Prepared for:

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Bettisworth North Architects and Planners Inc.
2600 Denali St. Ste 710
Anchorage, Alaska 99503

Haines Community Safety and Training Center

Geotechnical Report

Prepared by:

PND Engineers, Inc.
9360 Glacier Highway, Suite 100
Juneau, Alaska 99801

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Haines Community Safety and Training Center DRAFT Geotechnical Report

1. INTRODUCTION

This report presents the results of a geotechnical field investigation, performed by PND Engineers, Inc. (PND), in support of a new public safety building and parking lot for the Haines Borough. The development will cover approximately 3 acres of a currently vacant block of lots located near the intersection of Ed Shirley Drive and 2nd Avenue in Haines, Alaska. The current plans call for a 2-story building that will provide space for public employees including fire, police, dispatch, and EMS workers. The facility also includes detention quarters.

The report has been prepared by PND and provides geotechnical data gathered during the field investigation, laboratory results from testing performed on representative soil samples, as well as geotechnical engineering analyses and recommendations to be incorporated in the design of the site and structure. PND performed the work in accordance with the engineering scope of services and fee proposal, submitted to Bettisworth North Architects and Planners Inc. on September 20, 2021. The report contains six appendices:

- ✓ **Appendix A – Borehole Map and Logs** presents the borehole locations and complete graphical log set from the exploration.
- ✓ **Appendix B – Laboratory Test Results** presents the complete summary of results from the laboratory testing program.
- ✓ **Appendix C – Field Investigation Photographs** presents photographs taken during the field investigation.
- ✓ **Appendix D – Climate Summaries** presents data pertaining to historical temperature and precipitation variation near the project site.
- ✓ **Appendix E – Lateral Load Analysis** presents the results from the preliminary pile lateral load analysis.
- ✓ **Appendix F – GBA Publication *Important Information about your Geotechnical-Engineering Report*** describes the reports intended use, applicability and limitations.

2. PROJECT BACKGROUND

2.1 LOCAL GEOGRAPHY

Haines, Alaska is a census designated place (CDP) located on the Chilkat Peninsula, part of the greater Southeast Alaska Panhandle region. The Haines CDP encompasses approximately 21 square miles and is surrounded by the Chilkoot and Chilkat Inlets to the east and west, respectively. The 2020 population of Haines was 2,080 residents (United States Census Bureau). Haines can be accessed via the Haines Highway or by regularly scheduled commercial vessel or plane services.

The project site is located immediately south of the existing Haines Police Department building (Figure 2-1); situated between 3rd Avenue, Ed Shirley Drive (Cox Avenue), 2nd Avenue, and an unnamed right of way. The site is comprised of 16 individual lots (Lots 1 through 16, collectively Block M) transected by an alley. All lots and the alley are owned by the Haines Borough. A topographic survey was performed by PND in 2021 and is included in the project plans.

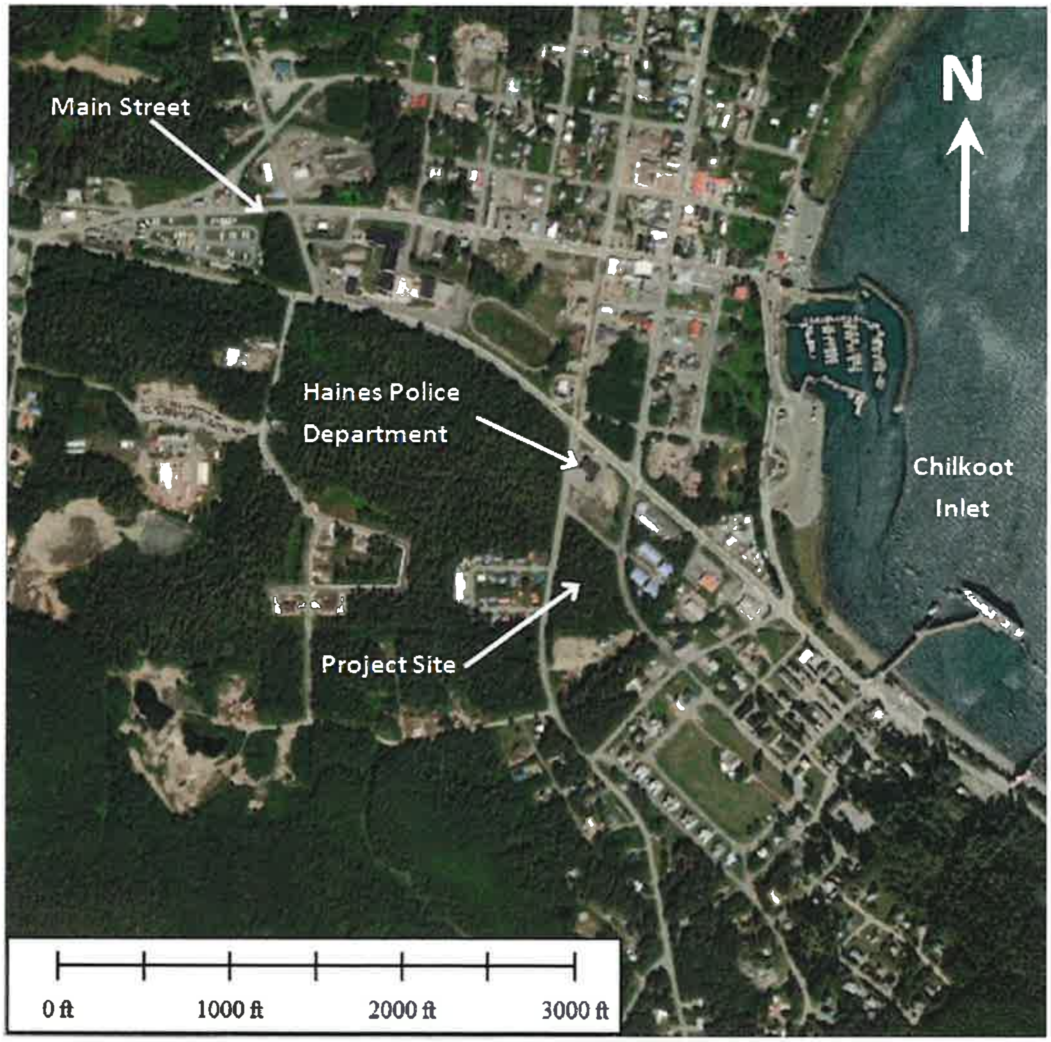


Figure 2-1: Project Location in Haines, Alaska

2.2 EXISTING SITE CONDITIONS

The project area is located at 59°13'48.81"N latitude, 135°26'52.22"W longitude on an undeveloped block of land. Site elevation ranges from about +47 feet mean lower low water (MLLW) at the north corner to +68 feet MLLW at the south corner. Much of the site is gently sloping, with the exception of the steeper southern corner. No existing access roads or utilities going through the block of lots.

At the time of the investigation, the project site was heavily vegetated with understory and thick forest. The ground surface was generally soft, saturated, and muddy. Standing water was noted along the northeast property line.

2.3 REGIONAL CLIMATE

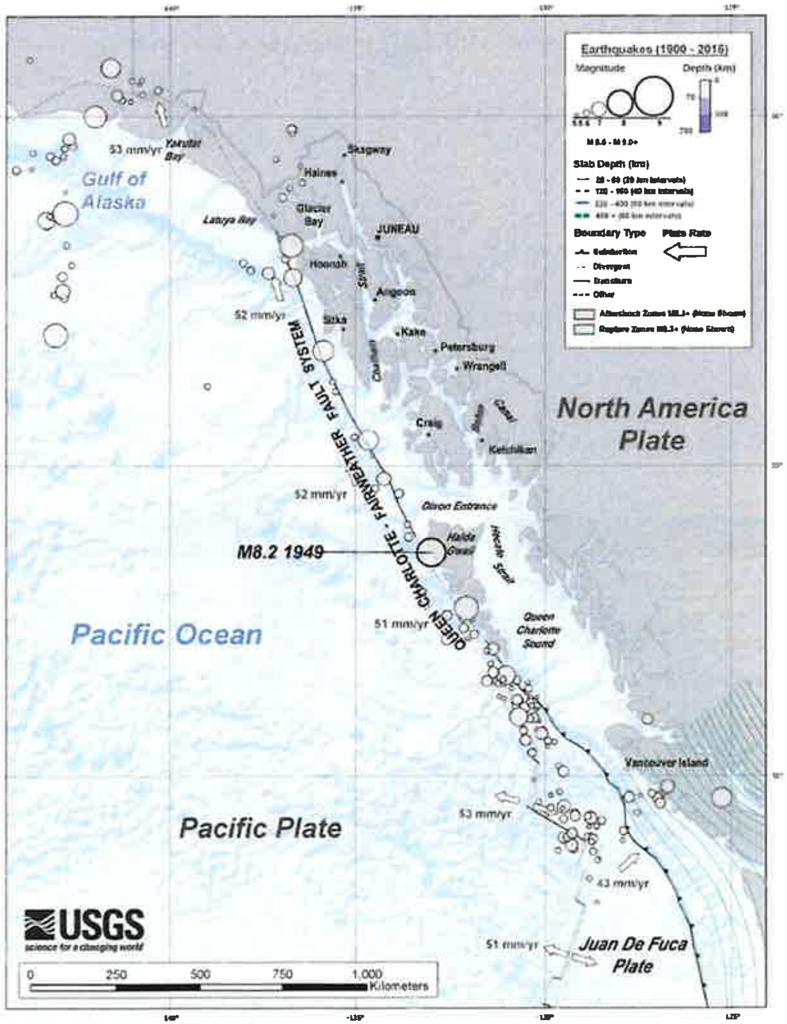
Haines experiences humid continental climate conditions with dry summers and wet winters. The average monthly temperatures recorded in Haines typically range from 16°F in January to 58°F in July. Daily extreme temperatures range from (-)24°F to 92°F. Average annual precipitation typically ranges from 32 to 73 inches per year. This climate data was obtained from the Western Regional Climate Center Haines station (Station 503504) for the years 1989 to 2012 and is attached in Appendix D.

2.4 REGIONAL SEISMICITY AND SEISMIC HAZARDS

The regional seismicity of Southeast Alaska is primarily defined by four known major faults: the Queen Charlotte-Fairweather Fault, Chatham Strait Fault, Denali Fault, and the Transition Fault. These four known faults are the main contributors to the seismic hazard of the project site. Wesson et al., (2007) found that these four faults could yield maximum moment magnitude (M_w) earthquakes of 7.8 to 8.2 for return intervals of 2% in 50 years. These ranges are consistent with the larger historic earthquakes that have previously been documented or recorded in Southeast Alaska (Brockman et al., 1988). Earthquakes of M_w 5.0 or less are common to Southeast Alaska, although they present low hazard to the project site.

The primary seismic-induced hazards for Haines and the surrounding region include strong ground shaking, slope failure, liquefaction, and landslides. Both the seismic setting and glacially-scoured, over-steepened terrain of the region contributes to the potential for both land-based and submarine landslides caused by earthquake-induced ground shaking or other triggering events, and are most likely to occur in saturated sediments and in unstable rock debris on steep slopes. Figure 2-2 shows previously recorded seismic activity in the Southeast Alaska region.

SOUTHEAST ALASKA SEISMICITY



Map Modified From: USGS Earthquakes Hazard Program

Figure 2-2: Seismic Activity and Features in Southeast Alaska

2.5 SITE GEOLOGY

The region's geomorphic features and extensive sediment deposits record a relatively brief, recent period in the geologic history of the region dominated by Quaternary-age glacial processes. Preserved in broad braided flood plains, intermontane alluvial fans, intertidal deltas, and marine basins are substantial amounts and types of glacially derived sediment, including boulders scattered throughout the region and extensive thick deposits of glacial outwash.

The surficial soils found in the vicinity of Haines are generally glacially derived sediments and include glacial erratics, glaciomarine diamicton, and outwash deposits. Glacial erratics are cobble- to boulder-sized rocks that were rafted aboard the glaciers and deposited throughout the surrounding Haines drainage valleys, upland marine shorelines, and lower intertidal beach areas and marine sediment floors. Glacial erratics are known to reach enormous sizes.

Glaciomarine diamicton deposits, typically massive and compact to dense, are comprised of coarse-grained sands and gravels up to cobble and boulder-sized rock, supported within a finer-grained matrix of primarily cohesive silt and clay with sand. Abundant broken and unbroken marine fossils may be present in diamicton deposits.

Other glacial sediments in the area include outwash deposits shed from the outlying river valley drainages, consisting of varying proportions of boulders, cobbles, gravels, sands, silts, and clays.

3. FIELD INVESTIGATION

3.1 GEOTECHNICAL EXPLORATION

The geotechnical investigation was performed from May 6th through May 11th, 2022 with Discovery Drilling, Inc. Six boreholes (BH-1 through BH-6) were advanced at the project site to depths ranging from 20 to 102 feet below the existing ground surface (bgs). Three of the six boreholes (BH-1 through BH-3) were located within or near the proposed building footprint while BH-4 through BH-6 were located near the proposed access road and parking areas. All boreholes were advanced using a track-mounted Geoprobe 6712DT drill rig. PND field personnel provided oversight, directed the work, and documented findings during the investigation. Approximate borehole locations are presented in Table 3-1 and are depicted graphically in Figure 3-1. A borehole location map that includes the topography of the site is also presented in Appendix A.

Table 3-1: Borehole Summary

Borehole	Latitude	Longitude	Collar Elevation (ft)	Total Depth (ft bgs)
BH-1	59.230500	135.447783	48.6	101.5
BH-2	59.230233	135.447233	49.1	101.5
BH-3	59.230200	135.448383	53.4	91.5
BH-4	59.230400	135.448317	49.3	32
BH-5	59.229850	135.447733	54.5	20
BH-6	59.229817	135.447033	55.5	32

Note: Coordinates based on WGS84 coordinate system. Elevations based on MLLW.

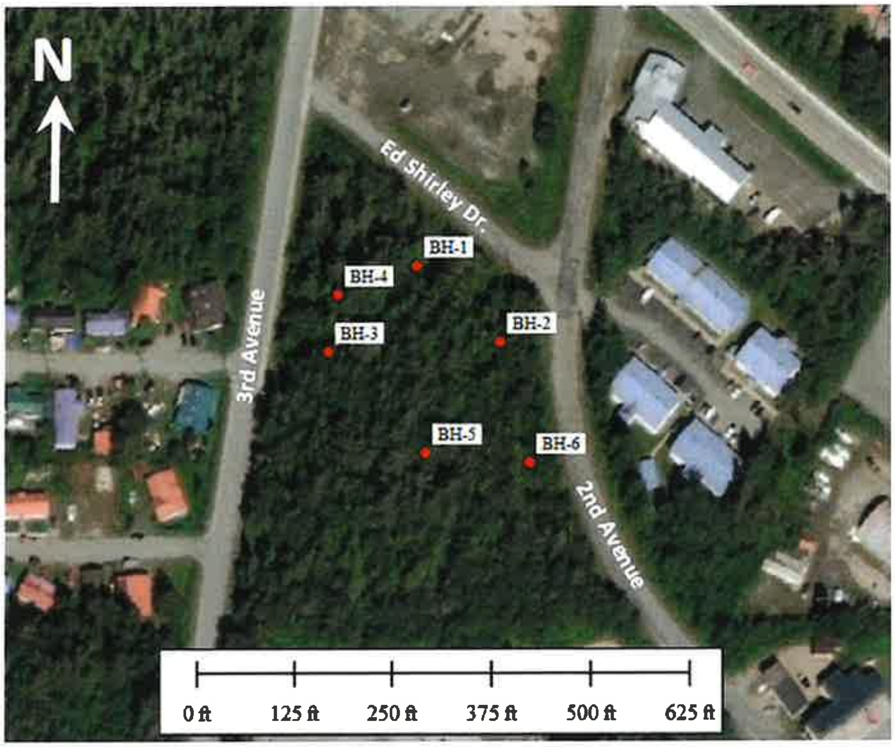


Figure 3-1: Borehole Location Map

3.2 EQUIPMENT AND METHODS

BH-1 through BH-3 were advanced using a combination of water- and mud-rotary drill tooling consisting of a tri-cone bit HWT casing advancer system. BH-4 through BH-6 were advanced using a hollow stem auger. Modified Penetration Tests (MPTs) were conducted using oversized split-spoon samplers and a 340-pound automatic drop-hammer falling 30 inches per stroke. The number of blows required to drive the sampler for each 6-inch interval, for a maximum total distance of 24 inches were recorded. MPT tests were generally performed every 5 feet at depths less than 50 feet bgs and every 10 feet at depths greater than 50 feet bgs. When predominantly fine-grained, saturated soils were encountered, the undrained shear strength was measured using a standard handheld shear vane. The blow counts shown on the borehole logs (presented in Appendix A) are field values that have not been corrected for overburden, rod length, and other factors. Shelby tube samples were also collected from BH-1 through BH-3 at various depths in order to collect relatively-undisturbed samples of the predominantly fine-grained material at the project site. MPTs were not performed at depths where Shelby tube samples were collected. Sampling methods and classifications for soil were based on the Unified Soil Classification System (USCS) and the following American Society for Testing and Materials (ASTM) standards:

- ASTM D1586 *Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils*
- ASTM D2488 *Standard Practice for Description and Identification of Soils (Visual-Manual Procedure)*
- ASTM D5434 *Standard Guide for Field Logging of Subsurface Explorations of Soil and Rock*

4. LABORATORY TESTING

Retained soil samples from the geotechnical boreholes were transported to PND's Soils-Materials laboratory in Anchorage for additional testing upon completion of the field investigation. A total of 124 laboratory tests were performed on the soil samples in accordance with the ASTM standards given in Table 4-1.

Table 4-1: Soil Sample Testing Summary

Test Type	Quantity
Description and Identification of Soils – Visual Manual Procedure (ASTM D2488)	60
Moisture Content of Soils (ASTM D2216)	60
Gradation of Soils (ASTM D6913)	1
Atterberg Limits (ASTM D4318)	3

5. INVESTIGATION RESULTS

5.1 SOIL LITHOLOGY AND COMPOSITION

The soil lithology at the project site generally consists of surficial organics underlain by soft silty clay and lean clay to depths of about 85 feet bgs. Below about 85 feet bgs, the soil is dense to very dense clayey sand with gravel and stiff sandy clay.

All laboratory test results, including a summary of soil classifications, gradation plots, and a plot of soil moisture content versus depth are presented in Appendix B. Select photographs taken during the field investigation are shown in Appendix C.

Figure 4-1: Very dense clayey sand with gravel encountered 100 feet bgs in BH-1.

5.2 GROUNDWATER

Test pits performed by PND during the preliminary geotechnical investigation in 2021 indicated that the groundwater surface is located approximately 3 feet bgs, near the interface between the surficial organic-rich material and the silty clay. All boreholes advanced for this investigation supported the previous findings as all material below 3 feet bgs had a wet, saturated consistency.

5.3 CORRECTED BLOW COUNTS

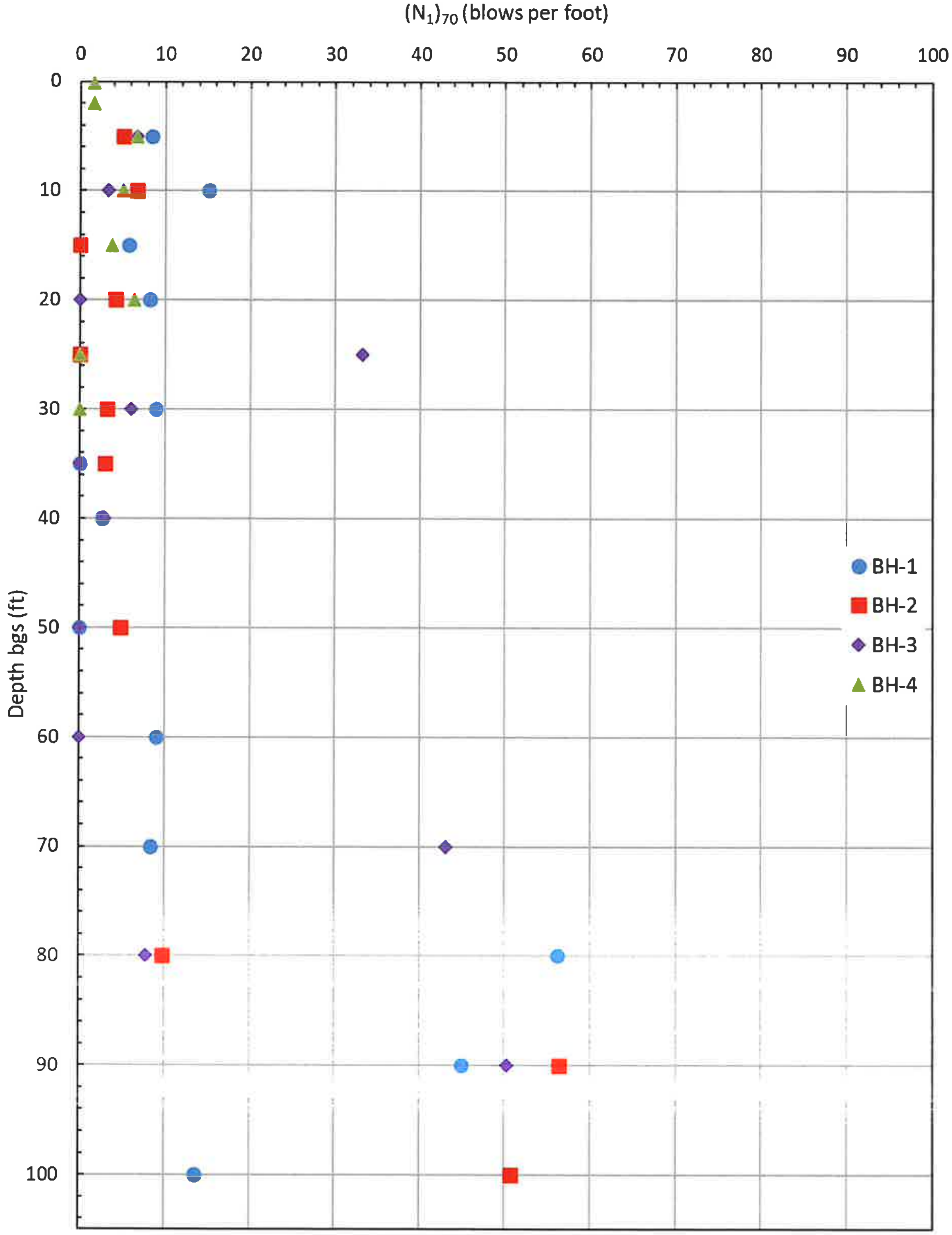


Figure 5-2 shows corrected blow counts versus depth below the existing ground surface at the project site.

