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FINAL REV3

GEOTECHNICAL RECOMMENDATIONS

Haines Maintenance & Operations Station; Project No. 57183-B

HAINES, ALASKA

July 6, 2024

Shannon & Wilson No: 110813-001

SHANNON & WILSON

Submitted To: RESPEC
9101 Mendenhall Mall Road, Suite 4
Juneau, Alaska 99801
Attn: Douglas Murray, P.E.

Subject: FINAL GEOTECHNICAL RECOMMENDATIONS, HAINES MAINTENANCE & OPERATIONS STATION; PROJECT NO. 57183-B, HAINES, ALASKA

Shannon & Wilson prepared this report and participated in this project as a subconsultant to RESPEC. Our scope of services was specified in document titled, "Subconsultant Agreement" between Shannon & Wilson, Inc. and RESPEC Company, L.L.C. dated August 23, 2023. This report presents our geotechnical recommendations and was prepared by the undersigned.

We appreciate the opportunity to be of service to you on this project. If you have questions concerning this report, or we may be of further service, please contact us.

Sincerely,

SHANNON & WILSON (AECC 125)

Wendy Presler, P.E.
Senior Associate

This report has been updated and supersedes all previous versions.

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ACRONYMS

AASHTO	American Association of State Highway and Transportation Officials
DOT&PF	Department of Transportation and Public Facilities
ASCE	American Society of Civil Engineers
ASTM	ASTM International
bgs	below the ground surface
FS	factor of safety
GeoTek	GeoTek Alaska, Inc.
hr-ft-˚F/Btu	hour-foot-degree Fahrenheit per British thermal unit
H:V	horizontal to vertical ratio
IBC	International Building Code
M&O	Maintenance and Operations
MCE	maximum considered earthquake
Ms	earthquake magnitude
MSPT	modified Standard Penetration Test
NFS	non-frost susceptible
N-value	Standard Penetration Test resistance value
PCASE	Pavement-Transportation Computer Assisted Structural Engineering
PGA_M	mean peak ground acceleration
psf	pounds per square foot
PVC	polyvinyl chloride
RSS	reduced subgrade strength
SPT	Standard Penetration Test
USACE	United States Army Corps of Engineers
USGS	United States Geological Survey

1 INTRODUCTION

This report was prepared for the Alaska Department of Transportation and Public Facilities (DOT&PF) in support of the proposed Maintenance and Operations (M&O) facility near Haines, Alaska. This report presents our geotechnical findings, engineering analyses, conclusions, and recommendations. The following introductory sections summarize our scope of services and project understanding.

1.1 Scope of Services

The scope of our geotechnical services included site subsurface exploration, laboratory testing, engineering analyses, and development of this geotechnical recommendations report.

The authorized scope of services was based on your objectives, schedule, and budget. Our scope of services did not include evaluating the presence of cultural resources, conducting wetland delineation, or research and evaluation of contaminated sites near the proposed project. If a service is not specifically indicated in this report, do not assume that it was performed.

This report was prepared for the exclusive use of our client, RESPEC, LLC, for this project.

1.2 Project Understanding

We understand the project includes the construction of a 10,800-square-foot shop on a lot northeast of the existing M&O building. Additional infrastructure proposed to be developed include a sand and chemical storage building, and a refueling station. We also understand the development will include gravel parking areas, an equipment yard, and access drives. We understand, and observed during field activities, that the site was previously developed but allowed to revegetate. Clearing activities were completed by DOT&PF prior to our site visit in October 2023.

A vicinity map showing the site location is presented in Figure 1.

1.3 Criteria and Guidance

The following documents pertaining to geotechnical design were referenced to perform calculations and develop recommendations for this report:

- American Society of Civil Engineers (ASCE) 7-16,

- ASCE 32-01 Frost Protected Shallow Foundations, and
- International Building Code (IBC) 2021.

2 FIELD AND LABORATORY STUDIES

The field explorations consisted of advancing six exploratory borings designated 23-01 through 23-06, at the site on October 9 and 10, 2023. We created field logs of the soil borings and collected representative soil samples from the borings for soil index testing in our Fairbanks soils laboratory. Borings 23-01 through 23-03 were advanced within the proposed maintenance facility footprint to depths of 41.5 feet to 44 feet below the ground surface (bgs). Boring 23-04 was advanced within the proposed refueling tank footprint to a depth of 21.5 feet bgs. Boring 23-05 was advanced near an existing gravel stockpile to a depth of 26.5 feet bgs. Boring 23-06 was advanced southeast of the proposed maintenance facility footprint to a depth of 21.5 feet bgs.

