIC CONTENT (%)	SALINITY (ppt) [(d) is directly meas.]	DESCRIPTION (USCS)	TESTS / OTHER TESTS	
		TOP	BOTTOM							GRAVEL	SAND	FINES (SILT & CLAY)					
WS2-12	1	0.8	1.3		GS	157											
WS2-12	2	3.0	4.0		GS												
WS2-12	3	5.0	6.0		GS	123											
WS2-13	1	3.0	4.0		GS									0(d)			
WS2-13	1	5.0	6.0		GS	600							40.9				
WS2-14	1	3.0	6.0		GS												
WS2-15	1	1.5	2.0		GS	24	26	25	1	0.0	1.5	98.5			ML		
WS2-15	2	4.5	6.0		GS	235											
WS2-16	1	3.0	4.0		GS												
WS2-16	2	4.0	5.0		GS												
WS2-16	3	5.0	6.0		GS												
WS2-17	1	3.0	5.0		GS												
WS2-18	1	3.0	4.0		GS	50											
WS2-18	2	4.5	5.5		GS	49											
WS2-19	1	3.0	4.0		GS												
WS2-19	2	4.5	6.0		GS												
WS2-20	1	2.0	3.0		GS	205											

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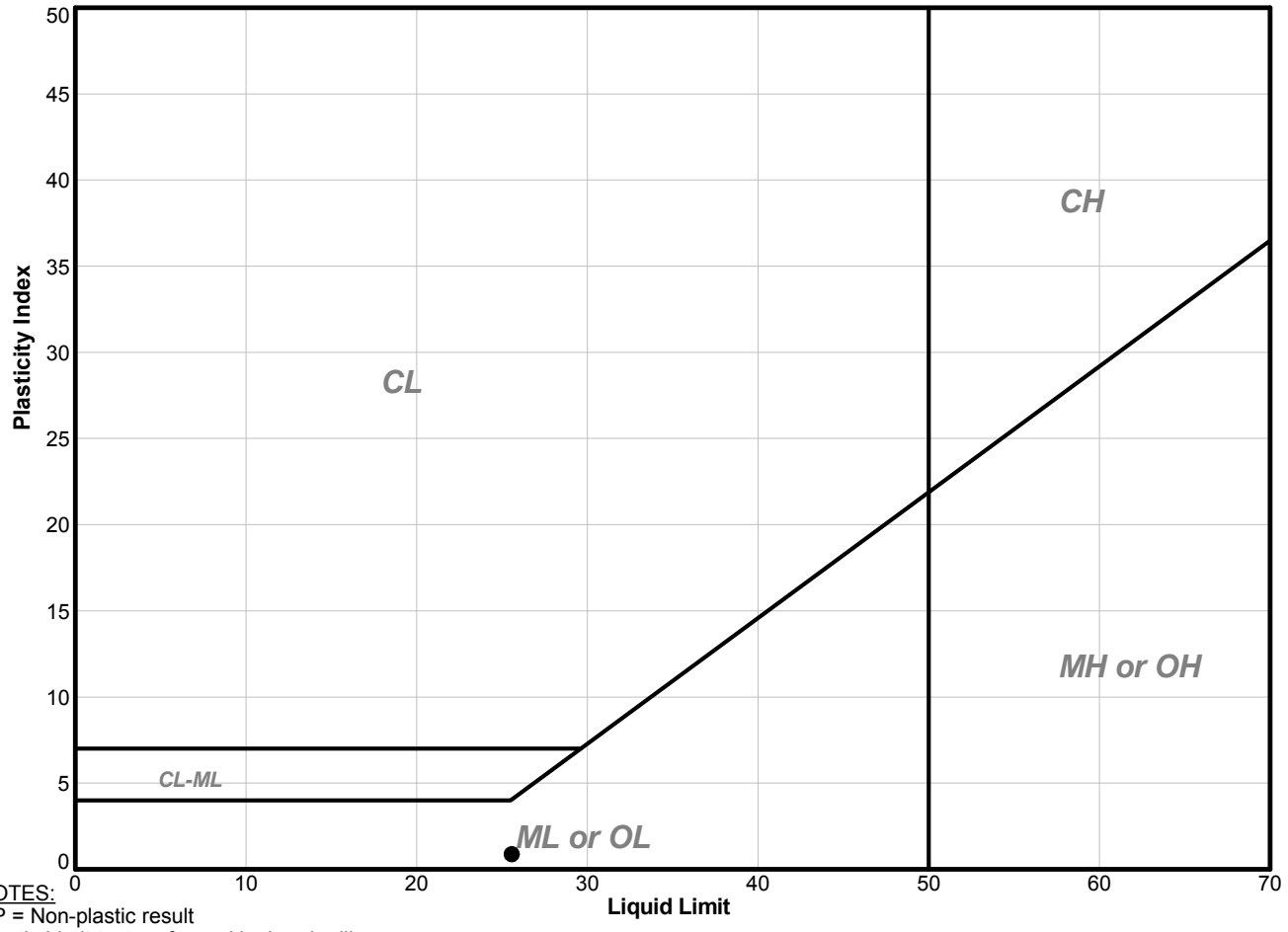
FIGURE B-1: LIQUID LIMIT, PLASTIC LIMIT AND PLASTICITY INDEX

Reference(s)
ASTM D 4318

Client:	CE2 Engineers Inc.	**Project No.:**	1405906		
Project:	Chefornak Pumphouse Sites	**QA/QC By:**	J. Randazzo	**Date:**	9/9/2014
Location:	Chefornak, AK	**Reviewed By:**	H. Weston	**Date:**	9/15/2014

PLASTICITY CHART



NOTES:
NP = Non-plastic result
Plastic Limit test performed by hand rolling
Liquid Limit test performed using mechanical device

	Sample Location	Sample Number	Depth (ft)	Bottom (ft)	Passing #40 Sieve (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index	USCS	Natural Moisture Content (%)
●	WS2-15	1	1.5	2.0		26	25	1	ML	24

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