Field blow counts were corrected using standard correlations found in most geotechnical texts. Note that no blow counts were recorded in BH-5 and BH-6 because the split-spoon samplers were advanced using a constant push rather than by strikes of the automatic trip hammer. The undrained shear strength of retrieved samples was measured, where applicable, using a standard shear vane. A plot of undrained shear strength versus depth is presented in Figure 5-3.

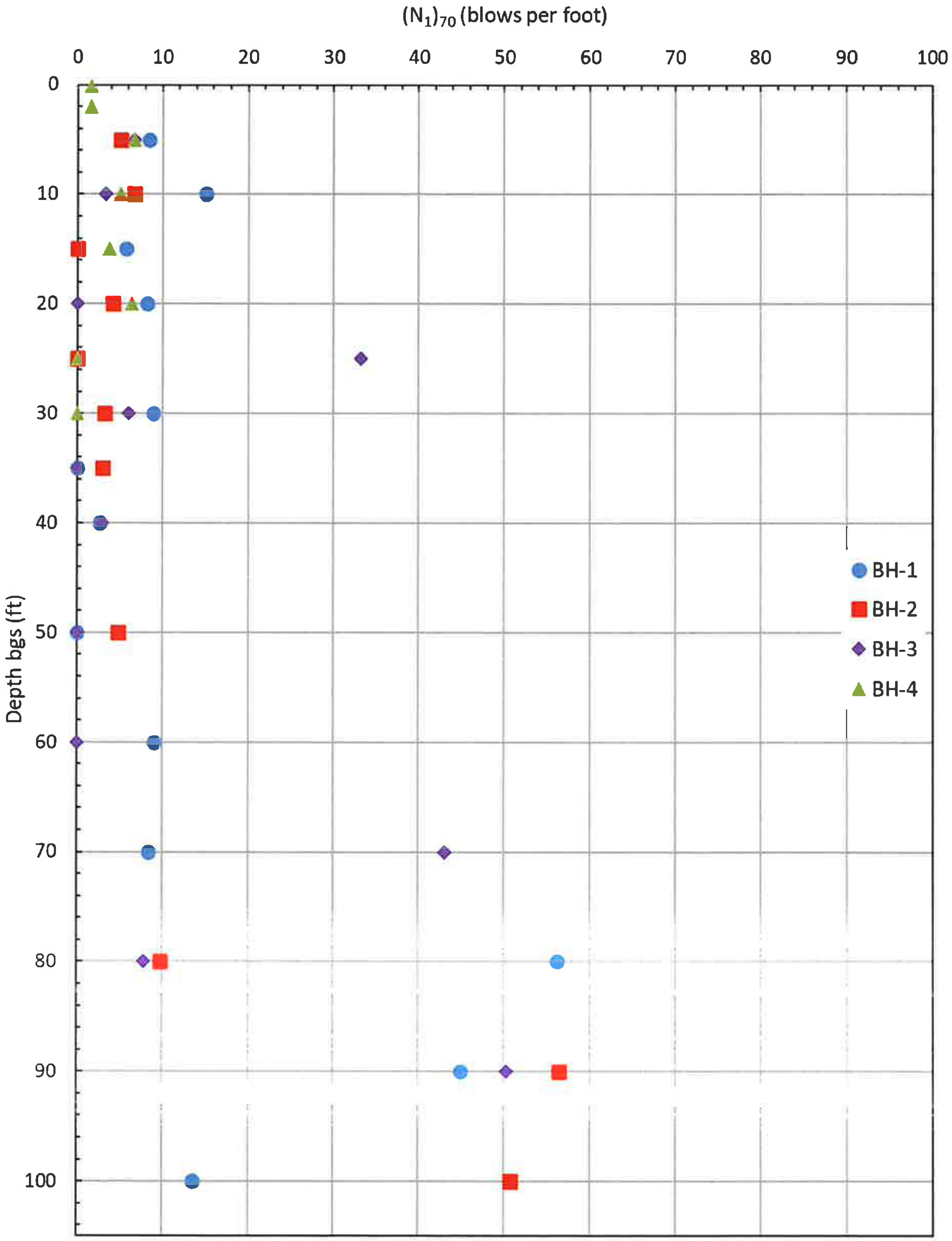


Figure 5-2: Corrected Blow Counts $(N_1)_{70}$ Versus Depth

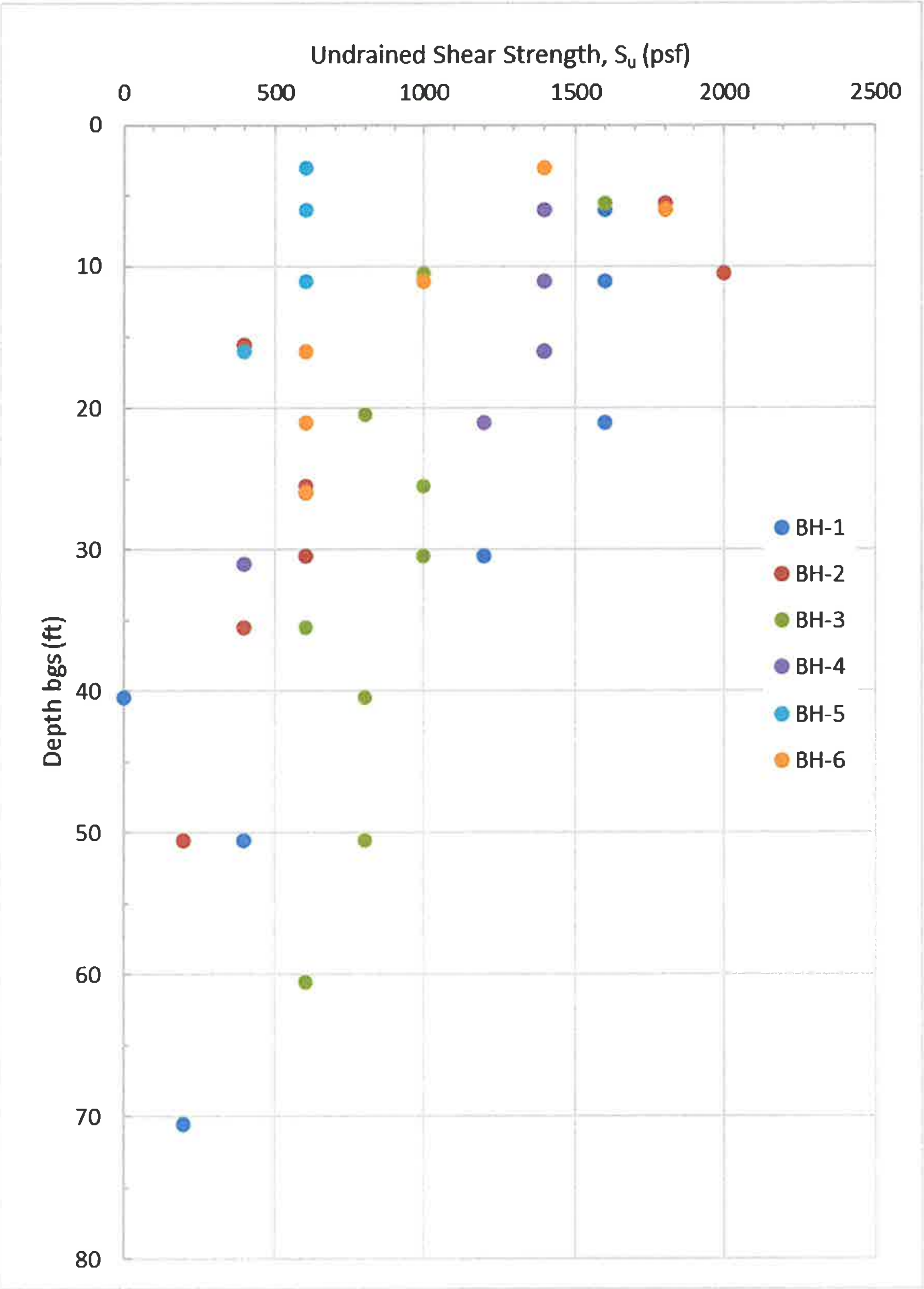


Figure 5-3: Undrained Shear Strength, S_u Versus Depth

6. GEOTECHNICAL ANALYSES and DESIGN RECOMMENDATIONS

6.1 SUMMARY

The following subsections present design parameters and recommendations based on the findings of the geotechnical investigation and known aspects of the project. Recommendations are general and subject to change as design progresses.

Significant deposits of compressible soil at the site are not conducive to conventional shallow foundations, which could settle under the predicted building loads. Deep foundations are recommended. Parking lot and utility excavations should include the use of a geotextile to help stabilize imported fill and keep fill segregated from soft fine-grained soils below.

6.1 DESIGN SOIL PROPERTIES

Table 6-1 presents the generalized soil properties recommended for design at the project site. Values for future Structural Fill are estimated based on experience with the specified material. All other parameters for the in-situ material were estimated based on blow count correlations found in most geotechnical textbooks, undrained shear strength measurements using a standard shear vane, engineering judgement, and experience. Note that the soil properties for the surficial organics at the site were omitted because it is assumed that they will be removed during construction.

Table 6-1: Generalized Soil Properties

Soil Type	Total Bulk Unit Weight (pcf)	Peak Effective Friction Angle, ϕ' (degrees)	Undrained Shear Strength, S_u (psf)
Structural Fill (Future)	140	40	-
Soft Silty Clay (CL-ML)	115	-	1000
Soft Lean Clay (CL)	105	-	500
Dense Clayey Sand with Gravel (SCg)	135	35	-

6.2 SEISMIC DESIGN PARAMETERS

Seismic parameters for the project site are given in Table 6-2. Moment magnitude was obtained from the United States Geological Survey (USGS) online Unified Hazard Tool. All other parameters were obtained from the Applied Technology Council (ATC) Hazards by Location online utility, ASCE 7-16 section 11.4.8, and Supplement 1 of ASCE 7-16 with the understanding that all structural design will utilize the 2021 version of the International Building Code.

Table 6-2: Seismic Design Parameters Per ASCE 7-16

Seismic Event (Return Period, Probability of Exceedance):	MCE (2,475 years, 2% in 50 years)
Site Class	E
Risk Category	IV
Peak Ground Acceleration (PGA)	0.50g
S_S (0.2 sec period acceleration)	1.09
S_1 (1.0 sec period acceleration)	0.54
F_{PGA}:	1.20
F_a:	1.20
F_V:	2.12

S_{DS}:	0.87
S_{D1}:	0.76
Site Modified PGA (PGA_M)	0.60
Moment Magnitude (M_W):	7.9

The parameters presented in Table 6-2 are applicable for structures with $T \leq T_s$ (where T is the structure's fundamental period and $T_s = S_{D1}/S_{DS}$) and the equivalent static force procedure is used for design, per ASCE 7-16 section 11.4.8. If these conditions are not satisfied, a site-specific ground motion analysis may be required.

6.3 LIQUEFACTION ANALYSIS

Liquefaction occurs when excess pore water pressure develops in saturated soils, typically as a result of vibrations or ground shaking caused by earthquakes, which results in a reduction of shear strength in the soil. Detrimental effects of liquefaction can include ground deformations in liquefied soils, slope instability, lateral spreading, and reduced bearing capacity due to loss of shear strength. Typically, liquefiable soils are very loose- to medium-dense, clean to moderately silty sands; liquefiable soils also include some silts. Other soils may experience liquefaction depending on other factors, such as the duration of ground shaking. Gravels and soils with high fines content (greater than approximately 35%), which exhibit clay-like behavior (Plasticity Index ≥ 7), are unlikely to liquefy, but may soften during cyclic earthquake loading (Boulanger and Idriss, 2006). Soils beyond a certain depth, often estimated at a maximum of 50 feet, may not liquefy due to high confining pressures (Day, 2002). This depth is based on site-specific conditions and engineering judgement and should not be assumed for all cases.

Most of the soils encountered at the project site that were above 50 feet bgs were fine-grained and had plasticity indices ranging from 6 to 9. The moisture contents in this depth range varied from 20 to 40 percent; indicating that the soil above 50 feet bgs will behave in a "clay-like" manner when subjected to cyclic loading according to Boulanger and Idriss (2006). Although there is a potential for cyclic softening to occur at the project site during the design earthquake, the performance of the structure will not be negatively affected if the foundation recommendations made in this report are followed.

6.3.1 LIQUEFACTION-INDUCED SETTLEMENT AND PILE DOWNDRAG

Considering the plasticity of the soft clay at the project site and the short duration that is typical of earthquakes in Southeastern Alaska, liquefaction settlement and the development of pile downdrag forces are not likely to be a concern for this project. Nevertheless, the foundation recommendations in the following sections considered the additional load that could develop due to cyclic softening and, if followed, will mitigate the potential for liquefaction settlement and the development of pile downdrag forces.

6.3.2 LATERAL SPREADING AND SLOPE STABILITY

Lateral spreading occurs when an unconfined soil or soil whose strata interfaces have non-horizontal gradient liquefies and behaves as a fluid. Due to lack of confinement or inclined stratification, the soil will flow and result in vertical and horizontal deformations.

While the soils at the project site have a potential to exhibit cyclic softening, they are generally confined and are stratified essentially horizontally. The site is also relatively flat so lateral spreading and slope instability are not expected to be an issue at the project site.

6.4 FOUNDATION RECOMMENDATIONS

Considering the soft nature of the fine-grained soils located in the upper 85 feet of the soil profile at the project site, and the performance requirements of a Risk Category IV facility, PND recommends that the proposed structure be supported by a pile foundation system. The foundation piles should extend in to the stiff/dense stratums of soil that were encountered in BH-1 through BH-3 below depths of approximately 85 feet bgs. The following sections present pile foundation options and additional aspects of the foundation that the structural designer should consider as the design process continues.

6.4.1 ALLOWABLE AXIAL PILE CAPACITY

Allowable axial pile capacities were calculated for ½-inch wall thickness open- and closed-ended pipe piles ranging from 12 to 18 inches in diameter. For all axial compressive capacity calculations, the soil strength between 3 and 50 feet bgs was neglected for load transfer considerations. This neglection also applies to a seismic condition in which the soil has softened and would not likely contribute to structural resistance.

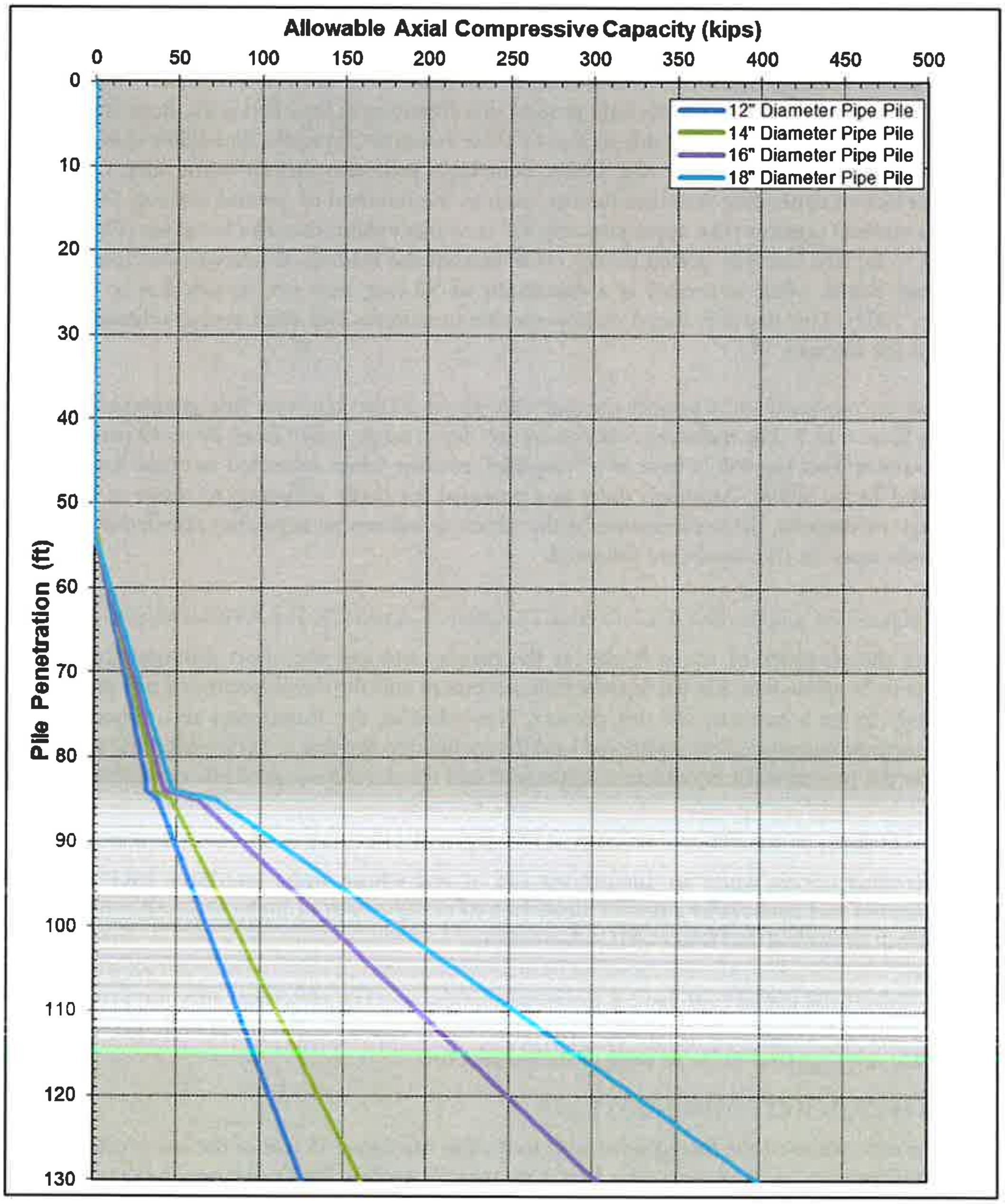


Figure 6-1: Allowable Compressive Capacities for Open-Ended Pipe Piles Under Seismic Conditions