The purpose of the borings was to observe subsurface ground conditions, including the depth to groundwater at the time of drilling, and to collect data for use in our engineering analyses. Data collected from our subsurface explorations are the basis for our recommendations for the proposed project.

Figure 2 shows the approximate boring locations at the site.

2.1 Field Exploration and Drilling Methods

We subcontracted GeoTek Alaska, Inc. (GeoTek) of Anchorage, Alaska to advance the soil borings. GeoTek advanced the borings using a Geoprobe 7718 DT rubber-track-mounted drill rig equipped with continuous-flight hollow-stem augers.

2.1.1 Geotechnical Soil Borings

GeoTek generally collected samples at 2.5-foot intervals from the ground surface to 20 feet bgs and at 5-foot intervals to the bottom of the borings using a 2.5-inch inside-diameter split-spoon sampler consistent with procedures discussed in ASTM International (ASTM) D1586 *Standard Method for Penetration Test and Split-Barrel Sampling of Soils*. The split-spoon samples were obtained by driving the sampler into the soil at the base of the auger using a 340-pound automatic hammer falling 30 inches onto the drill rods, consistent with Modified Standard Penetration Test (MSPT) procedures. For each sample, the number of blows required to advance the sampler the 12-inch interval between 6 inches and 18 inches is termed the penetration resistance, a measure of the relative consistency of unfrozen fine-grained soil and relative density of unfrozen granular soil. We classified soil samples recovered using these techniques in the field, sealed them in airtight containers, and returned them to our laboratory for testing.

Stephen Chase, an experienced geotechnical staff member with our firm, observed drilling operations, created a boring log for each boring, and collected the soil samples for index testing. Soil samples were photographed, recorded, and placed in air-tight containers for transport to our Fairbanks, Alaska laboratory. The borings were backfilled with drill cuttings.

Soil and drilling observations are included in the boring logs presented in Appendix A. Our observations are specific to the locations, depths, and dates noted on the logs, and may not be applicable to all areas of the site. Photographs of the site, drilling operations, and select soil samples are presented in Appendix D.

2.2 Laboratory Testing

We visually reviewed field soil classifications in our laboratory and selected samples for testing. We performed moisture-content analyses consistent with ASTM D2216 *Standard Test Methods for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass* on all samples collected above the water table. We conducted grain-size distribution analyses on 8 samples, consistent with ASTM C136 *Standard Test Method for Sieve Analysis of Fine and Coarse Aggregate*. We conducted hydrometer analyses consistent with ASTM D422 *Standard Test Method for Particle-Size Analysis of Soils* on 2 samples and Atterberg limit testing consistent with ASTM D4318 *Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils* on 2 samples. Moisture-content results are plotted on the boring logs in Appendix A. Grain-size distribution curves and Atterberg limit results are presented in Appendix B.

2.3 Piezometer Installation

After drilling Boring 23-03, a two-inch polyvinyl chloride (PVC) casing was installed to 14.4 feet bgs. The bottom 12 inches of the PVC casing was slotted to allow groundwater to flow freely into the casing. After allowing the groundwater level to equilibrate overnight, we collected groundwater level data. We collected groundwater level data using a Slope Indicator Company water level indicator. We observed groundwater at 4.8 feet bgs in Boring 23-03. The casing was left in place to allow for future measurements, if desired.

3 SITE CONDITIONS

3.1 Surface Conditions

During our field activities, we observed a mixture of vegetated and gravel areas. The project areas are described below.

The proposed maintenance facility was located in a recently cleared area on the northeast corner of the site. Remaining vegetation was a mixture of mature aspen and willow trees with thick shrubbery and young spruce trees. Based on conversations with DOT&PF personnel on-site, the site was cleared a few days prior to our arrival on October 9, 2023. Further conversations revealed that the site was previously used as an RV park. Near Boring 23-06, we observed a sewage dump station and a water/electricity hookup.

The remaining areas of the property were found to be developed with existing structures and a storage yard. We observed a drainage ditch between the two gravel driveways in the southeast corner of the property, north of Boring 23-05. A large pile of gravel was present on the planned location of Boring 23-05; therefore, we relocated Boring 23-05 as shown in Figure 2. We observed three buildings on the site, generally located on the west half of the property. DOT&PF personnel in Haines indicated the existing maintenance facility foundation is performing well.