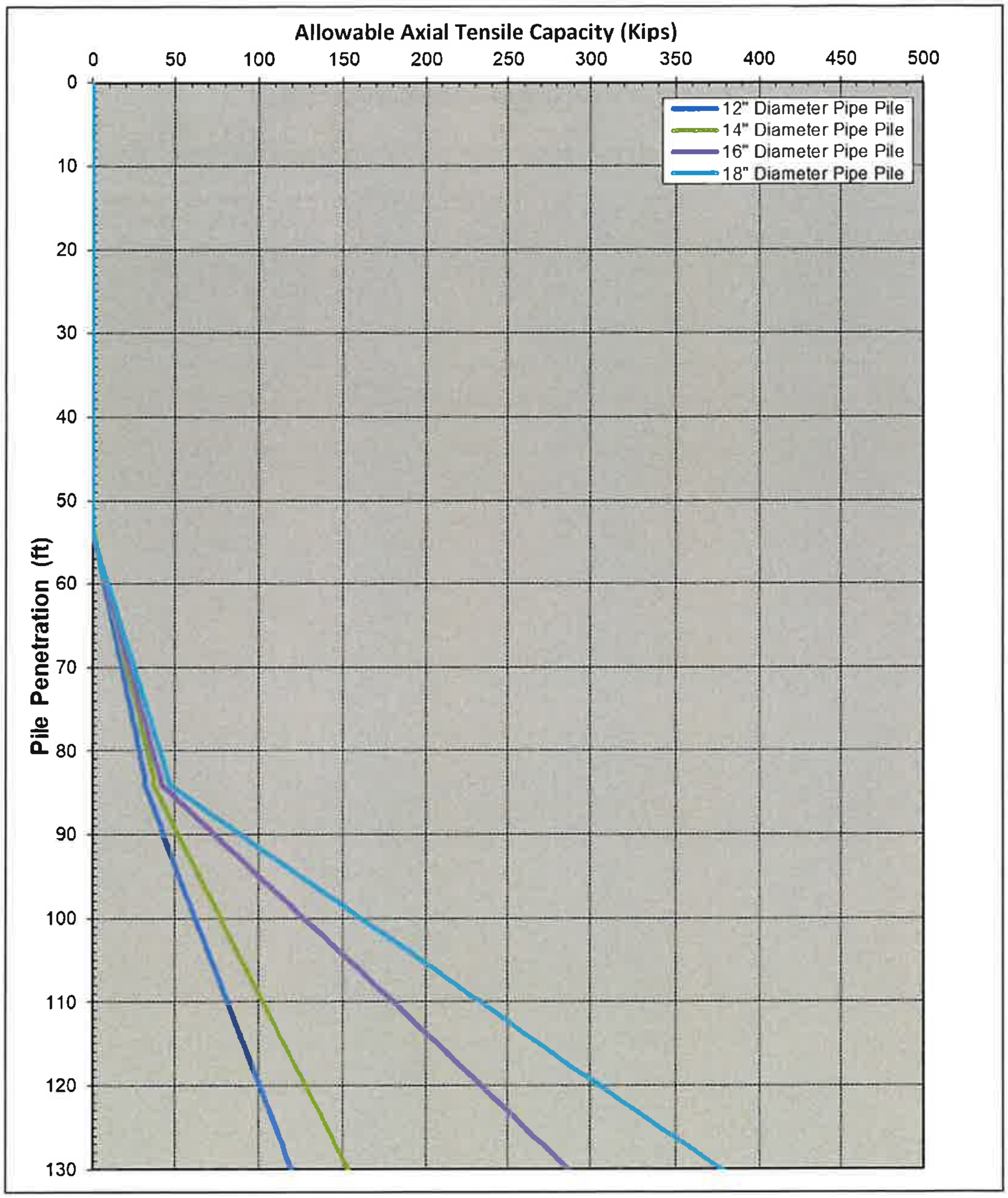


Figure 6-2: Allowable Tensile Capacities for Open-Ended Pipe Piles Under Seismic Conditions

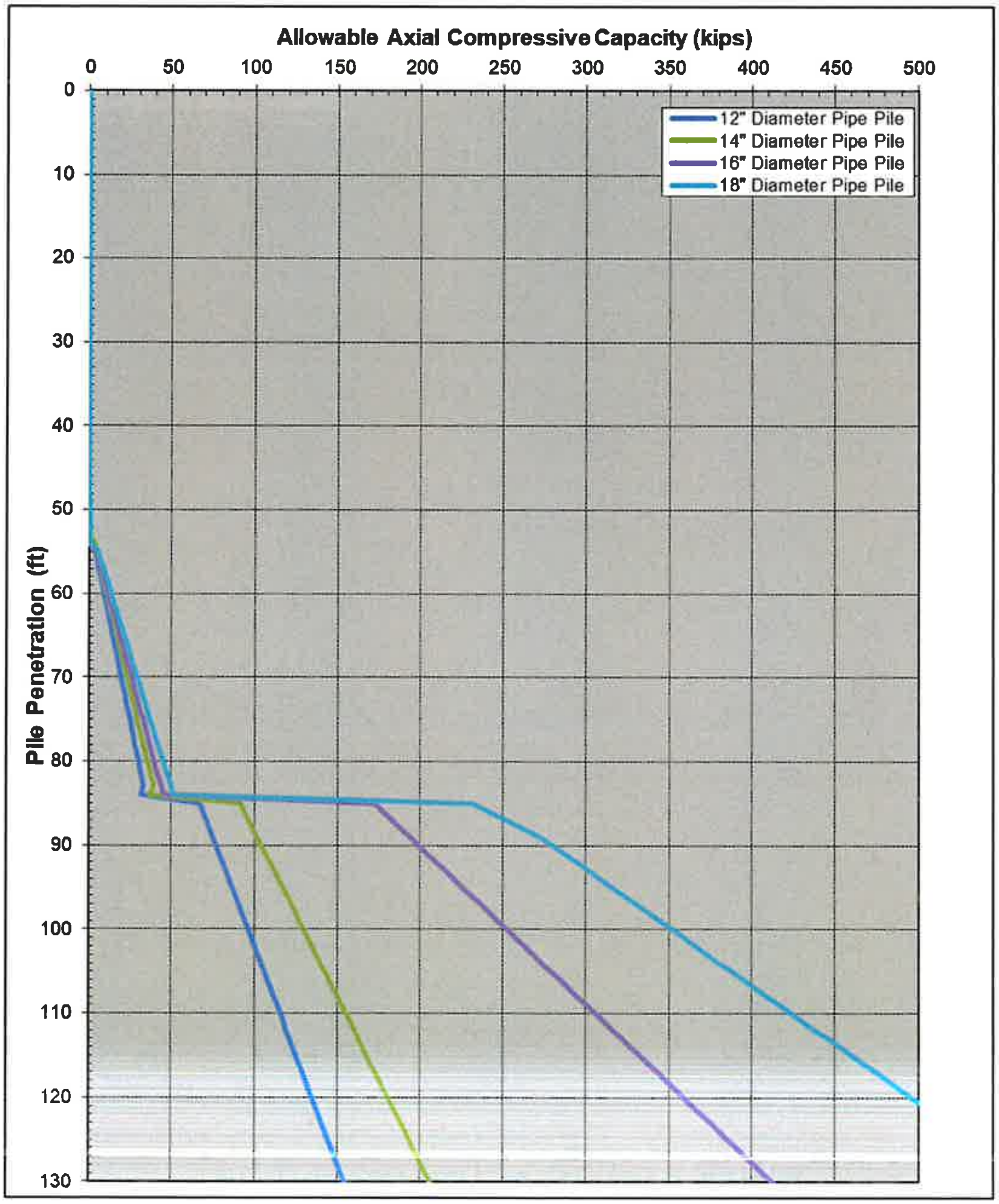


Figure 6-3: Allowable Compressive Capacities for Closed-Ended Pipe Piles Under Seismic Conditions

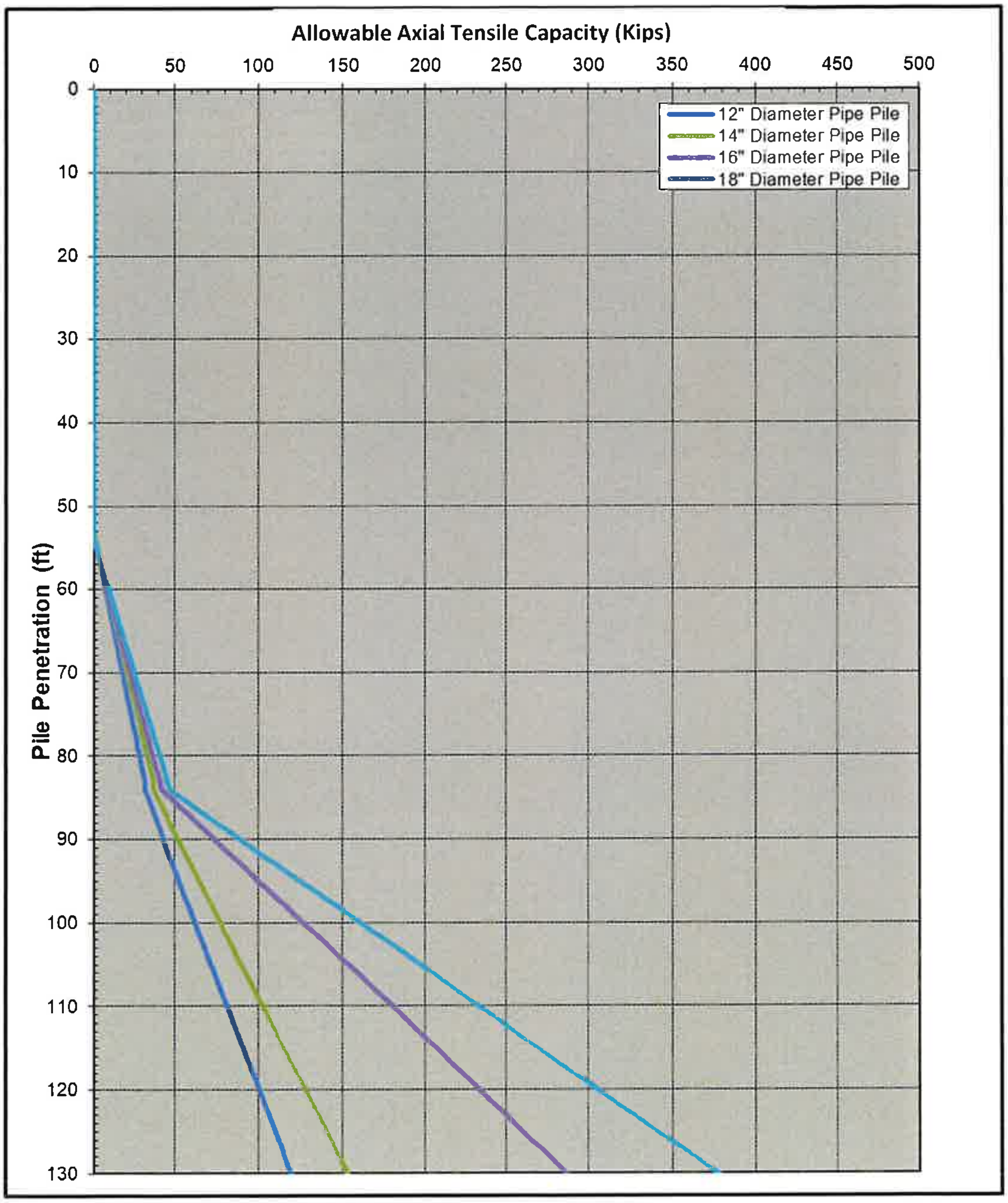


Figure 6-4: Allowable Tensile Capacities for Closed-Ended Pipe Piles Under Seismic Conditions

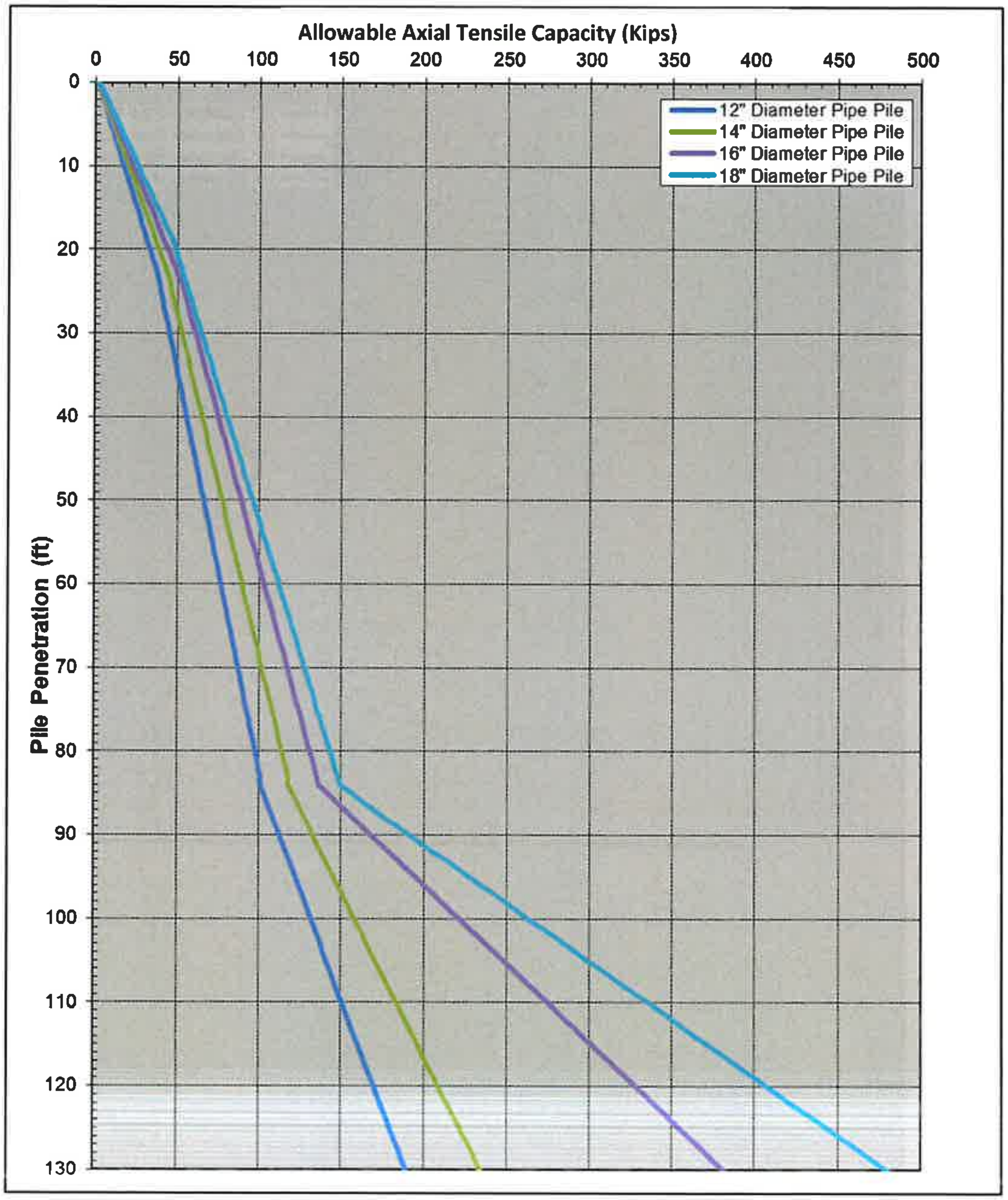


Figure 6-5: Allowable Tensile Capacities for Open-Ended Piles under Non-Seismic Conditions

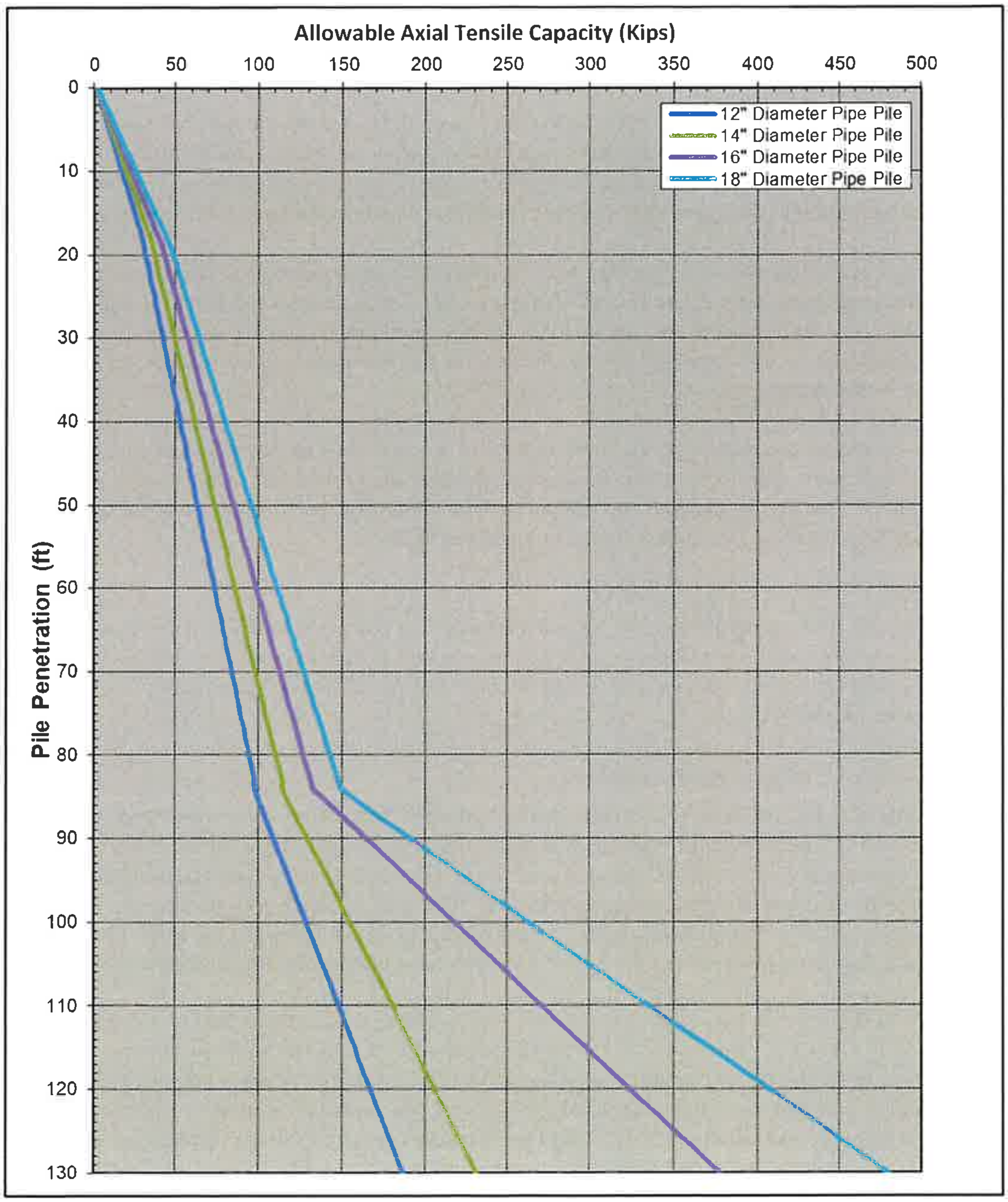


Figure 6-6: Allowable Tensile Capacities for Closed-Ended Piles under Non-Seismic Conditions

The capacities shown in these figures are allowable capacities that incorporate a factor of safety of 1.5. The capacities do not account for pile group effects. Per ASCE 7-16 section 12.13.8.8, pile group effects must be accounted for in the calculation of vertical capacities if the nearest neighboring pile is less than a distance of three pile diameters away. If closed-ended pipe piles are selected, PND recommends that the structural design team specify conical tips, as opposed to plug plates, for all foundation piles. Piles should be driven to a minimum tip elevation of approximately – 40 feet MLLW, or deeper, to ensure that all piles are bearing on the dense stratums encountered during the field investigation. This should be verified by a qualified inspector during construction.

6.4.2 ELASTIC SETTLEMENT, PRIMARY CONSOLIDATION, AND SECONDARY SETTLEMENT

Long-term settlement occurs when soil consolidates over time due to an induced load, such as from embankments or structural foundations. Two types of long-term settlement are typically considered when designing a foundation: primary consolidation and secondary compression. Both are typically attributed to compressible soils such as saturated silts and clays, as well as organic-rich soils such as peat and organic silt.

Although most of the soil above 85 feet bgs is prone to primary consolidation, embedding the piles into the dense/stiff bearing layers encountered in BH-1 through BH-3 will minimize the load being transferred via skin friction to the fine-grained soil above 85 feet bgs. Settlement due to primary consolidation and secondary settlement is not expected to be an issue if the recommendations made in this report are implemented in the design.

Elastic pile settlement calculations performed using Rocscience's RSPile software indicated that less than 1/2-inch of settlement will occur if the piles are embedded into the dense stratums encountered below approximately 85 feet bgs. A detailed settlement analysis should be performed once the design team has finalized the pile geometries, spacing, and pile embedment depths.

6.4.3 DIFFERENTIAL SETTLEMENT

The compressible soils at the project site were distributed in a relatively uniform manner and most of the building loads will be transferred to the dense stratums starting at 85 feet bgs. Differential settlement of the building's foundation is not a concern for this project, provided that all recommendations in this report are implemented in the design.

6.4.4 LATERAL LOAD RESISTANCE

Temporary lateral loads are likely to develop on the building's foundation from wind and seismic loading. These forces will be counteracted by passive pressures that will develop between the sides of the piles and the soil surrounding them, as well as the stiffness of the pile itself. Preliminary lateral load analyses were performed for 16- and 18-inch diameter pipe piles with 3/8- and 1/2-inch wall thicknesses. The piles were assumed to have an 8-kip lateral load applied to the pile head and a 200-kip axial load. For these loading conditions and pile geometries, all piles were estimated to have pile head deflections less than 0.1 inches.

These pile head deflection estimates assume that all piles will be spaced sufficiently far enough apart to avoid group effects. Per ASCE 7-16 section 12.13.8.8, pile group effects must be accounted for in the calculation of lateral capacities if the nearest neighboring pile is less than a distance of eight pile diameters away. Once pile loads and geometries have been finalized, an updated pile lateral load analysis should be conducted. Table 6-3 presents the soil parameters that PND recommends for use in future lateral load analyses.

Table 6-3: Recommended Soil Parameters for Lateral Load Analyses

Soil Type	Soil Model	Effective Unit Weight (pcf)	Friction Angle (degrees)	Undrained Cohesion, c_u (psf)	Strain Factor, E50
Structural Fill (Future)	Sand (Reese)	140	40	-	-
Silty Clay (CL-ML)	Soft Clay (Matlock)	53	-	1000	0.02
Lean Clay (CL)	Soft Clay (Matlock)	43	-	500	0.02

Clayey Sand with Gravel (SCg)	Sand (Reese)	73	35	-	-

6.4.5 UPLIFT RESISTANCE

Uplift loads may manifest in some foundation elements due to overturning moments that occur as a result of wind and seismic forces. Uplift loads may be resisted by the weight of the building and the shaft friction that will develop between each pile and the soil surrounding it. The capacities shown in Figure 6-2 and Figure 6-4 present the estimated allowable uplift resistance that each pile will provide.

7. CONSTRUCTION RECOMMENDATIONS

This section provides recommended construction practices and materials for the project. In general, practices and materials should conform to the IBC adopted by local authorities having jurisdiction, the local design specifications, and any other project-specific design criteria.

7.1 SITE PREPARATION

All trees, small brush, surface debris, organics, and other deleterious surficial materials should be removed prior to constructing the building's foundation and the parking lot's pavement section.

7.2 EXCAVATIONS

Temporary excavations of soil should be performed with care and follow OSHA or other agency guidelines and recommendations for trenching and slope angles based on the soil types encountered.

Excavations should be performed utilizing a backhoe or excavator with a smooth-bladed bucket from outside the excavation to minimize disturbance of the subgrade soils. Soils that are disturbed, pumped, or rutted by construction activity should be removed prior to placement of any structural, classified, or unclassified fill as recommended in this report.

7.3 DRAINAGE AND CONTROL OF WATER

Water may flow into excavations due groundwater, or rain and surface runoff during periods of intense rainfall. Excavations should be monitored at all times for excessive water accumulation that could endanger workers or negatively affect the project. The contractor may be required to implement measures to control water flow and effectively dewater the site depending on the magnitude and impact of rain and surface runoff. These measures may include installing and grading perimeter trenches, sump wells with pumps, or other water control measures. Significant groundwater infiltration at the top of the lean clay layer, approximately 3 feet bgs, should be anticipated.

Parking areas should have a positive gradient toward drainage structures and away from buildings. Site grading should be established to provide drainage of surface water or roof drainage away from proposed and existing buildings and toward suitable drainage structures.

7.4 CONSTRUCTION MATERIALS AND COMPACTION

This section provides general recommendations for the use of aggregate (structural) fill to be used during construction. Generally, imported structural fill should comply with the project specifications, with modifications as determined necessary. All structural fill should be angular, clean, sound, durable, and free of any frozen clumps, ice, or any deleterious material prior to placement. Structural fill should follow all project specifications and be a well-graded mixture of non-frost susceptible (NFS) sand and gravel. For the purposes of this report, structural fill can be segregated into two material sub-types: subbase and base course.

Subbase should have a maximum particle size of 4 to 6 inches and less than 6% passing the No. 200 sieve size. Subbase shall be placed In lifts not exceeding 12 inches in loose thickness. Compaction of subbase shall be achieved by performing a minimum level of effort consisting of six complete passes with a 15-ton vibratory steel drum roller. In areas that are too small to accommodate a roller, compaction shall be accomplished by a minimum level of effort of six complete passes with a vibratory plate compactor with a minimum rated centrifugal force of 15,000 lbs.

Base course should have a maximum particle size of 1 to 1.5 inches and less than 6% passing the No. 200 sieve size. Base course shall be placed in lifts not exceeding 8 inches in loose thickness and shall be compacted to 95% of the maximum density as determined by a control strip test, such as Alaska Test Method (ATM) 309 Relative Standard Density of Soils by the Control Strip Method.

In areas where fill sections exceed the minimum 42-inch structural section, a coarser fill material may be used below the subbase. This coarser material should have less than 6% passing the No. 200 sieve and a maximum particle size of 12 inches. This material may not be desired for areas containing underground utilities as it may be difficult to excavate.

All structural fill material should be protected from freezing during construction. No frozen soil should be used as fill, nor should any fill be placed over frozen soil. Any frozen soil should be removed, replaced, or thawed prior to fill placement.

Moisture control of materials should be implemented when stockpiling and placing fill material. Stockpiles should be covered to prevent saturation during wet weather conditions Additional moistening or drying of fill material may be required in order to obtain the optimum moisture content for maximum compaction.

No hauling or grading equipment should be used in lieu of appropriate compaction equipment. Any loosening of fill material by hauling or other equipment should be repaired by re-compacting as needed. The number of passes required to meet the compaction requirement will depend on the size of the compaction equipment used.

7.5 PILE DRIVING

Prior to driving pile for the foundation, a wave equation pile driving analysis should be conducted to ensure that an appropriately sized hammer is specified. Project personnel should be on-site during the pile driving activities to record pile penetration rates, verify pile capacities, and monitor all piles for damage during installation.

Although no obstructions were encountered during the investigation, the design team and Contractor should be aware of the potential for encountering obstructions as piles are driven to the design tip elevation. In general, larger diameter, open-ended piles are less susceptible to being negatively affected by obstructions. Conical tips for closed-ended piles could help push obstructions laterally into the soft subgrade, but may also cause the pile to delfect.

7.6 PARKING LOTS

Paved surfaces for light vehicle traffic should be constructed as follows, at a minimum: 3 inches of asphalt concrete pavement; 4 inches of base course; 35 inches of structural fill. The intent is to create a minimum 42-inch-thick structural section. A thicker section may be warranted if subgrade conditions vary from those described herein. If heavy vehicle loading is anticipated, a specific pavement design should be performed.

Lean clay subgrade will likely be present at the bottom of excavations for parking lots. A separation/reinforcing geotextile should be placed between the subgrade and structural fill to both stabilize the fill and prevent contamination that will reduce performance.

7.7 UTILITIES

Buried utilities, including storm sewer, sanitary sewer and water conduits and structures, should bear on a minimum 6-inch-thick lift of base course or comparable material that will not damage the utility component. The utility should be bedded on all sides with appropriate bedding material, such as base course. Any remaining open excavation for utilities should be backfilled with structural fill to the appropriate grade. Utility trenches in lean clay subgrade should be lined with a separation/reinforcing geotextile, and a 12-inch-thick lift of structural fill should be placed below the base course.

Water and sanitary sewer lines shall have a minimum cover of 6 feet. Lines shall be insulated if the specified minimum cover cannot be achieved.

8. LIMITATIONS and CLOSURE

The information submitted in this report is based on our interpretation of data from a field and lab geotechnical investigation conducted for this project and other sources discussed in this report. Effort was made to obtain information that is representative of the actual conditions at the site. However, actual subsurface conditions will vary and additional information may be discovered that could impact our recommendations. If conditions significantly different from those indicated in this report are encountered by subsequent investigations or during construction, the recommendations of this report should be reviewed by PND.

This report was prepared by PND Engineers, Inc., for use on this project only, and may not be used in any manner that would constitute a detriment to PND. PND is not responsible for conclusions, opinions, or recommendations made by others based on data presented in this report.

Included in Appendix F is a copy of the Geoprofessional Business Association (GBA) publication "Important Information about Your Geotechnical-Engineering Report." The publication is included in this report to help the Owner, Contractor, and others who read this document understand the limitations described above and the additional limitations contained in the publication and made a part of this report. This document should be read carefully. If in the opinion of Contractors bidding this project, sufficient information has not been made available to satisfactorily bid the project then the Contractor should perform additional geotechnical investigations as necessary to satisfy themselves as to site conditions.

STATE OF ALASKA
Sean C. Sjostedt
CE-102428
REGISTERED PROFESSIONAL ENGINEER

Sean Sjostedt, P.E.
Senior Geotechnical Engineer
July 12, 2022

STATE OF ALASKA
Charles R. Somerville
CE-8845
REGISTERED PROFESSIONAL ENGINEER

Dick Somerville, P.E.
Principal
July 12, 2022

9. REFERENCES

ASTM D420. "Standard Guide to Site Characterization for Engineering Design and Construction Purposes."

ASTM D1586. "Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils."

ASTM D2216. "Standard Test Methods for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass."

ASTM D2487. "Classification of Soils for Engineering Purposes (Unified Soil Classification System)."

ASTM D2488. "Description and Identification of Soils (Visual Manual Method)."

ASTM D4318. "Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils."

ASTM D5434. "Standard Guide for Field Logging of Subsurface Explorations of Soil and Rock."

ASTM D6913. "Particle-Size Distribution (Gradation) of Soils Using Sieve Analysis."

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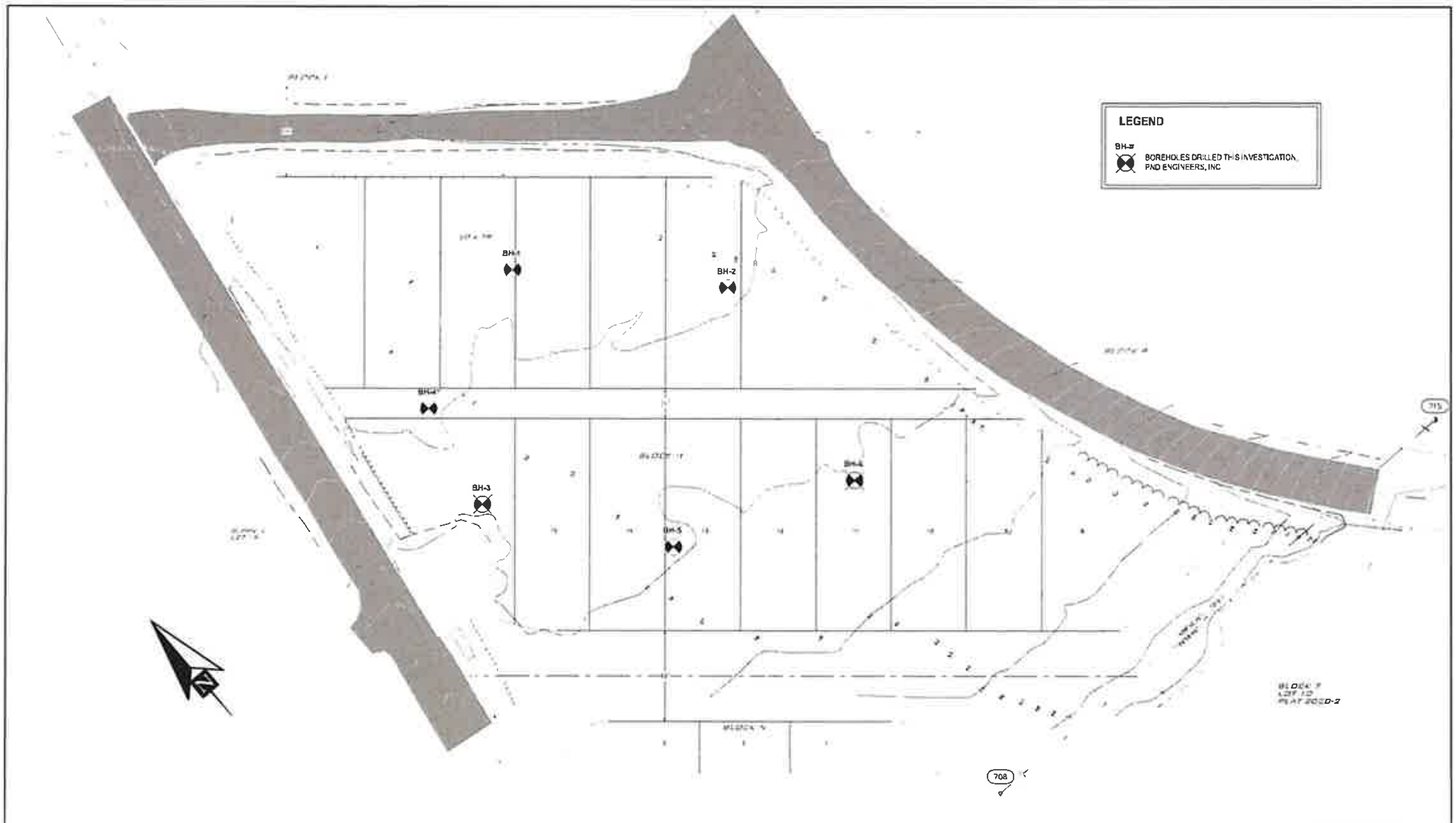
Wesson, R.L., Boyd, O.S., Mueller, C.S., Bufe, C.G., Frankel, A.D., and Petersen, M.D. (2007). "Revision of time-independent probabilistic seismic hazard maps for Alaska." *U.S. Geological Survey Open-File Report*, 2007-1043.

APPENDIX A – BOREHOLE LOGS



PRELIMINARY

PND

HAINES BOROUGH, ALASKA
HAINES PUBLIC SAFETY BUILDING

GEOTECHNICAL SITE PLAN w/
BOREHOLE LOCATIONS

A

DESIGN SCS CHECKED SCS

SCALE IN FEET
0 30 60 FT

DATE MAY 2022

SOILS CLASSIFICATION, CONSISTENCY AND SYMBOLS

CLASSIFICATION

Identification and classification of soil samples is accomplished in general accordance with the ASTM version of the Unified Soil Classification System (USCS) as presented in ASTM Standard D2487. The standard is a qualitative method of classifying soil into the following major divisions (1) coarse grained soil, (2) fine grained soil, and (3) highly organic soils. Classification is performed on a soil sample which passes the 75 mm (3 inch) sieve, oversize material (> 75 mm particles) is noted on the soil logs as well. Classification of oversize material is not always possible because the oversize particles are typically too large to be captured in the sampling equipment. Oversize materials greater than 300 mm (12 inches) are termed boulders, while materials between 75 mm and 300 mm are termed cobbles. Coarse grained soils are described as having 50% or more of the sample retained on the No. 200 sieve (0.075 mm) while fine grained soils will have 50% or more of the sample passing the No. 200 sieve. Coarse samples containing >50% material retained on the No. 4 sieve is classified as gravel. If a majority of the sample is retained on the No. 200 sieve but passes the No. 4 sieve it is classified as a sand. Fine grained soils are those having more than 50% of the sample passing the No. 200 sieve; these may be classified as silt or clay depending their Atterberg limits or observations of field consistency. Refer to the most recent version of ASTM D2487 for a complete discussion of the classification method.

SOIL CONSISTENCY - CRITERIA

Soil consistency as defined below and determined by normal field and laboratory methods applies only to non-frozen material. For these materials, the influence of such factors as soil structure, i.e., Fissure systems, shinkage cracks, slickensides, etc., must be taken into consideration in making any correlation with the consistency values listed below. In permafrost zones, the consistency and strength of frozen soils may vary significantly and unexplainably with ice content, thermal regime and soil type.

STANDARD PENETRATION TEST (BLOWS/FT) RELATIVE TO DENSTIY/CONSISTENCY

N_{60}	Density	Relative Density	N_{60}	Consistency
0-4	Very Loose	0-15%	< 2	Very Soft
4-10	Loose	15-35%	2 - 4	Soft
10-30	Medium Dense	35-65%	4 - 8	Medium Stiff
30-50	Dense	65-85%	8 - 15	Stiff
> 50	Very Dense	>85%	15 - 30	Very Stiff
			> 30	Hard

UNDRAINED SHEAR STRENGTH

psf
< 250
250 - 500
500 - 1000
1000 - 2000
2000 - 4000
> 4000

(*correlations based upon standard 1.4" O.D. split spoon and 140 lb manual hammer dropped from a height of 30 inches)

(*Adjust as required for other sampler types)

Ref: Terzaghi and Peck, Soil Mechanics in Engineering Practice, 3rd Edition, pg 60-63
ASTM D1586 Standard Test Method for Penetration Test and Split-Barrel Sampling of Soils
ASTM D2487 Standard Practice for Classification of Soils for Engineering Purposes (USCS)

LIST OF ABBREVIATIONS

Drill Methods:

AR	Air Rotary
CC60	Continuous Coring (RS-60)
CD	Case and Drill
CCm	Continuous Coring (Macro Core)
CME	Continuous Augering
CWR	Casing with Wash Rotary
DH	Down-hole hammer
HSA	Hollow Stem Auger
MR	Mud Rotary
NQ3	NQ3 Triple Tube
MC7	MC7 Coring
WR	Wash Rotary
TP	Test Pit
DP	Direct Push
SC	Sonic Core
SCC	Sonic Core with Wash Rotary
SSA	Solid Stem Auger

Sample Methods:

AR	Air Rotary
Cc	Continuous Core
GR	Grab Sample
Sh	Oversize Split-Spoon
Ss	Standard Split-Spoon
ST	Shelby Tube
Cs	Core Sample
SC	Sonic Core

Color:

BK	Black
BN	Brown
DG	Dark Brown
DG	Dark Gray
G	Gray
GG	Greenish Gray
LB	Light Brown
LG	Light Gray
OG	Olive Gray
P	Pink
R	Reddish
RO	Rusty Orange
TN	Tan
YO	Yellowish Orange
BG	Brownish Gray

Particle Angularity

A	Angular
R	Rounded
SA	Sub-Angular
SR	Sub-Rounded

Particle Shape:

E	Elongated
F	Flat

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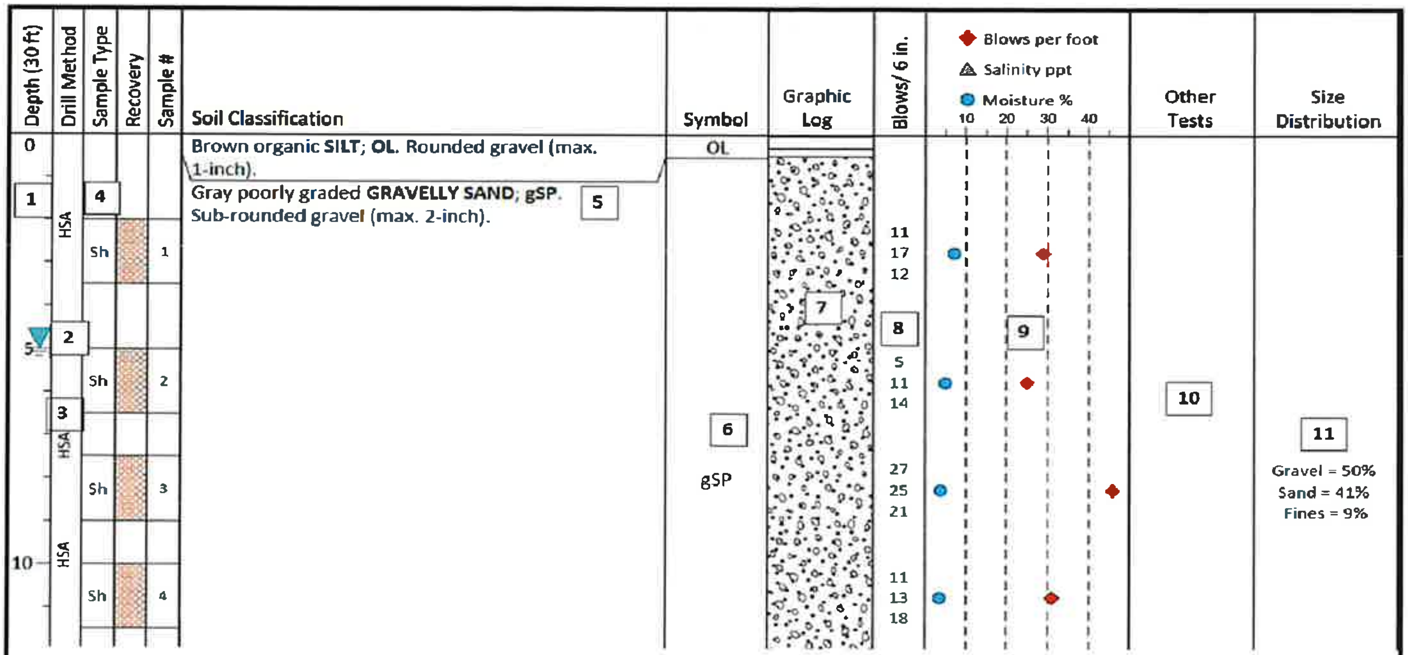
STANDARD
GEOTECHNICAL EXPLORATION
BOREHOLE LEGEND



1	Depth	Depth (in feet) below the mudline.
2	Water Table	Water Table depth.
3	Drill Method	Drilling methods recorded in depth intervals. Symbols explained on Fig. B-01.
4	Sample Type	Type of soil sample collected at depth interval depicted; symbols explained on Fig. B-04.
5	Material Description - General	General description of soil encountered in a lithological layer.
6	Symbol	Group symbols for soil and ice classification for each specimen.
7	Graphic Log	Graphic depiction of lithological layers encountered. Graphic symbols explained in Fig. B-03 and B-04.
8	Blows/ 6 in	Number of blows to advance sampler each 6-inch interval using sampler type specified with a 30-inch drop.
9	Graphs	Graphic log depicting moisture content and blow count per foot of soil specimens.
10	Other Tests	Results of other tests including hydrometer, Atterberg limits, etc.
11	Size Distribution	Results of particle size distribution analysis.

GENERAL NOTES

1. Field descriptions may have been modified to reflect laboratory test results.
2. Descriptions on these boring logs apply only at the specific locations at the time the borings were drilled. They are not warranted to be representative of subsurface conditions at other locations or times.
3. Split spoon blow counts shown are uncorrected raw data. Various hammer sizes and split spoon sizes were used and have not been corrected to a Standard Penetration Test (SPT). Blow counts may vary substantially between SPT and these methods.