The topography of the site was gently sloping down to the west. Additionally, there is a gently sloped hill northeast of the site. Mountains were observed in all four cardinal directions at varying distances. The mountains to the north were closest to the site.

3.2 Subsurface Conditions

Based on the varying properties of the surface, we present our findings of subsurface conditions below.

3.2.1 Vegetated Areas

Borings drilled in the vegetated areas include Boring 23-01 through Boring 23-03 and Boring 23-06.

In general, we observed 3 to 6 feet of fill (presumably from the previous RV park) consisting of cobbles, gravel, sand, and varying quantities of silt overlying medium plasticity, fine-grained soil intermixed with wood debris and organics. The fine-grained soil was underlain by silty or clayey sandy soil underlain by lean clay to the depths explored with few exceptions. The exceptions were:

- Boring 23-02 encountered clayey fine gravel between 14.5 feet bgs and 17 feet bgs, and between 19.5 feet bgs and 22.5 feet bgs.
- Boring 23-06 encountered an interbedded layer of lean clay from 18.2 feet bgs to 19.5 feet bgs. The lean clay layer was within the sandy soil layer.

Groundwater was observed during drilling between approximately 5 and 9.5 feet bgs. Groundwater levels are subject to seasonal and annual fluctuation and may vary by 2 to 3 feet or more. We anticipate the groundwater levels were high at the time of drilling. DOT&PF personnel reported that there had been approximately 7 inches of rain in the week prior to drilling. Based on the presence of fine grain, clayey soils beneath the fill, we believe the groundwater levels underlying some portions of the site may have been shallower than regional groundwater.

We did not observe seasonally frozen soils or permafrost in borings drilled in the vegetated areas.

3.2.2 Gravel Areas

Borings drilled in the gravel areas included Boring 23-04 and Boring 23-05. These borings were located within the proposed refueling station footprint and near the gravel storage area, respectively.

In general, we observed 2 to 5.5 feet of fill underlain by medium plasticity, fine-grained soil intermixed with wood debris and organic material. The fine-grained soil was underlain by silty or clayey sand underlain by lean clay to the depths explored.

Groundwater was observed between 4.5 and 12 feet bgs. As stated above, we believe the groundwater levels underlying some portions of the site may have been shallower than regional groundwater during drilling.

We did not observe seasonally frozen soils or permafrost in borings drilled in the gravel areas.

4 SITE GEOLOGICAL SETTING AND SEISMICITY

4.1 Geological Setting

Haines lies on the southern end of the Chilkat Peninsula and has a complicated geologic past, including igneous intrusion, metamorphism, faulting, and considerable uplift. Both bedrock and surficial soil deposits comprise the ground surface.

The Chilkat Peninsula is part of the Wrangellia Terrane (Brew and Ford, 1994), which is composed of Paleozoic igneous extrusions intruded by Late-Paleozoic felsic rocks and Cretaceous ultramafic rocks. Exposed bedrock includes Mesozoic metabasalts and Cretaceous clinopyroxenites, diorites, and tonalites. Tertiary sedimentary rocks underlie Quaternary surficial deposits but are not exposed. Surficial deposits include undifferentiated drift, outwash deposits, fine-grained marine sediments, elevated delta and shore deposits, alluvial fans, and colluvial deposits.

The area has most recently undergone isostatic uplift due to the relief of overburden stress by the Pleistocene ice sheets (Lemke and Yehle, 1972). The presence of marine deposits located several hundred feet above sea level indicate a substantial uplift since the last major glaciation.

4.2 Seismicity

Haines lies between four mapped faults. The Chatham Strait section of the Denali Fault lies within approximately 10 miles to the east, the Coast Shear Zone lies within approximately 10 miles to the east, the Fairweather Fault lies approximately 80 miles west, and the Queen Charlotte-Fairweather Fault lies about 200 miles south. These faults are understood to be slip-strike fault as part of, or resultants of, the North American Plate slipping with the Pacific Plate along the Gulf of Alaska. These faults can be seen in Exhibit 4-1.