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BOREHOLE LEGEND

SOIL LEGEND - (1 of 3)

▨	**GW**	*WELL GRADED GRAVEL*
▨	**GWs**	*WELL GRADED GRAVEL WITH SAND*
▨	**sGW**	*WELL GRADED SANDY GRAVEL*
▨	**GP**	*POORLY GRADED GRAVEL*
▨	**GPs**	*POORLY GRADED GRAVEL WITH SAND*
▨	**sGP**	*POORLY GRADED SANDY GRAVEL*
▨	**GW-GM**	*WELL GRADED GRAVEL WITH SILT*
▨	**(GW-GM)s**	*WELL GRADED GRAVEL WITH SILT AND SAND*
▨	**s(GW-GM)**	*WELL GRADED SANDY GRAVEL WITH SILT*
▨	**GW-GC**	*WELL GRADED GRAVEL WITH CLAY*
▨	**(GW-GC)s**	*WELL GRADED GRAVEL WITH CLAY AND SAND*
▨	**s(GW-GC)**	*WELL GRADED SANDY GRAVEL WITH CLAY*
▨	**GP-GM**	*POORLY GRADED GRAVEL WITH SILT*
▨	**(GP-GM)s**	*POORLY GRADED GRAVEL WITH SILT AND SAND*
▨	**s(GP-GM)**	*POORLY GRADED SANDY GRAVEL WITH SILT*
▨	**GP-GC**	*POORLY GRADED GRAVEL WITH CLAY*
▨	**(GP-GC)s**	*POORLY GRADED GRAVEL WITH CLAY AND SAND*
▨	**GM**	*SILTY GRAVEL*
▨	**GMs**	*SILTY GRAVEL WITH SAND*
▨	**sGM**	*SILTY, SANDY GRAVEL*
▨	**GC**	*CLAYEY GRAVEL*
▨	**GCs**	*CLAYEY GRAVEL WITH SAND*
▨	**sGC**	*CLAYEY, SANDY GRAVEL*
▨	**GC-GM**	*SILTY, CLAYEY GRAVEL*
▨	**(GC-GM)s**	*SILTY, CLAYEY GRAVEL WITH SAND*
▨	**s(GC-GM)**	*SILTY, CLAYEY, SANDY GRAVEL*
□	**SW**	*WELL GRADED SAND*
▨	**SWg**	*WELL GRADED SAND WITH GRAVEL*
▨	**gSW**	*WELL GRADED GRAVELLY SAND*
□	**SP**	*POORLY GRADED SAND*

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STANDARD
GEOTECHNICAL EXPLORATION
BOREHOLE LEGEND

SOIL LEGEND - (2 of 3)

- ▨ **SPg** *POORLY GRADED SAND WITH GRAVEL*
- ▨ **gSP** *POORLY GRADED GRAVELLY SAND*
- ▨ **SW-SM** *WELL GRADED SAND WITH SILT*
- ▨ **(SW-SM)g** *WELL GRADED SAND WITH SILT AND GRAVEL*
- ▨ **g(SW-SM)** *WELL GRADED GRAVELLY SAND WITH SILT*
- ▨ **SW-SC** *WELL GRADED SAND WITH CLAY*
- ▨ **(SW-SC)g** *WELL GRADED SAND WITH CLAY AND GRAVEL*
- ▨ **g(SW-SC)** *WELL GRADED GRAVELLY SAND WITH CLAY*
- ▨ **SP-SM** *POORLY GRADED SAND WITH SILT*
- ▨ **(SP-SM)g** *POORLY GRADED SAND WITH SILT AND GRAVEL*
- ▨ **g(SP-SM)** *POORLY GRADED GRAVELLY SAND WITH SILT*
- ▨ **SP-SC** *POORLY GRADED SAND WITH CLAY*
- ▨ **(SP-SC)g** *POORLY GRADED SAND WITH CLAY AND GRAVEL*
- ▨ **g(SP-SC)** *POORLY GRADED GRAVELLY SAND WITH CLAY*
- ▨ **SM** *SILTY SAND*
- ▨ **SMg** *SILTY SAND WITH GRAVEL*
- ▨ **gSM** *SILTY, GRAVELLY SAND*
- ▨ **SC** *CLAYEY SAND*
- ▨ **SCg** *CLAYEY SAND WITH GRAVEL*
- ▨ **gSC** *CLAYEY, GRAVELLY SAND*
- ▨ **SC-SM** *SILTY, CLAYEY SAND*
- ▨ **(SC-SM)g** *SILTY, CLAYEY SAND WITH GRAVEL*
- ▨ **g(SP-SC)** *POORLY GRADED GRAVELLY SAND WITH CLAY*
- ▨ **CL** *CLAY*
- ▨ **CLs** *CLAY WITH SAND*
- ▨ **CLg** *CLAY WITH GRAVEL*
- ▨ **sCL** *SANDY LEAN CLAY*
- ▨ **gCL** *GRAVELLY LEAN CLAY*
- ▨ **sCLg** *SANDY LEAN CLAY WITH GRAVEL*
- ▨ **gCLs** *GRAVELLY LEAN CLAY WITH SAND*

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STANDARD
GEOTECHNICAL EXPLORATION
BOREHOLE LEGEND

SOIL LEGEND - (3 of 3)

▨ **ML**
SILT

▨ **MLs**
SILT WITH SAND

▨ **MLg**
LEAN SILT WITH GRAVEL

▨ **sML**
SANDY SILT

▨ **gML**
GRAVELLY SILT

▨ **sMLg**
SANDY SILT WITH GRAVEL

▨ **gMLs**
GRAVELLY SILT WITH SAND

▨ **CH**
FAT CLAY

▨ **CHs**
FAT CLAY WITH SAND

▨ **CHg**
FAT CLAY WITH GRAVEL

▨ **sCH**
SANDY FAT CLAY

▨ **gCH**
GRAVELLY FAT CLAY

▨ **gCHs**
GRAVELLY FAT CLAY WITH SAND

▨ **MH**
ELASTIC SILT

▨ **MHs**
ELASTIC SILT WITH SAND

▨ **MHg**
ELASTIC SILT WITH GRAVEL

▨ **sMH**
SANDY ELASTIC SILT

▨ **gMH**
GRAVELLY ELASTIC SILT

▨ **sMHg**
SANDY ELASTIC SILT WITH GRAVEL

▨ **gMHs**
GRAVELLY ELASTIC SILT WITH SAND

▨ **OH**
ORGANIC SILT

▨ **OL**
ORGANIC CLAY

▨ **ICE**
ICE

▨ **ICE+S**
ICE WITH SOIL

▨ **PT**
PEAT

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Date: July 2022

STANDARD
GEOTECHNICAL EXPLORATION
BOREHOLE LEGEND

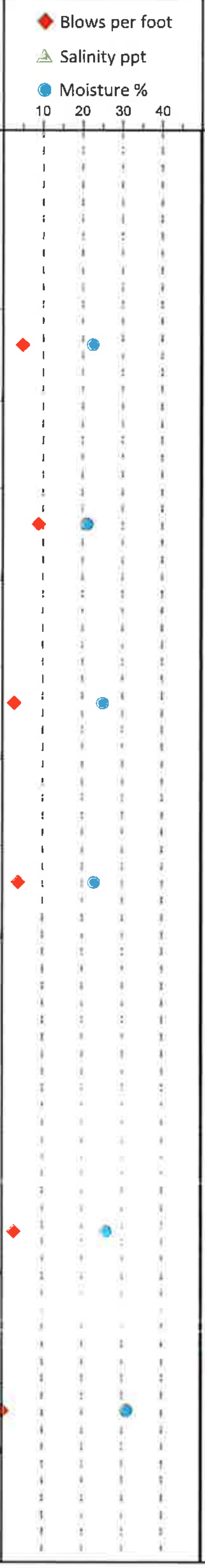
LOG OF BOREHOLE BH-1

Project: Haines Community Safety and Training Center
Project Number: 202103
Project Phase: 4

Elevation: 48.6 ft MLLW
Horizontal Datum: NAD83 AKSP Zone 1
Latitude: 59.23050 Longitude: 135.44778

Logged By: PJD
Reviewed By: SS
Review Date: 7/8/2022

Depth (101.5 ft)	Drill Method	Sample Type	Recovery	Sample #	Soil Classification	Symbol	Blows/ 6 in.	Other Tests	Size Distribution
0					Brown **PEAT; PT**. Organic mat.	PT			
	WR				Light gray lean **SILTY CLAY; CL-ML**. Average standard shear vane measurement: 0.7 tsf.	CL-ML			
5		Sh	▨	1			1 2 3 6		
	WR								
10	WR	Sh	▨	2			2 4 5 6		
15	WR	Sh	▨	3			1 1 2 3	LL = 26 PL = 20	
	WR								
20		Sh	▨	4	Light gray lean **CLAY; CL**. Average standard shear vane measurement: 0.7 tsf.	CL	2 2 2 4		
	WR								
25		ST	▨	5					
	WR								
30	WR	Sh	▨	6			1 1 2		
35	WR	Sh	▨	7			0 0 0		
40	WR								



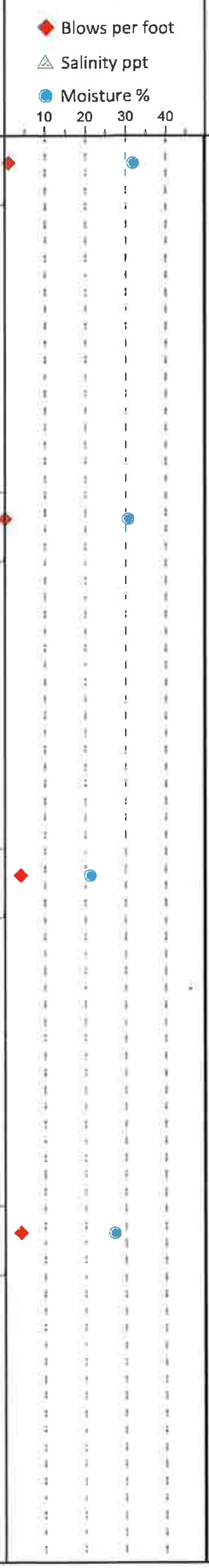
Borehole terminated at 101.5 ft
Client: Bettisworth North Architects and Planners Inc.
Drill Start: 5/6/2022

Drilling Contractor: Discovery Drilling, Inc.
Drill Equipment: Geoprobe 6712DT
Driller: BV

LOG OF BOREHOLE BH-1

Project: Haines Community Safety and Training Center Project Number: 202103 Project Phase: 4	Elevation: 48.6 ft MLLW Horizontal Datum: NAD83 AKSP Zone 1 Latitude: 59.23050 Longitude: 135.44778	Logged By: PJD Reviewed By: SS Review Date: 7/8/2022

Depth (101.5 ft)	Drill Method	Sample Type	Recovery	Sample #	Soil Classification	Symbol	Graphic Log	Blows/ 6 in.	Other Tests	Size Distribution
40	WR	Sh	▨	8	Light gray lean **CLAY; CL**. Sub-rounded gravel (max. 1-inch). Average standard shear vane measurement: 0.1 tsf.	CL	▨	0 0 1		
45	WR									
50	WR	Sh	▨	9				1 0 0		
55	WR									
60	WR	Sh	▨	10				5 2 2		
65	WR									
70	WR	Sh	▨	11				0 2 2		
75	WR									
	WR				Gray poorly graded **SAND; SP**.	SP	⁙			
80	WR									



Borehole terminated at 101.5 ft
Client: Bettisworth North Architects and Planners Inc.
Drill Start: 5/6/2022

Drilling Contractor: Discovery Drilling, Inc.
Drill Equipment: Geoprobe 6712DT
Driller: BV

LOG OF BOREHOLE BH-1

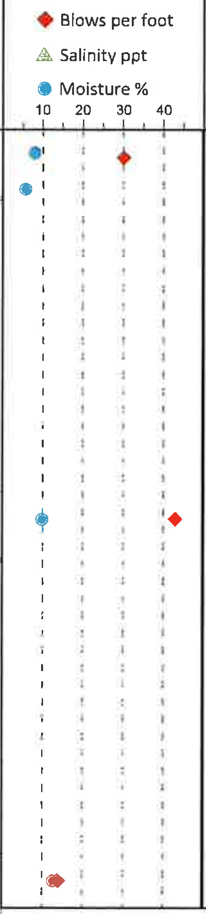
Project: Haines Community Safety and Training Center
Project Number: 202103
Project Phase: 4

Elevation: 48.6 ft MLLW
Horizontal Datum: NAD83 AKSP Zone 1
Latitude: 59.23050 Longitude: 135.44778

Logged By: PJD
Reviewed By: SS
Review Date: 7/8/2022

Depth (101.5 ft)	Drill Method	Sample Type	Recovery	Sample #	Soil Classification	Symbol	Blows/ 6 in.	Other Tests	Size Distribution
80	WR	Sh	▨	12	Gray poorly graded **SAND; SP**. Free water encountered at 81.5'.	SP	18 14 16		
		Sh		12b	Gray **CLAYEY GRAVEL with SAND; GCs**. Sub-rounded gravel (max. 1.5-inch).	GCs			
85	WR				Gray **SANDY** lean **CLAY with GRAVEL; sCLg.** Sub-rounded gravel (max. 3/4-inch). Lenses of sand and clay from 101' to 101.5'.	sCLg			
90	WR	Sh	▨	13			10 20 23		
95	WR				Gray **CLAYEY SAND with GRAVEL; SCg**. Sub-rounded gravel (max. 3/4-inch).	SCg			
100	WR	Sh	▨	14			8 10 4		Gravel = 27% Sand = 59% Fines = 14%

Graphic Log



Borehole terminated at 101.5 ft
Client: Bettisworth North Architects and Planners Inc.
Drill Start: 5/6/2022

Drilling Contractor: Discovery Drilling, Inc.
Drill Equipment: Geoprobe 6712DT
Driller: BV

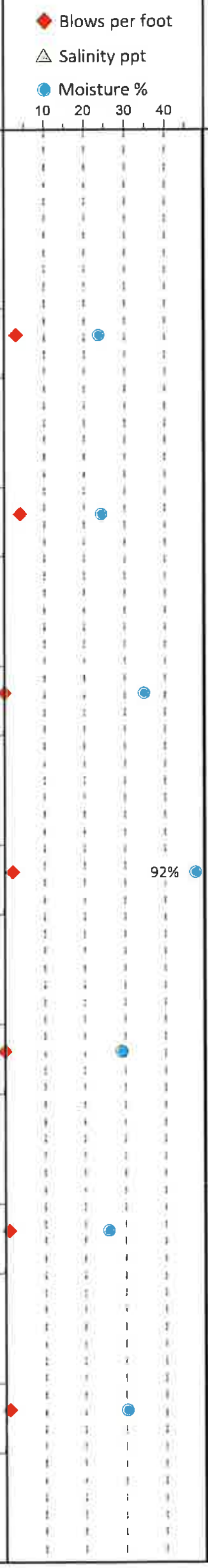
LOG OF BOREHOLE BH-2

Project: Haines Community Safety and Training Center
Project Number: 202103
Project Phase: 4

Elevation: 49.1 ft MLLW
Horizontal Datum: NAD83 AKSP Zone 1
Latitude: 59.23023 Longitude: 135.44723

Logged By: PJD
Reviewed By: SS
Review Date: 7/8/2022

Depth (101.5 ft)	Drill Method	Sample Type	Recovery	Sample #	Soil Classification	Symbol	Blows/ 6 in.	Other Tests	Size Distribution
0					Brown **PEAT**; **PT**. Organic mat.	PT			
	WR				Light gray lean **SILTY CLAY**; **CL-ML**. Average standard shear vane measurement: 0.9 tsf.	CL-ML			
5	WR	Sh	▨	1			1 1 2		
10	WR	Sh	▨	2			1 2 2		
15	WR	Sh	▨	3			1 0 0		
20	WR	Sh	▨	4	Light gray lean **CLAY**; **CL**. Sub-rounded/sub-angular gravel (max. 1-inch). 4" brown organics at 20'. Average standard shear vane measurement: 0.2 tsf.	CL	1 1 1		
25	WR	Sh	▨	5			0 0 0		
30	WR	Sh	▨	6			0 0 1		
35	WR	Sh	▨	7			0 0 1	LL = 27 PL = 17	
40	WR								



Borehole terminated at 101.5 ft
Client: Bettisworth North Architects and Planners Inc.
Drill Start: 5/7/2022

Drilling Contractor: Discovery Drilling, Inc.
Drill Equipment: Geoprobe 6712DT
Driller: BV

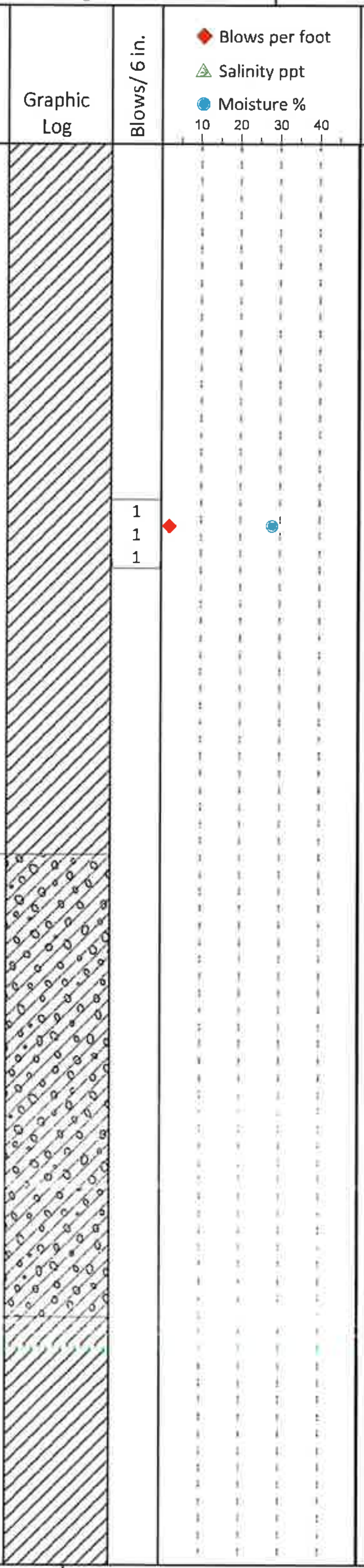
LOG OF BOREHOLE BH-2

Project: Haines Community Safety and Training Center
Project Number: 202103
Project Phase: 4

Elevation: 49.1 ft MLLW
Horizontal Datum: NAD83 AKSP Zone 1
Latitude: 59.23023 Longitude: 135.44723

Logged By: PJD
Reviewed By: SS
Review Date: 7/8/2022

Depth (101.5 ft)	Drill Method	Sample Type	Recovery	Sample #	Soil Classification	Symbol	Blows/ 6 in.	Other Tests	Size Distribution
40		ST	▨	8	Light gray lean **CLAY**; **CL**. Sub-rounded/sub-angular gravel (max. 1-inch). Average standard shear vane measurement: 0.2 tsf.	CL			
	WR								
45	WR								
50	WR	Sh	▨	9			1 1 1		
55	WR								
60	WR				Light gray lean **CLAY with GRAVEL**; **CLg**. Sub-rounded/sub-angular gravel (max. 1-inch). Trace sand from 60' to 62' bgs.	CLg			
65	WR								
	WR								
70	WR	ST	▨	11					
					Light gray lean **CLAY**, **CL**. Lenses of SP between 80' and 82' bgs.	CL			
75	WR								
80	WR								



Borehole terminated at 101.5 ft
Client: Bettisworth North Architects and Planners Inc.
Drill Start: 5/7/2022

Drilling Contractor: Discovery Drilling, Inc.
Drill Equipment: Geoprobe 6712DT
Driller: BV

LOG OF BOREHOLE BH-2

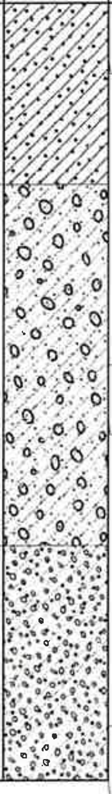
Project: Haines Community Safety and Training Center
Project Number: 202103
Project Phase: 4

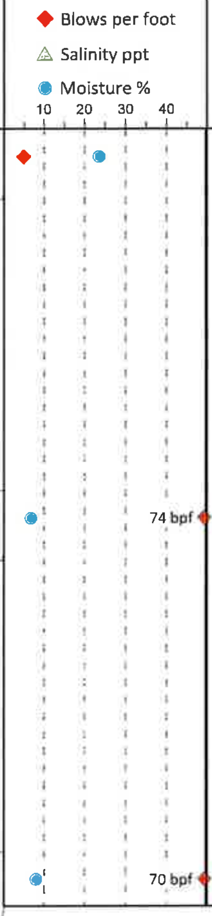
Elevation: 49.1 ft MLLW
Horizontal Datum: NAD83 AKSP Zone 1
Latitude: 59.23023 Longitude: 135.44723

Logged By: PJD
Reviewed By: SS
Review Date: 7/8/2022

Depth (101.5 ft)	Drill Method	Sample Type	Recovery	Sample #	Soil Classification	Symbol	Blows/ 6 in.	Other Tests	Size Distribution
80	WR	Sh		12	Light gray **SANDY** lean **CLAY; sCL.**	sCL	1 1 4		
85	WR				Light gray **CLAYEY GRAVEL with SAND; GCs**. Sub-rounded/sub-angular gravel (max. 1.5-inch).	GCs			
90	WR	Sh		13			24 48 26		
95	WR				Light gray **CLAYEY, GRAVELLY SAND; gSC**. Sub-rounded gravel (max. 3/4-inch).	gSC			
100	WR	Sh		14			16 32 38		

Graphic Log





Borehole terminated at 101.5 ft
Client: Bettisworth North Architects and Planners Inc.
Drill Start: 5/7/2022

Drilling Contractor: Discovery Drilling, Inc.
Drill Equipment: Geoprobe 6712DT
Driller: BV

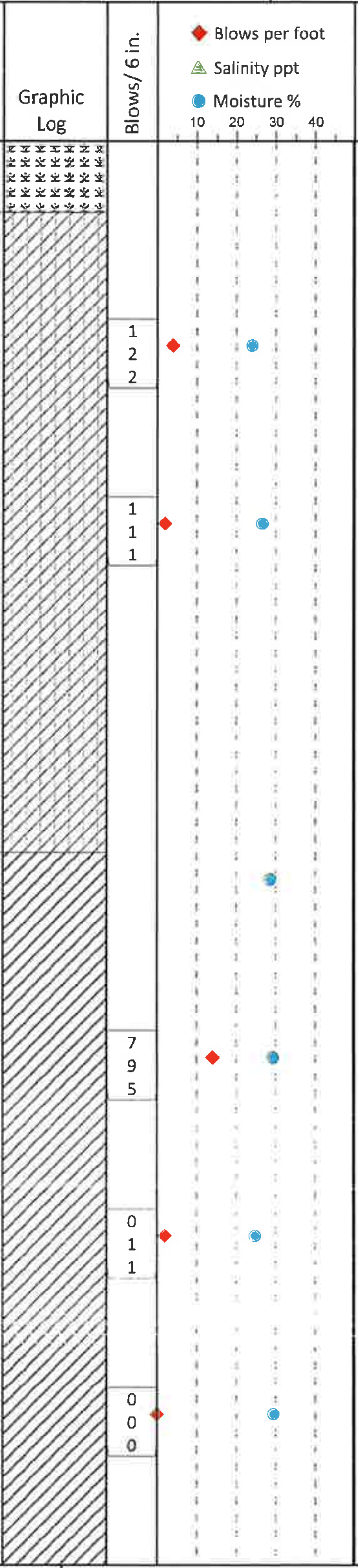
LOG OF BOREHOLE BH-3

Project: Haines Community Safety and Training Center
Project Number: 202103
Project Phase: 4

Elevation: 53.4 ft MLLW
Horizontal Datum: NAD83 AKSP Zone 1
Latitude: 59.23020 Longitude: 135.44838

Logged By: PJD
Reviewed By: SS
Review Date: 7/8/2022

Depth (91.5 ft)	Drill Method	Sample Type	Recovery	Sample #	Soil Classification	Symbol	Blows/ 6 in.	Other Tests	Size Distribution
0					Brown **PEAT**; **PT**. Organic mat.	PT			
	WR				Light gray lean **SILTY CLAY**; **CL-ML**. Average standard shear vane measurement: 0.6 tsf.				
5		Sh	▨	1			1 2 2		
	WR								
10		Sh	▨	2		CL-ML	1 1 1		
	WR								
15	WR	ST	▨	3					
	WR								
20		Sh	▨	4	Light gray lean **CLAY**; **CL**. Sub-rounded/sub-angular gravel (max. 2-inch). Standard shear vane measurements at 25' and 30': 0.5 tsf.				
	WR								
25		Sh	▨	5			7 9 5		
	WR								
30		Sh	▨	6		CL	0 1 1		
	WR								
35	WR	Sh	▨	7			0 0 0		
40	WR								



Borehole terminated at 91.5 ft
Client: Bettisworth North Architects and Planners Inc.
Drill Start: 5/8/2022

Drilling Contractor: Discovery Drilling, Inc.
Drill Equipment: Geoprobe 6712DT
Driller: BV

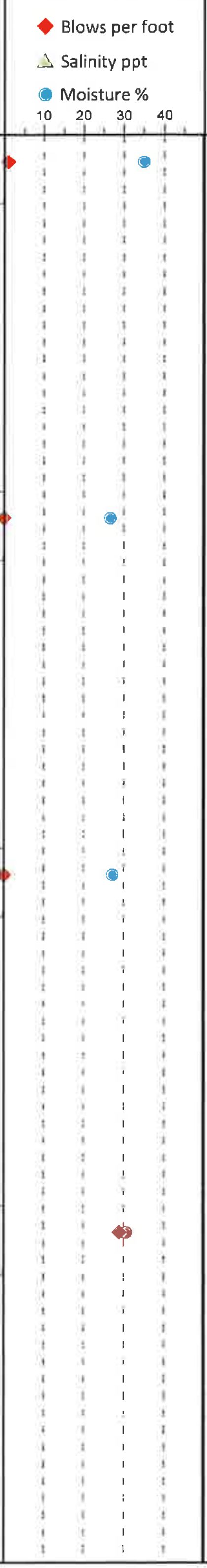
LOG OF BOREHOLE BH-3

Project: Haines Community Safety and Training Center
Project Number: 202103
Project Phase: 4

Elevation: 53.4 ft MLLW
Horizontal Datum: NAD83 AKSP Zone 1
Latitude: 59.23020 Longitude: 135.44838

Logged By: PJD
Reviewed By: SS
Review Date: 7/8/2022

Depth (91.5 ft)	Drill Method	Sample Type	Recovery	Sample #	Soil Classification	Symbol	Graphic Log	Blows/ 6 in.	Other Tests	Size Distribution
40	WR	Sh	▨	8	Light gray lean **CLAY**; **CL**. Standard shear vane measurement: 0.4 tsf.	CL	▨	0 0 1		
45	WR									
50	WR	Sh	▨	9				0 0 0	LL = 30 PL = 21	
55	WR									
60	WR	Sh	▨	10				0 0 0		
65	WR									
	WR				Light gray **CLAYEY GRAVEL with SAND**; **GCs**. Sub-rounded gravel (max. 1-inch).	GCs				
70	WR	Sh	▨	11				5 14 15		
75	WR									
					Light gray **SANDY** lean **CLAY**; **sCL**. Sub-rounded gravel (max. 1/2-inch).	sCL				
80	WR									



Borehole terminated at 91.5 ft
Client: Bettisworth North Architects and Planners Inc.
Drill Start: 5/8/2022

Drilling Contractor: Discovery Drilling, Inc.
Drill Equipment: Geoprobe 6712DT
Driller: BV

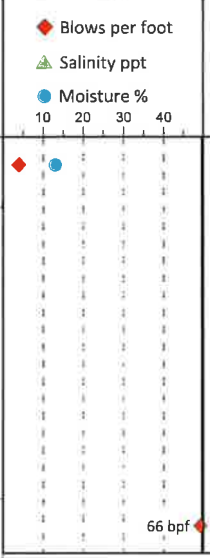
LOG OF BOREHOLE BH-3

Project: Haines Community Safety and Training Center
Project Number: 202103
Project Phase: 4

Elevation: 53.4 ft MLLW
Horizontal Datum: NAD83 AKSP Zone 1
Latitude: 59.23020 Longitude: 135.44838

Logged By: PJD
Reviewed By: SS
Review Date: 7/8/2022

Depth (91.5 ft)	Drill Method	Sample Type	Recovery	Sample #	Soil Classification	Symbol	Graphic Log	Blows/ 6 in.	Other Tests	Size Distribution
80		Sh	▨	12	Light gray **SANDY** lean **CLAY**; **sCL**. Sub-rounded gravel (max. 1/2-inch).	sCL	▨	2 2 2		
	WR									
85										
	WR									
90								5 18 48		



Borehole terminated at 91.5 ft
Client: Bettisworth North Architects and Planners Inc.
Drill Start: 5/8/2022

Drilling Contractor: Discovery Drilling, Inc.
Drill Equipment: Geoprobe 6712DT
Driller: BV

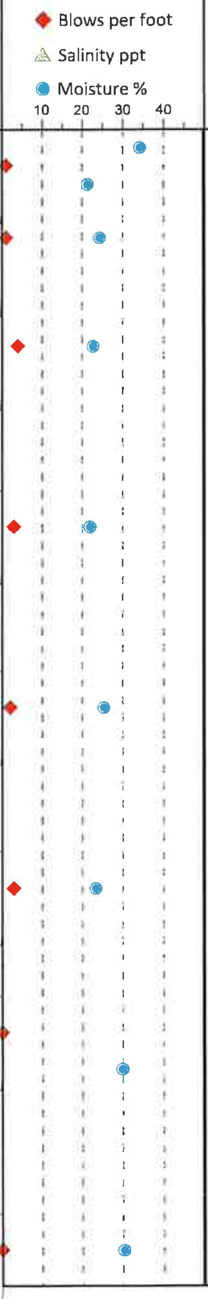
LOG OF BOREHOLE BH-4

Project: Haines Community Safety and Training Center
Project Number: 202103
Project Phase: 4

Elevation: 49.3 ft MLLW
Horizontal Datum: NAD83 AKSP Zone 1
Latitude: 59.23040 Longitude: 135.44832

Logged By: PJD
Reviewed By: SS
Review Date: 7/8/2022

Depth (32 ft)	Drill Method	Sample Type	Recovery	Sample #	Soil Classification	Symbol	Graphic Log	Blows/ 6 in.	Other Tests	Size Distribution
0	WR	Sh		1a	Brown organic **SILT**; **OL**. Organic mat.	OL		1 1 0 2		
		Sh		1b	Light gray lean **SILTY CLAY**; **CL-ML**. Average standard shear vane measurement: 0.7 tsf. Trace fibrous organics in upper 5'.	CL-ML				
	WR	Sh		2				1 0 1 2		
5	WR	Sh		3				0 2 2 4		
10	WR	Sh		4				1 1 2 3		
15	WR	Sh		5				1 0 2 2		
20	WR	Sh		6	Light gray lean **CLAY**; **CL**. Average standard shear vane measurement: 0.3 tsf.	CL		0 1 2 2		
	WR							0 0 0 1		
25	WR	Sh		7						
30	WR	Sh		8				0 0 0 1		



Borehole terminated at 32 ft
Client: Bettisworth North Architects and Planners Inc.
Drill Start: 5/10/2022

Drilling Contractor: Discovery Drilling, Inc.
Drill Equipment: Geoprobe 6712DT
Driller: BV

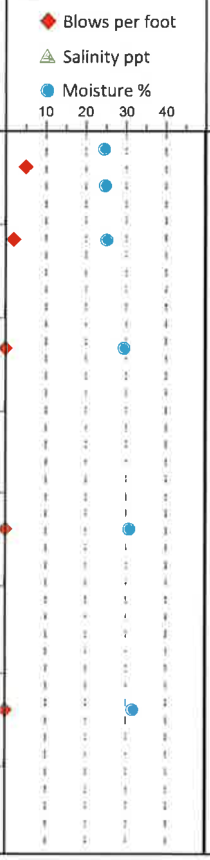
LOG OF BOREHOLE BH-5

Project: Haines Community Safety and Training Center
Project Number: 202103
Project Phase: 4

Elevation: 54.5 ft MLLW
Horizontal Datum: NAD83 AKSP Zone 1
Latitude: 59.22985 Longitude: 135.44773

Logged By: PJD
Reviewed By: SS
Review Date: 7/8/2022

Depth (20 ft)	Drill Method	Sample Type	Recovery	Sample #	Soil Classification	Symbol	Blows/ 6 in.	Other Tests	Size Distribution
0		Sh	▨	1a	Dark brown **SILTY GRAVEL**; **GM**. Sub-rounded gravel (max. 1-inch). 35% fibrous organics by volume.	GM	0 2 3 0		
		Sh	▨	1b	Light gray lean **SILTY CLAY**; **CL-ML**. Average standard shear vane measurement: 0.3 tsf.	CL-ML			
	HSA	Sh	▨	2			1 0 2 1		
5	HSA	Sh	▨	3			0 0 0 0		
10	HSA	Sh	▨	4			0 0 0 0		
	HSA								
15	HSA	Sh	▨	5			0 0 0 1		
20									



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Borehole terminated at 20 ft
Client: Bettisworth North Architects and Planners Inc.
Drill Start: 5/11/2022

Drilling Contractor: Discovery Drilling, Inc.
Drill Equipment: Geoprobe 6712DT
Driller: BV

Page: 1 of 1

LOG OF BOREHOLE BH-6

Project: Haines Community Safety and Training Center
Project Number: 202103
Project Phase: 4

Elevation: 55.5 ft MLLW
Horizontal Datum: NAD83 AKSP Zone 1
Latitude: 59.22982 Longitude: 135.44703

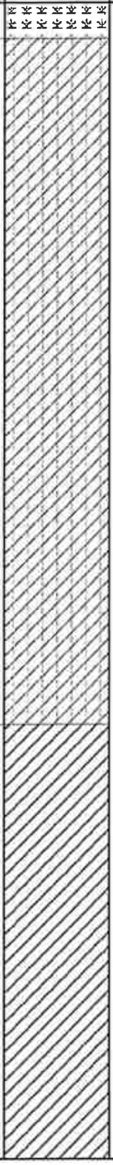
Logged By: PJD
Reviewed By: SS
Review Date: 7/8/2022

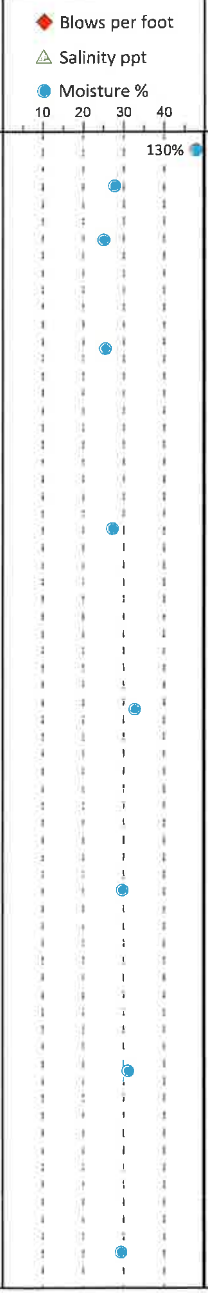
Drill Method	Sample Type	Sample #	Soil Classification	Symbol
	Sh	1a	Dark brown **PEAT**; **PT**. Organic mat.	PT
HSA	Sh	1b	Light gray lean **SILTY CLAY**; **CL-ML**. Sub-rounded gravel (max. 3/8-inch). Average standard shear vane measurement: 0.7 tsf.	CL-ML
	Sh	2		
HSA	Sh	3		
HSA	Sh	4		
HSA	Sh	5		
HSA	Sh	6	lean **CLAY**; **CL**. Average standard shear vane measurement: 0.3 tsf.	CL
HSA	Sh	7		
HSA	Sh	8		

Depth (32 ft): 0, 5, 10, 15, 20, 25, 30

Blows/ 6 in.

Graphic Log





Other Tests

Size Distribution

Borehole terminated at 32 ft
Client: Bettisworth North Architects and Planners Inc.
Drill Start: 5/11/2022

Drilling Contractor: Discovery Drilling, Inc.
Drill Equipment: Geoprobe 6712DT
Driller: BV

APPENDIX B – LABORATORY TEST RESULTS

Summary of Sample Characteristics

Client: Bettisworth North Architects and Planners Inc.
Project: Haines Community Safety and Training Center
Project #: 202103

PND ENGINEERS, INC.

Borehole	Sample #	From	To	Sample Method	Liquid Limit (%)	Plastic Limit (%)	Gradation (%)			Max Particle Size (in)	Laboratory Classification*	Salinity (ppt)	Moisture (%)	Particle Shape	Angularity	Other Tests**
							Gravel	Sand	Fines*							
BH-1	1	5	7	Sh							CL-ML		23			
BH-1	2	10	12	Sh							CL-ML		21			
BH-1	3	15	17	Sh	26	20					CL-ML		25			
BH-1	4	20	22	Sh							CL		23			
BH-1	5	25	27	ST												
BH-1	6	30	31.5	Sh							CL		26			
BH-1	7	35	36.5	Sh							CL		31			
BH-1	8	40	41.5	Sh							CL		32			
BH-1	9	50	51.5	Sh						3/4	CL		31		SR	
BH-1	10	60	61.5	Sh						1	CL		21		SR	
BH-1	11	70	71.5	Sh							CL		27			
BH-1	12	80	81.3	Sh							SP		8			
BH-1	12b	81.3	82	Sh						1.5	GCs		6		SR	
BH-1	13	90	91.5	Sh						1/2	sCLg		10		SR	
BH-1	14	100	102	Sh			26.9	59.1	13.9	1	SCg		13		R	
BH-2	1	5	6.5	Sh							CL-ML		24			
BH-2	2	10	11.5	Sh							CL-ML		24			
BH-2	3	15	16.5	Sh							CL-ML		35			
BH-2	4	20	20.3	Sh							OL		92			
BH-2	5	25	26.5	Sh						3/8	CL		29		SR	
BH-2	6	30	31.5	Sh							CL		26			
BH-2	7	35	36.5	Sh	27	17					CL		30			
BH-2	8	40	42	ST												
BH-2	9	50	51.5	Sh							CL		28			
BH-2	11	70	71.5	ST												
BH-2	12	80	81.5	Sh							sCL		23			
BH-2	13	90	91.5	Sh						1.5	GCs		7		SR	
BH-2	14	100	102	Sh						3/4	gSC		8		SR	
BH-3	1	5	6.5	Sh							CL-ML		24			

*Fines type and content estimated with ASTM D2488 when ASTM D422 or D4318 were not performed

**Other tests: DEN = Bulk Density, SPG = Specific Gravity, HYD = Hydrometer, CONSL = Consolidation, UCS = Unconfined Compression Strength, TRIAX = Triaxial

Summary of Sample Characteristics

Client: Bettisworth North Architects and Planners Inc.
Project: Haines Community Safety and Training Center
Project #: 202103

PND ENGINEERS, INC.

Borehole	Sample #	From	To	Sample Method	Liquid Limit (%)	Plastic Limit (%)	Gradation (%)			Max Particle Size (in)	Laboratory Classification*	Salinity (ppt)	Moisture (%)	Particle Shape	Angularity	Other Tests**
							Gravel	Sand	Fines*							
BH-3	2	10	11.5	Sh							CL-ML		26			
BH-3	3	15	17	ST												
BH-3	4	20	21.5	Sh							CL		28			
BH-3	5	25	26.5	Sh							CL		29			
BH-3	6	30	31.5	Sh						3/8	CL		25		SR	
BH-3	7	35	36.5	Sh							CL		29			
BH-3	8	40	41.5	Sh							CL		35			
BH-3	9	50	51.5	Sh	30	21					CL		27			
BH-3	10	60	61.5	Sh							CL		27			
BH-3	11	70	71.5	Sh						1	GCs		31		SR	
BH-3	12	80	81.5	Sh						1/2	sCL		13		SR	
BH-4	1a	0	1	Sh							OL		34			
BH-4	1b	1	2	Sh							CL-ML		21			
BH-4	2	2	4	Sh							CL-ML		24			
BH-4	3	5	7	Sh							CL-ML		23			
BH-4	4	10	12	Sh							CL-ML		22			
BH-4	5	15	17	Sh							CL-ML		25			
BH-4	6	20	22	Sh							CL		23			
BH-4	7	25	27	Sh							CL		30			
BH-4	8	30	32	Sh							CL		30			
BH-5	1a	0	1	Sh						1	GM		24		SR	
BH-5	1b	1	2	Sh							CL-ML		25			
BH-5	2	2	4	Sh							CL-ML		25			
BH-5	3	5	7	Sh							CL-ML		29			
BH-5	4	10	12	Sh							CL-ML		31			
BH-5	5	15	17	Sh							CL-ML		32			
BH-6	1a	0	1	Sh							PT		130			
BH-6	1b	1	2	Sh							CL-ML		28			
BH-6	2	2	4	Sh							CL-ML		25			

*Fines type and content estimated with ASTM D2488 when ASTM D422 or D4318 were not performed
**Other tests: DEN = Bulk Density, SPG = Specific Gravity, HYD = Hydrometer, CONSL = Consolidation, UCS = Unconfined Compression Strength, TRIAX = Triaxial

Summary of Sample Characteristics

Client: Bettisworth North Architects and Planners Inc.
Project: Haines Community Safety and Training Center
Project #: 202103

Borehole	Sample #	From	To	Sample Method	Liquid Limit (%)	Plastic Limit (%)	Gradation (%)			Max Particle Size (in)	Laboratory Classification*	Salinity (ppt)	Moisture (%)	Particle Shape	Angularity	Other Tests**
							Gravel	Sand	Fines*							
BH-6	3	5	7	Sh						3/8	CL-ML		25		SR	
BH-6	4	10	12	Sh							CL-ML		27			
BH-6	5	15	17	Sh							CL-ML		33			
BH-6	6	20	22	Sh							CL		30			
BH-6	7	25	27	Sh							CL		31			
BH-6	8	30	32	Sh							CL		29			

64 samples

*Fines type and content estimated with ASTM D2488 when ASTM D422 or D4318 were not performed

**Other tests: DEN = Bulk Density, SPG = Specific Gravity, HYD = Hydrometer, CONSL = Consolidation, UCS = Unconfined Compression Strength, TRIAX = Triaxial

Atterberg Test Results

Client: Bettisworth North Architects and Planners Inc.
Project: Haines Community Safety and Training Center
Project #: 202103