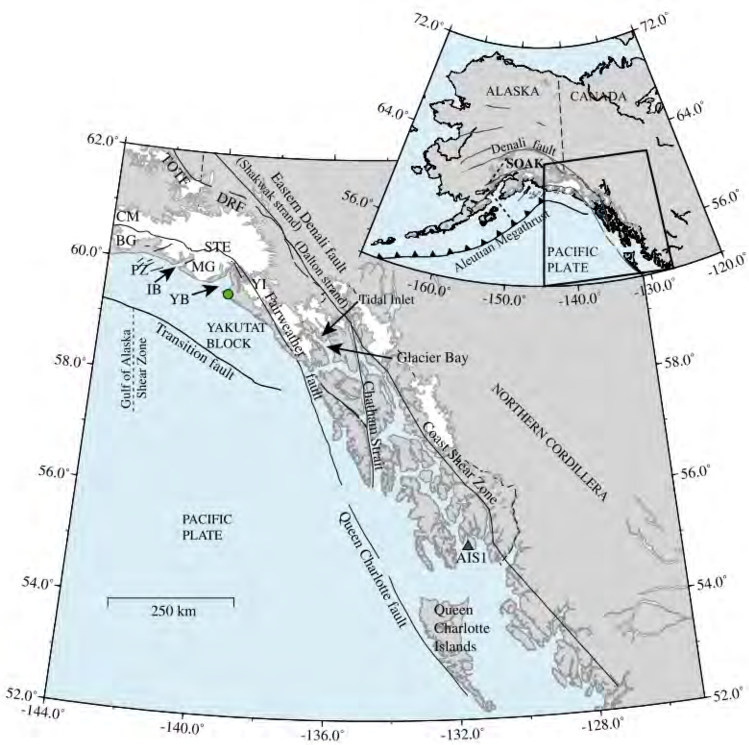

Exhibit 4-1: Faults around Haines (Elliott, 2010)

Seismicity in the Haines area has historically been concentrated in the Fairweather Fault, Queen Charlotte-Fairweather Fault, and the Coast Shear Zone. All three of these faults are seismically active with the Fairweather Fault and the Queen Charlotte-Fairweather Fault having a history of producing large earthquakes. Research of the Chatham Strait portion of the Denali Fault finds the fault is not seismically active; however, more research is needed to understand this fault's full behavior (Elliott, 2010).

Between 1927 and 2017, two earthquakes exceeding Magnitude (M_S) 7 occurred within approximately 110 miles of Haines:

- An M_S 7.8 event occurred on July 10, 1958, approximately 65 miles southwest of Haines.
- An M_S 7.3 event occurred on October 24, 1927, about 110 miles southwest of Haines.

Additionally, Haines has felt the effects of seven more earthquakes greater than Ms 7.0 since 1900 ranging from Valdez (Ms 9.3) to the northwest to Prince Rupert, Canada (Ms 7.8) to the southeast. The Queen Charlotte-Fairweather Fault has produced most of the earthquakes that have been measured in Haines. These large earthquakes have occurred to the north, west, and south all along the Queen Charlotte-Fairweather Fault.

5 EARTHQUAKE HAZARD ANALYSIS

The project is in a seismic area where major earthquakes can and have occurred. Earthquake-induced geologic hazards that may affect the site include ground-surface fault rupture, and liquefaction and associated effects (e.g. loss of shear strength, bearing-capacity failures, loss of lateral support, ground oscillation, and lateral spreading). An associated effect of earthquake shaking is densification of the soils and potential settlement of the ground surface.

Due to the presence of relatively loose, fine-grained soils and a shallow ground water table liquefaction is a moderate hazard at this site. The sandy zones encountered in the borings are the most likely to be subject to liquefaction because they have uncorrected, MSPT blow counts of less than 20 and are saturated. We believe liquefaction of soils underlying the site may cause differential settlement should an earthquake of sufficient magnitude and duration occur during the life of the structures.

Our analyses of earthquake ground motions and earthquake-induced geologic hazards that may affect the site are described below.

5.1 Earthquake Ground Motion

Structural design performed in seismic regions generally requires a site-specific seismic analysis. Sample penetration resistance values from the explorations suggest that Site Class E soil conditions prevail at the site, without regard for liquefaction.

We developed seismic ground motions for the liquefaction analyses in general accordance with the IBC 2021 Code. The five percent damped design spectral response acceleration is defined as two-thirds of the site-adjusted maximum considered earthquake (MCE). The MCE was determined using maps for bedrock ground motions published by the U.S. Geological Survey (USGS) for ground motions with a two percent chance of occurrence in 50 years. We adjusted these values assuming Site Class E conditions at the site. The mapped MCE geometric mean peak ground acceleration (PGA_M) was derived using 2016 ASCE 7. Exhibit 5-1 provides the earthquake ground motion parameters developed for this site.