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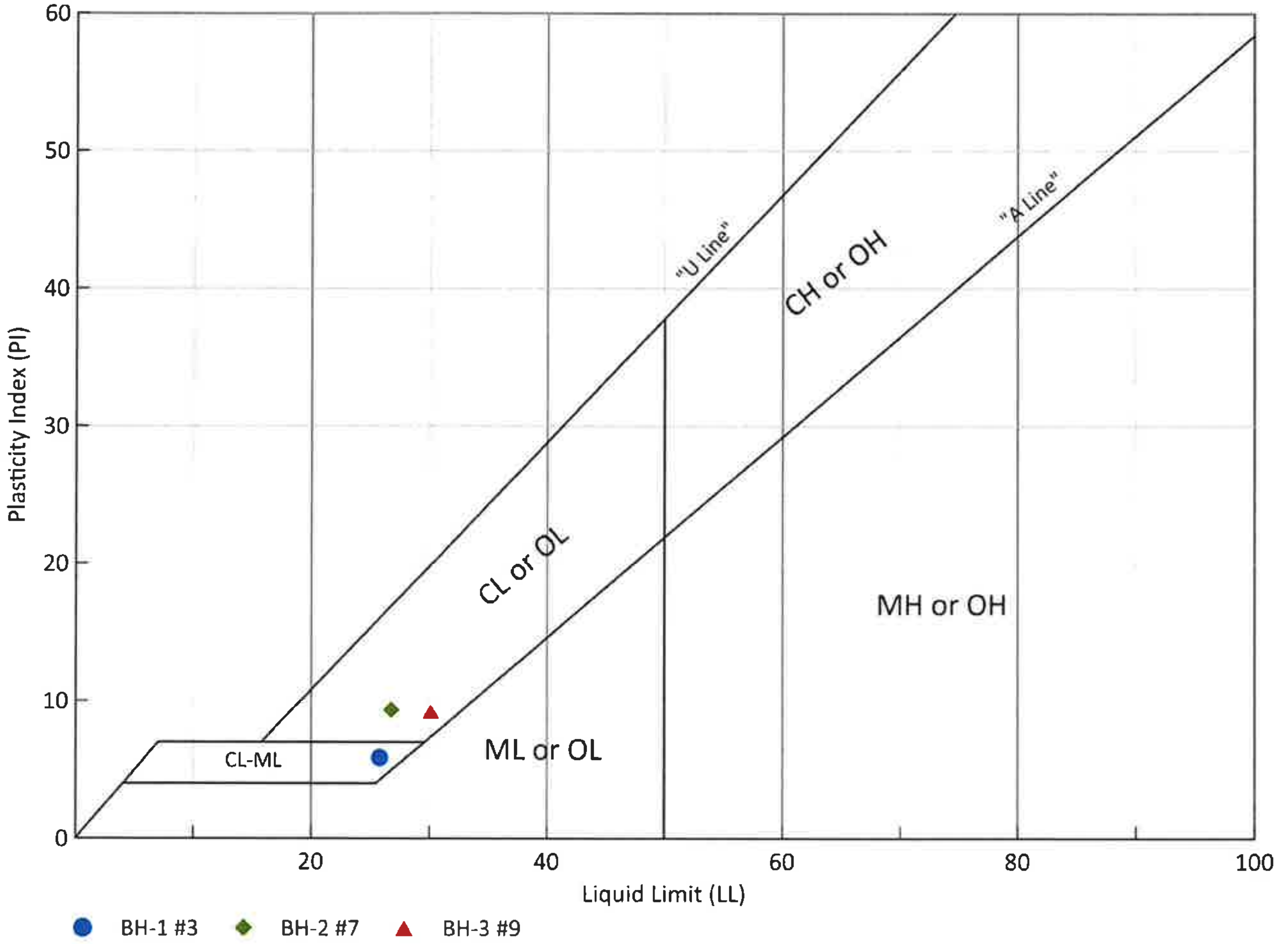
ENGINEERS, INC.



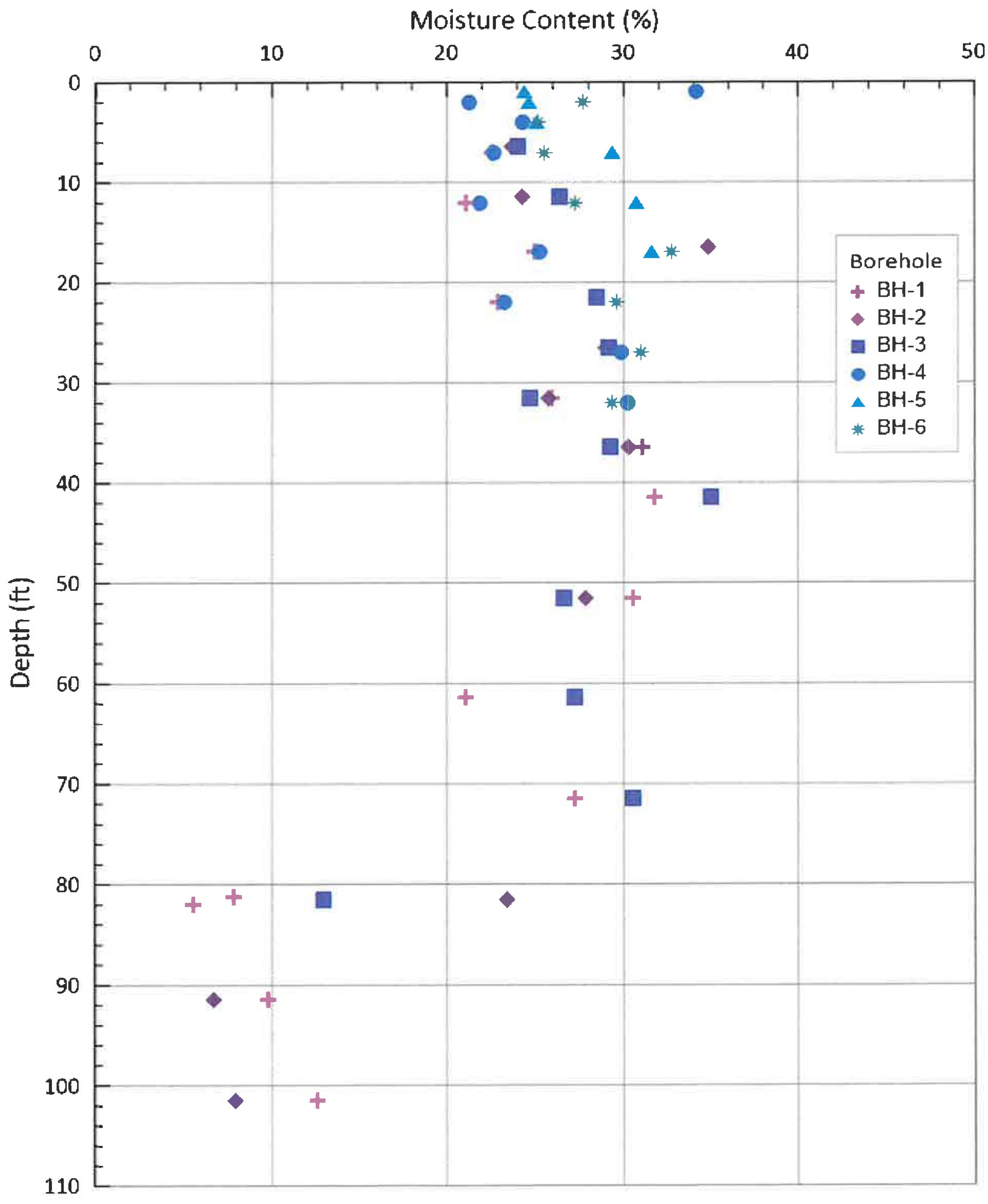
Borehole	Sample #	From	To	Moisture %	LL	PL	PI	Soil Type
BH-1	3	15	17	25.01%	25.8	19.93	5.9	CL-ML
BH-2	7	35	36.5	30.27%	26.8	17.44	9.3	CL
BH-3	9	50	51.5	26.65%	30.1	20.88	9.2	CL

P | N | D
ENGINEERS, INC.

MOISTURE CONTENT BY DEPTH
HAINES COMMUNITY SAFETY AND TRAINING CENTER

Client: Bettisworth North Architects and Planners, Inc.
Project: Haines Community Safety and Training Center
Project #: 202103

Location: Haines, Alaska
Reviewed By: SS
Review Date: 7/8/2022



Grain Size Distribution

Client: Bettisworth North Architects and Planners Inc.
Project: Haines Community Safety and Training Center
Project #: 202103

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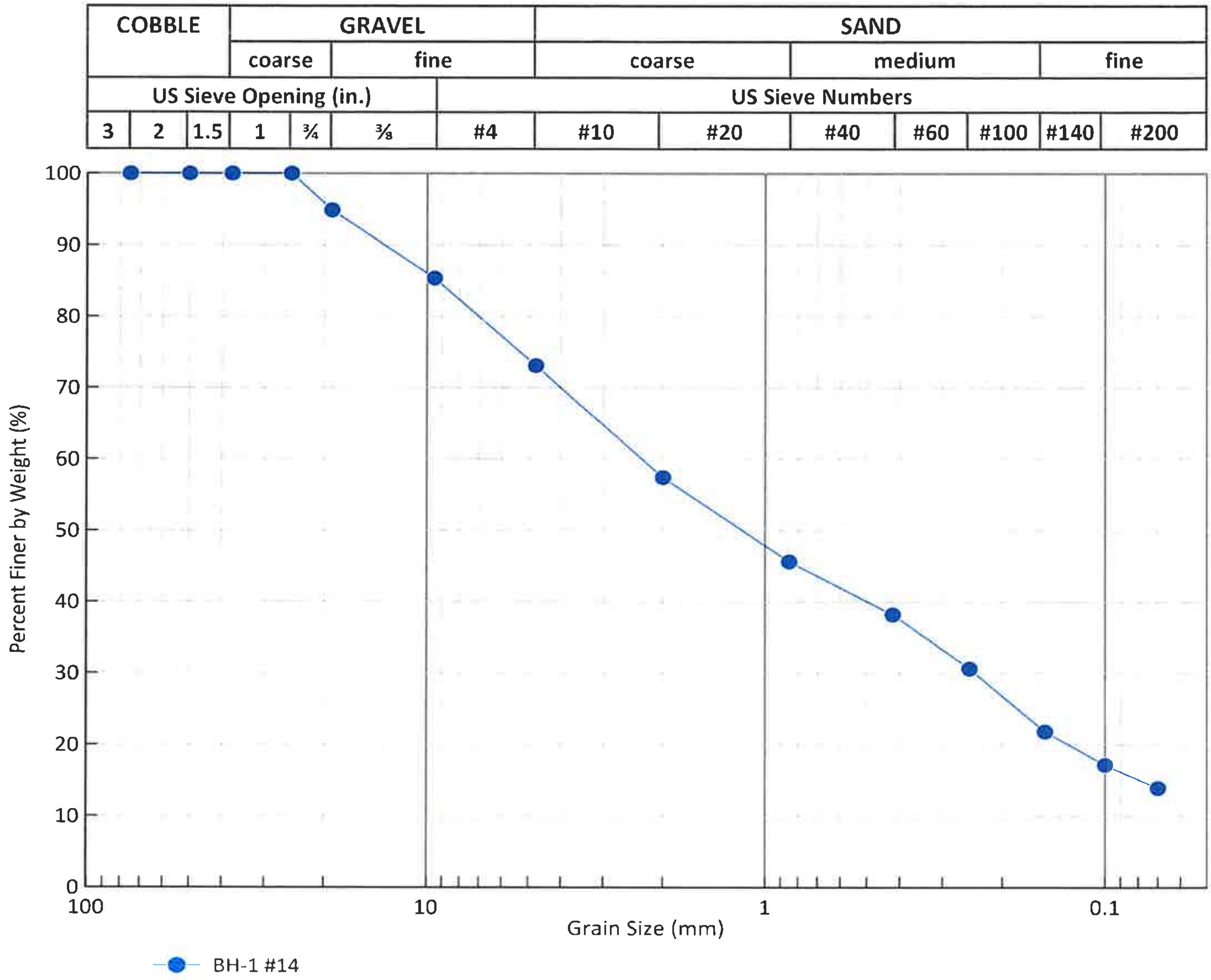
ENGINEERS, INC.



Borehole	Sample #	From	To	Laboratory Classification*	Gradation (%)			D50	P10
					Gravel	Sand	Fines		
BH-1	14	100	101.5	SCg	26.9	59.1	13.9	1.2	57.4

*Fines type estimated with ASTM D2488 when ASTM D422 or D4318 were not performed

APPENDIX C – FIELD INVESTIGATION PHOTOGRAPHS

Photo 1:

B-1, Sample #1, 5'-7' bgs

Photo 2:

B-1, Sample #2, 10'-12' bgs

Photo 3:

B-1, Sample #3, 15'-17' bgs

Photo 4:

B-1, Sample #4, 20'-22' bgs

	
Photo 5: B-1, Sample #5, 25’-27’	**Photo 6:** B-1, Sample #6, 30’-31.5’ bgs
	
Photo 7: B-1, Sample #7, 35’-36.5’ bgs	**Photo 8:** B-1, Sample #8, 40’-41.5’ bgs

Photo 9:

B-1, Sample #9, 50'-51.5' bgs

Photo 10:

B-1, Sample #10, 60'-61.5' bgs

Photo 11:

B-1, Sample #11, 70'-71.5'

Photo 12:

B-1, Sample #12, 80'-81.5' bgs

Photo 13:

B-1, Sample #13, 90.0’-91.5’ bgs

Photo 14:

B-1, Sample #13, 90.0’-91.5’ bgs

Photo 15:

B-1, Sample #14, 100.0’-101.5’ bgs

Photo 16:

B-2, Sample #1, 5.0’-6.5’ bgs

Photo 17:

B-2, Sample #2, 10.0-11.5' bgs

Photo 18:

B-2, Sample #3, 15.0'-16.5' bgs

Photo 19:

B-2, Sample #4, 20.0'-21.5' bgs

Photo 20:

B-2, Sample #5, 25.0'-26.5' bgs

Photo 21:

B-2, Sample #6, 50.0'-51.5' bgs

Photo 22:

B-2, Sample #7, 55.0'-56.5' bgs

Photo 23:

B-2, Sample #8, 40.0'-42.0' bgs

Photo 24:

B-2, Sample #9, 50.0'-51.5' bgs

Photo 25:

B-2, Sample #11, 70.0'-71.5' bgs

Photo 26:

B-2, Sample #12, 80.0'-81.5' bgs

Photo 27:

B-2, Sample #13, 90.0'-91.5' bgs

Photo 28:

B-2, Sample #14, 100.0'-101.5' bgs

	
Photo 29: B-3, Sample #1, 5.0'-6.5' bgs	**Photo 30:** B-3, Sample #2, 10.0'-11.5' bgs
	
Photo 31: B-3, Sample #3, 15.0'-17.0' bgs	**Photo 32:** B-3, Sample #4, 20.0'-21.5' bgs

Photo 33:

B-3, Sample #5, 25.0'-26.5' bgs

Photo 34:

B-3, Sample #6, 30.0'-31.5' bgs

Photo 35:

B-3, Sample #7, 35.0'-36.5' bgs

Photo 36:

B-3, Sample #8, 40.0'-41.5' bgs

	
Photo 37: B-3, Sample #9, 50.0'-51.5' bgs	**Photo 38:** B-3, Sample #10, 60.0'-61.5' bgs
	
Photo 39: B-3, Sample #11, 70.0'-71.5' bgs	**Photo 40:** B-3, Sample #12, 80.0'-81.5' bgs

Photo 39:

B-4, Sample #1, 0.0'-2.0' bgs

Photo 40:

B-4, Sample #2, 2.0'-4.0' bgs

Photo 39:

B-4, Sample #3, 5.0'-7.0' bgs

Photo 40:

B-4, Sample #4, 10.0'-12.0' bgs

	
Photo 41: B-4, Sample #5, 15.0'-17.0' bgs	**Photo 42:** B-4, Sample #6, 20.0'-22.0' bgs
	
Photo 43: B-4, Sample #7, 25.0'-27.0' bgs	**Photo 44:** B-4, Sample #8, 30.0'-32.0' bgs

	
Photo 45: B-5, Sample #1, 0.0'-2.0' bgs	**Photo 46:** B-5, Sample #2, 2.0'-4.0' bgs
	
Photo 47: B-5, Sample #3, 5.0'-7.0' bgs	**Photo 48:** B-5, Sample #4, 10.0'-12.0' bgs

	
Photo 49: B-6, Sample #1, 0.0'-2.0' bgs	**Photo 50:** B-6, Sample #2, 2.0'-4.0' bgs
	
Photo 51: B-6, Sample #3, 5.0'-7.0' bgs	**Photo 52:** B-6, Sample #4, 10.0'-12.0' bgs

Photo 53:

B-6, Sample #5, 15.0'-17.0' bgs

Photo 54:

B-6, Sample #6, 20.0'-22.0' bgs

Photo 55:

B-6, Sample #7, 25.0'-27.0' bgs

Photo 56:

B-6, Sample #8, 30.0'-32.0' bgs

	
Photo 57: Geoprobe 6712 DT enroute to B-6	**Photo 58:** Geoprobe 6712 DT stalled enroute to B-6
	
Photo 59: Excavating Geoprobe 6712 DT	**Photo 60:** Auger casing after B-5

APPENDIX D – CLIMATE DATA

HAINES 40 NW, ALASKA

Period of Record General Climate Summary - Temperature

Station:(503504) HAINES 40 NW

From Year=1989 To Year=2012

	Monthly Averages			Daily Extremes				Monthly Extremes				Max. Temp.		Min. Temp.	
	Max.	Min.	Mean	High	Date	Low	Date	Highest Mean	Year	Lowest Mean	Year	>= 90 F	<= 32 F	<= 32 F	<= 0 F
	F	F	F	F	dd/yyyy or yyyymmdd	F	dd/yyyy or yyyymmdd	F	-	F	-	# Days	# Days	# Days	# Days
January	22.1	10.2	16.2	44	20/2009	-24	31/1990	28.6	2001	2.4	1996	0.0	24.3	30.7	7.1
February	29.0	14.2	21.6	54	28/1991	-24	01/1990	30.4	2010	9.4	1994	0.0	15.3	27.7	4.1
March	34.4	17.4	25.9	58	30/1994	-16	04/2007	33.5	2005	19.9	2007	0.0	9.5	30.4	2.3
April	45.8	27.2	36.5	70	28/2003	1	02/1996	39.5	2005	29.7	2002	0.0	0.7	24.9	0.0
May	56.9	34.6	45.7	77	31/2011	22	06/2002	50.0	2005	42.2	1992	0.0	0.0	9.9	0.0
June	65.7	42.8	54.3	89	26/2004	25	30/2008	58.5	2004	50.1	2008	0.0	0.0	0.4	0.0
July	68.4	47.6	58.0	90	29/2009	31	23/1999	62.7	2009	53.1	2008	0.0	0.0	0.1	0.0
August	66.2	45.8	56.0	92	13/2005	19	27/1999	60.4	1994	52.8	2011	0.1	0.0	0.3	0.0
September	56.1	39.3	47.7	77	09/1989	21	08/2011	52.2	2006	41.6	1992	0.0	0.0	4.9	0.0
October	43.5	31.1	37.3	67	02/2003	10	26/2012	44.1	2006	33.0	1996	0.0	1.0	18.0	0.0
November	29.5	18.6	24.1	51	02/2003	-15	22/2011	33.5	2002	12.3	1990	0.0	16.4	29.0	2.4
December	25.0	14.6	19.8	49	23/1999	-25	02/1990	27.1	1997	10.6	1990	0.0	22.2	30.5	3.7
Annual	45.2	28.6	36.9	92	20050813	-25	19901202	39.4	2004	34.3	1996	0.2	89.4	206.6	19.6
Winter	25.4	13.0	19.2	54	19910228	-25	19901202	23.2	2010	13.0	1996	0.0	61.8	88.8	14.9
Spring	45.7	26.4	36.1	77	20110531	-16	20070304	41.0	2005	31.8	2002	0.0	10.2	65.1	2.3
Summer	66.8	45.4	56.1	92	20050813	19	19990827	60.0	2004	52.6	2008	0.2	0.0	0.8	0.0
Fall	43.0	29.7	36.3	77	19890909	-15	20111122	40.6	2002	31.7	1990	0.0	17.4	51.8	2.4

Table updated on Oct 31, 2012
For monthly and annual means, thresholds, and sums:
Months with 5 or more missing days are not considered
Years with 1 or more missing months are not considered
Seasons are climatological not calendar seasons
Winter = Dec., Jan., and Feb. Spring = Mar., Apr., and May
Summer = Jun., Jul., and Aug. Fall = Sep., Oct., and Nov.

HAINES 40 NW, ALASKA

Period of Record General Climate Summary - Precipitation

Station:(503504) HAINES 40 NW

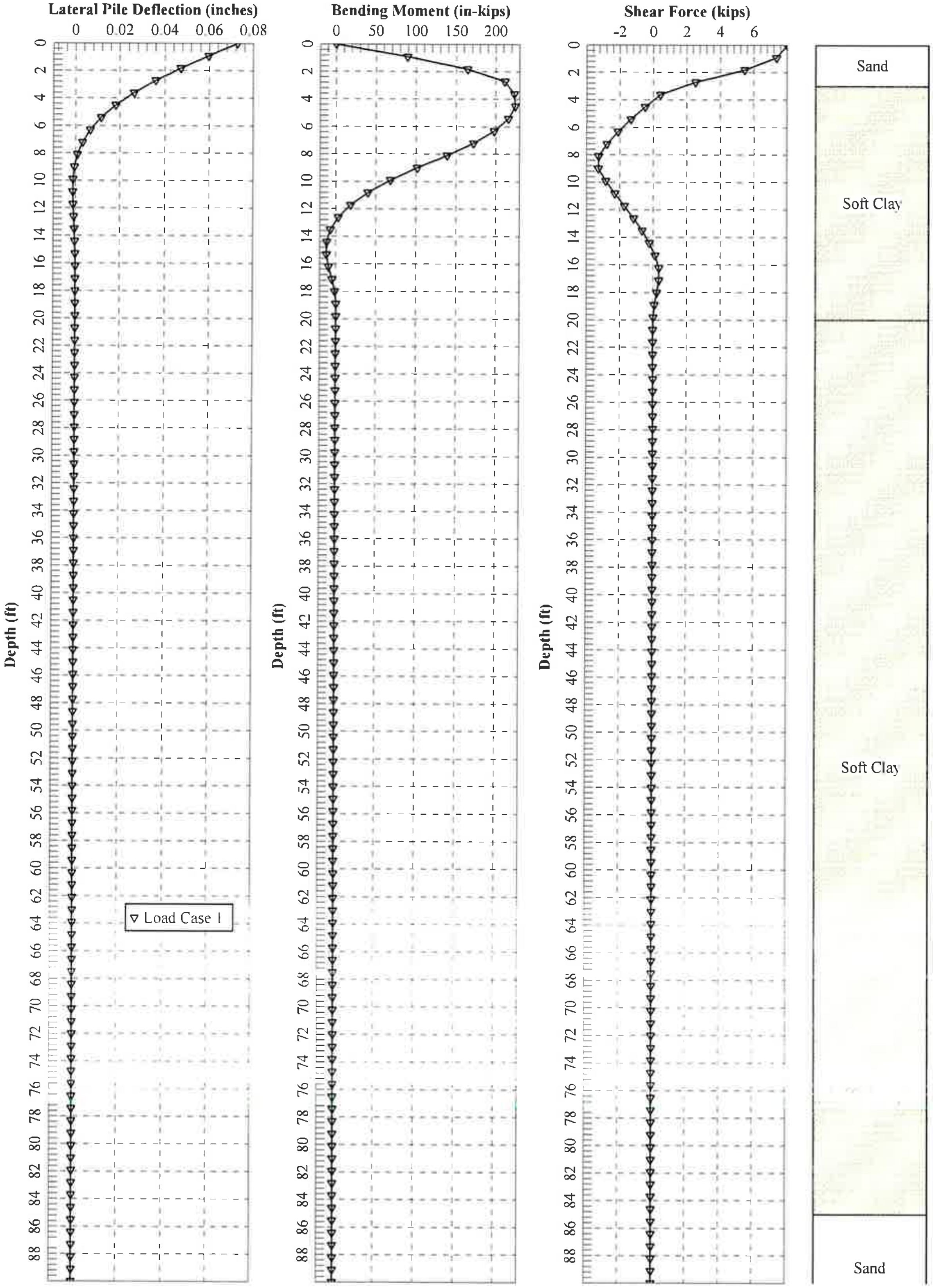
From Year=1989 To Year=2012

	Precipitation											Total Snowfall		
	Mean	High	Year	Low	Year	1 Day Max.		>= 0.01 in.	>= 0.10 in.	>= 0.50 in.	>= 1.00 in.	Mean	High	Year
	in.	in.	-	in.	-	in.	dd/yyyy or yyyymmdd	# Days	# Days	# Days	# Days	in.	in.	-
January	5.21	10.68	1992	1.87	2011	2.87	06/2003	15	11	3	1	52.4	123.5	2012
February	4.54	7.62	2004	0.38	1994	3.11	09/2004	13	9	3	1	40.2	79.7	1999
March	4.38	12.15	2010	0.61	2002	7.70	13/2009	12	9	2	1	36.4	110.9	2010
April	2.10	4.02	1999	0.13	2002	1.52	02/1997	10	5	1	0	6.0	20.5	1997
May	1.78	4.15	2012	0.06	2004	1.52	04/1992	10	5	1	0	0.2	1.8	1992
June	1.13	1.88	2001	0.08	1998	0.71	02/2001	9	4	0	0	0.0	0.0	1990
July	1.27	3.45	2007	0.04	2009	0.91	17/2007	11	4	0	0	0.0	0.0	1990
August	2.61	7.06	2009	0.58	2007	2.10	28/2009	14	7	1	0	0.0	0.0	1990
September	5.79	9.95	2011	2.33	2002	4.00	16/2009	18	10	4	2	0.2	2.3	1992
October	6.98	11.93	2008	1.96	2012	3.58	20/1998	19	13	5	2	10.3	38.1	2001
November	6.21	12.79	1999	1.13	2006	5.09	01/1999	16	12	4	1	50.0	107.9	1999
December	7.42	17.04	2006	2.22	1996	3.40	22/2003	18	14	6	2	65.4	159.5	2006
Annual	49.43	73.16	1999	32.06	1996	7.70	20090313	164	104	31	10	261.2	429.4	1999
Winter	17.18	27.03	1992	9.77	2011	3.40	20031222	45	34	12	4	158.1	245.8	2007
Spring	8.25	14.35	2010	1.80	2002	7.70	20090313	32	20	5	1	42.6	112.4	2010
Summer	5.02	8.43	2009	2.02	2004	2.10	20090828	34	15	2	0	0.0	0.0	1990
Fall	18.98	32.00	1999	11.55	2003	5.09	19991101	53	35	13	5	60.5	128.7	1999

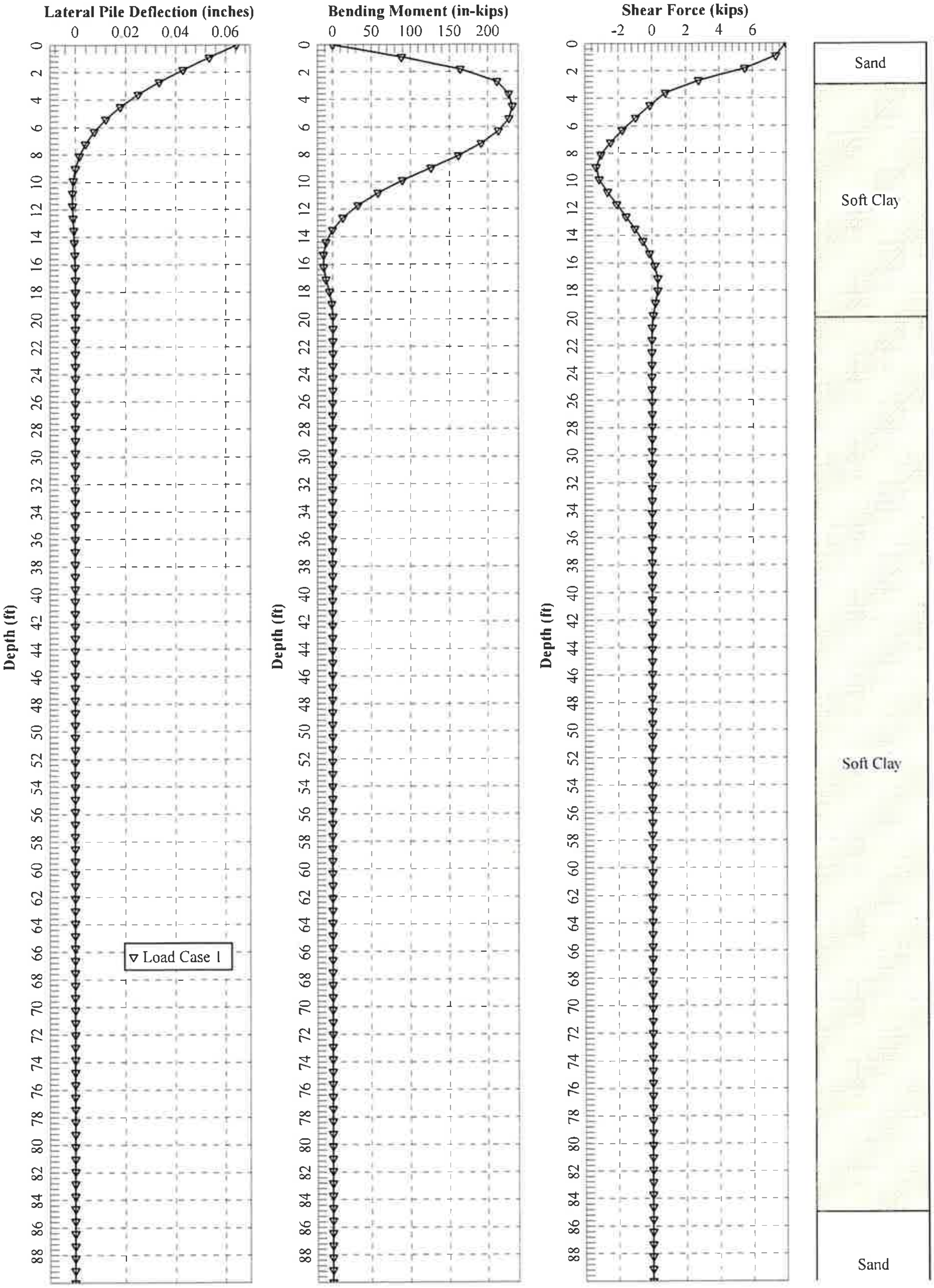
Table updated on Oct 31, 2012
For monthly and annual means, thresholds, and sums:
Months with 5 or more missing days are not considered
Years with 1 or more missing months are not considered
Seasons are climatological not calendar seasons
Winter = Dec., Jan., and Feb. Spring = Mar., Apr., and May
Summer = Jun., Jul., and Aug. Fall = Sep., Oct., and Nov.

APPENDIX E – LATERAL LOAD ANALYSES

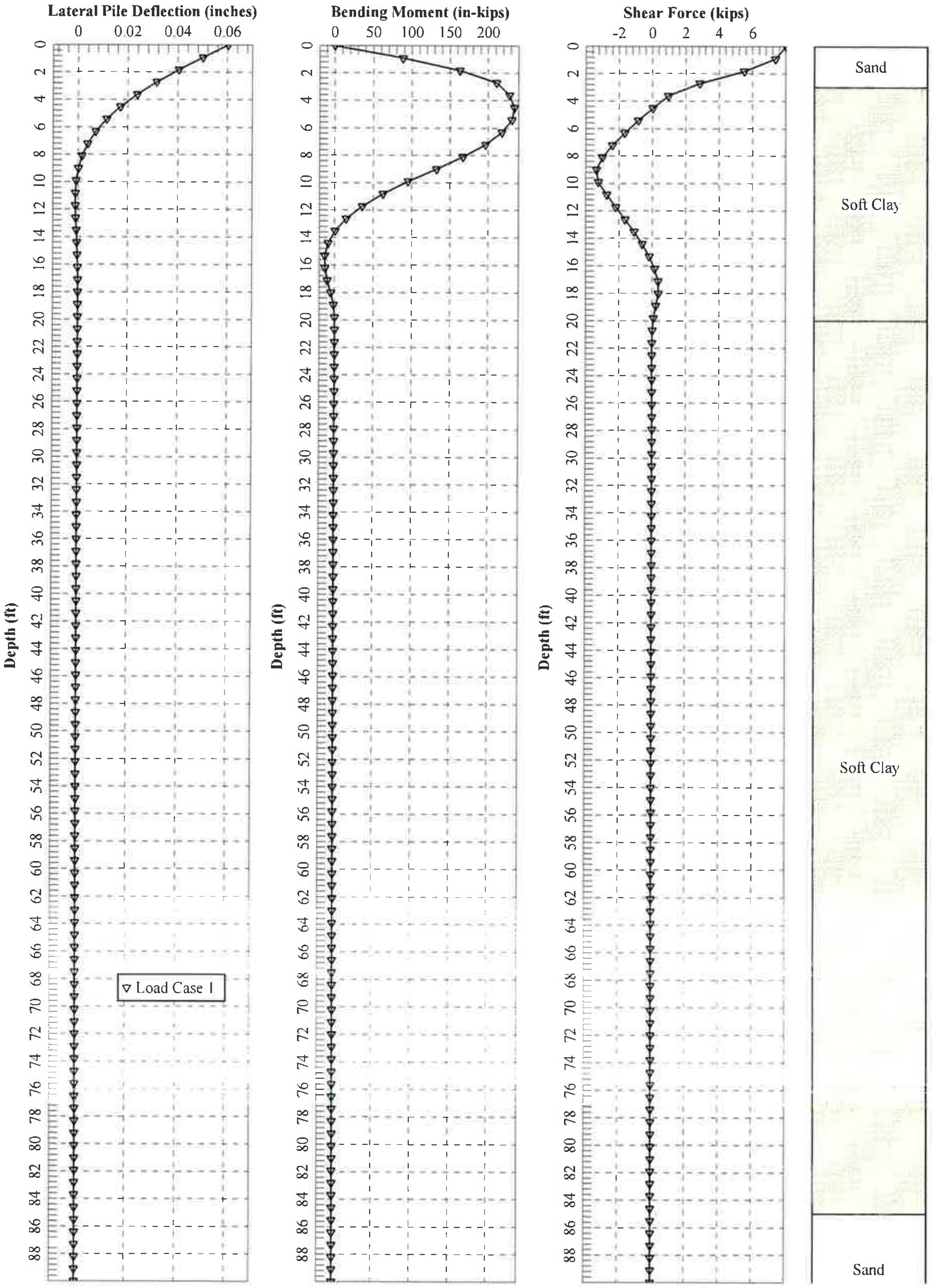
16" Diameter Pipe Pile
3/8" Wall Thickness



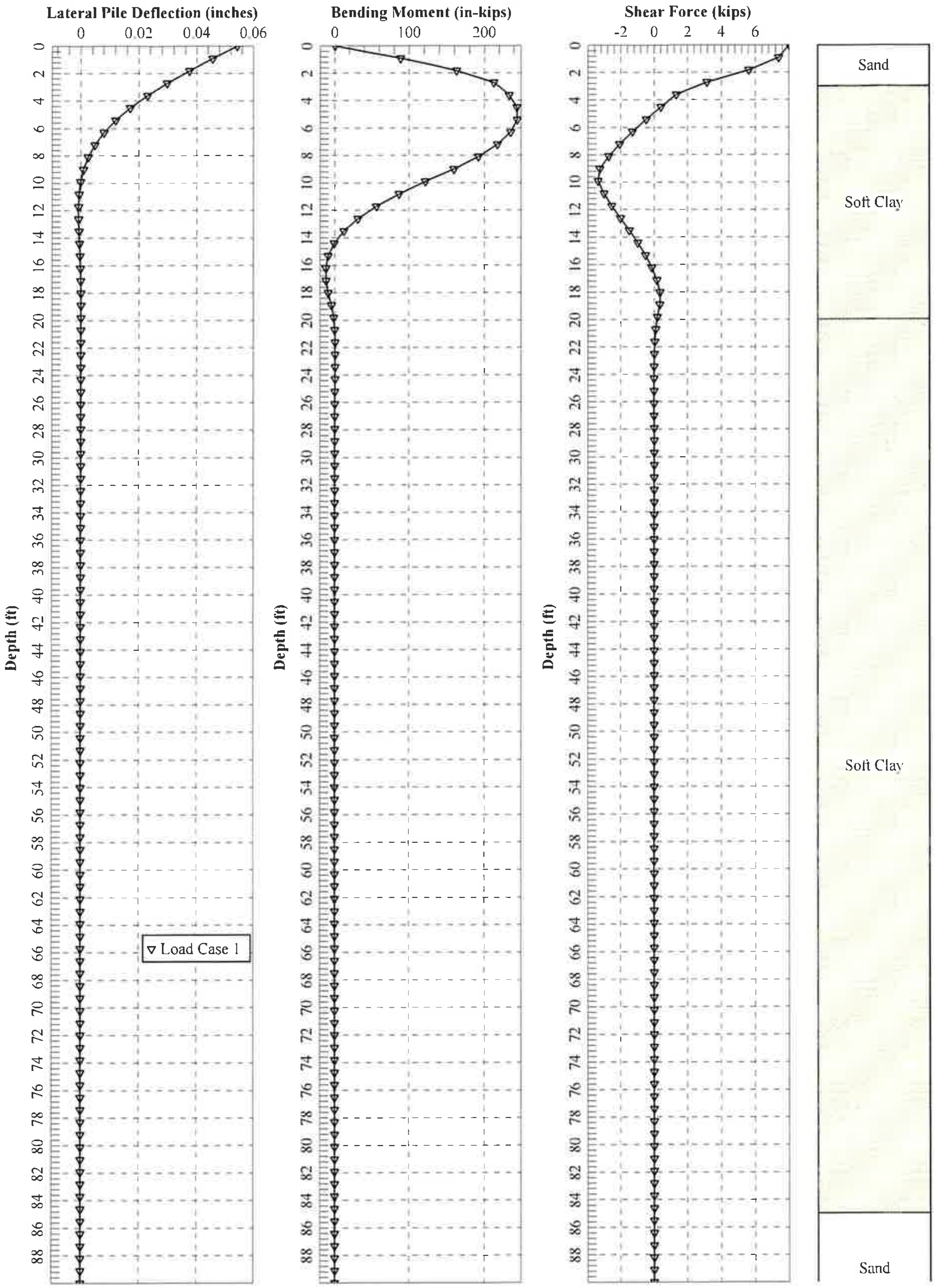
16" Diameter Pipe Pile
1/2" Wall Thickness



18" Diameter Pipe Pile
3/8" Wall Thickness



18" Diameter Pipe Pile
1/2" Wall Thickness



APPENDIX E – GBA PUBLICATION

Important Information about This
Geotechnical-Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way, clients can benefit from a lowered exposure to the subsurface problems that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed below, contact your GBA-member geotechnical engineer. Active involvement in the Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Geotechnical-Engineering Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical-engineering study conducted for a given civil engineer will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client. *Those who rely on a geotechnical-engineering report prepared for a different client can be seriously misled.* No one except authorized client representatives should rely on this geotechnical-engineering report without first conferring with the geotechnical engineer who prepared it. *And no one – not even you – should apply this report for any purpose or project except the one originally contemplated.*

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read it *in its entirety.* Do not rely on an executive summary. Do not read selected elements only. *Read this report in full.*

You Need to Inform Your Geotechnical Engineer about Change

Your geotechnical engineer considered unique, project-specific factors when designing the study behind this report and developing the confirmation-dependent recommendations the report conveys. A few typical factors include:

- the client's goals, objectives, budget, schedule, and risk-management preferences;
- the general nature of the structure involved, its size, configuration, and performance criteria;
- the structure's location and orientation on the site; and
- other planned or existing site improvements, such as retaining walls, access roads, parking lots, and underground utilities.

Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light-industrial plant to a refrigerated warehouse;
- the elevation, configuration, location, orientation, or weight of the proposed structure;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes – even minor ones – and request an assessment of their impact. *The geotechnical engineer who prepared this report cannot accept responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.*

This Report May Not Be Reliable

Do not rely on this report if your geotechnical engineer prepared it:

- for a different client;
- for a different project;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, that it could be unwise to rely on a geotechnical-engineering report whose reliability may have been affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If your geotechnical engineer has not indicated an "apply-by" date on the report, ask what it should be*, and, in general, *if you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying it. A minor amount of additional testing or analysis – if any is required at all – could prevent major problems.

Most of the "Findings" Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site's subsurface through various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing were performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgment to form opinions about subsurface conditions throughout the site. Actual sitewide-subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team from project start to project finish, so the individual can provide informed guidance quickly, whenever needed.

This Report's Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, *they are not final*, because the geotechnical engineer who developed them relied heavily on judgment and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* revealed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation.*

This Report Could Be Misinterpreted

Other design professionals' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a full-time member of the design team, to:

- confer with other design-team members,
- help develop specifications,
- review pertinent elements of other design professionals' plans and specifications, and
- be on hand quickly whenever geotechnical-engineering guidance is needed.

You should also confront the risk of constructors misinterpreting this report. Do so by retaining your geotechnical engineer to participate in prebid and preconstruction conferences and to perform construction observation.

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note conspicuously that you've included the material for informational purposes only.* To avoid misunderstanding, you may also want to note that "informational purposes" means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report, but they may rely on the factual data relative to the specific times, locations, and depths/elevations referenced. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually relate any environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures.* If you have not yet obtained your own environmental information, ask your geotechnical consultant for risk-management guidance. As a general rule, *do not rely on an environmental report prepared for a different client, site, or project, or that is more than six months old.*

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, none of the engineer's services were designed, conducted, or intended to prevent uncontrolled migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, ***proper implementation of the geotechnical engineer's recommendations will not of itself be sufficient to prevent moisture infiltration.*** Confront the risk of moisture infiltration by including building-envelope or mold specialists on the design team. ***Geotechnical engineers are not building-envelope or mold specialists.***

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ASSOCIATION

Telephone: 301/565-2733
e-mail: info@geoprofessional.org www.geoprofessional.org

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