Exhibit 5-1: Earthquake Ground Motion Parameters

Description (Parameter)	Value
Site Class	E
Mapped spectral acceleration for 0.2 seconds, Site Class B, 5% damping (S_S)	1.18 g
Mapped spectral acceleration for 1 second, Site Class B, 5% damping (S_1)	1.85 g
S_s adjusted for site class (S_{MS})	1.52 g
Design spectral response acceleration at short periods (S_{DS})	1.01 g
Mapped MCE geometric mean peak ground acceleration (PGA_M)	0.59 g

We note the IBC requires a site-specific ground motion study for Seismic Design Category F soils, which includes potentially liquefiable soils. The code allows for a waiver from the site-specific study if the period of vibration of the building is equal to or less than 0.5 seconds as outlined in ASCE 7, referenced in tables 1613.2.3(1) and 1613.2.3(2) of the 2021 IBC.

5.2 Liquefaction Analyses

Liquefaction of loose, saturated, cohesionless, unfrozen soil occurs when excess pore pressures are generated because of earthquake shaking. Additionally, densification of the granular soils above and below the water table could occur when subjected to earthquake shaking, resulting in differential ground settlement at the site.

The methods to evaluate liquefaction potential are empirical and based on correlations between standard penetration test (SPT) resistance (N-value), PGA, and earthquake magnitude. We performed our liquefaction analysis using soil data from Borings 23-01 through 23-03, advanced at the site October 9 and 10, 2023. Based on earthquakes that have occurred within 70 miles of the site in the last 100 years, we assumed an earthquake magnitude of 7.8 for our analyses. We assumed a PGA_M of 0.59 g, as listed above.

We used three procedures to evaluate liquefaction potential at this site:

- Youd and others (2001);
- Cetin and others (2004); and
- Idriss and Boulanger (2014)

In these procedures, the SPT N-value (blow count) is correlated to the liquefaction resistance of the soil (expressed as cyclic resistance ratio). The soil resistance is compared to the earthquake-induced loading (expressed as cyclic stress ratio) to calculate a corresponding factor of safety (FS) against liquefaction.

We performed our liquefaction analyses for earthquake ground motion parameters described above. Triggering of liquefaction is predicted for a FS of less than one. All three procedures predict liquefaction in samples from the groundwater table to 30 feet bgs. The primary effects of liquefaction at the adjacent site are reduction in soil shear strength, dynamic settlement, and a potential for lateral spreading.

The blow count data obtained during drilling was collected with a large-diameter sampler. The large sampler is frequently used in the area to obtain more representative samples and reduce the occurrence of large gravel particles interfering with the accuracy of the blow-count data. N-values obtained with the large-diameter sampler were corrected to SPT N-values by multiplying the N-value by 1.3, consistent with the recommendations by Daniel and others (2003). Where fractured gravels are noted in the boring logs, or blow counts are elevated due to the presence of gravel during drilling, the increase of thirty percent was not applied to the sample value for use in the liquefaction analyses.

5.3 Seismically Induced Settlement Analyses

An associated effect of earthquake shaking is densification (and the potential for associated settlement) of loose to medium-dense, cohesionless soil that undergoes liquefaction. We used the relationships developed by Youd, Idriss, and Andrus (2001); Tokimatsu and Seed (1987); and Idriss and Boulanger (2014); relating earthquake ground motion and penetration resistance with volumetric strain, to estimate the potential for free-field ground settlement.

Using these relationships, in conjunction with the three procedures used to evaluate liquefaction potential at this site, our liquefaction analyses of in situ soil suggest 2 to 5 inches, or more, of free-field settlement could occur at the ground surface, all of which could be differential.

We also considered the potential effect surcharging *could* have on the magnitude of anticipated dynamic settlement. Assuming surcharging will lead to densification of the upper 25 feet of the soil profile, and that the influence of surcharging decreases with depth, we anticipate 1 to 1.5 inches of anticipated dynamic settlement *may be* eliminated by surcharging, reducing the estimated total potential free-field settlement to 1 to 4 inches. Because liquefaction is controlled by the magnitude and duration of a seismic event, it is difficult to more accurately predict the potential magnitude of settlement after surcharging. Additionally, subsurface conditions may vary from those observed in our borings, and the effective depth of surcharging is estimated based on available data. For these reasons, we do not believe a reduction in dynamic settlement should be a factor in determining whether surcharging would be beneficial for the project.

Summary plots of our liquefaction analyses are presented in Appendix C.

5.4 Soil Strength Reduction during Liquefaction

We calculated reduction in soil-shear strength during liquefaction in borings using relationships by Olson and Stark (2003), Idriss & Boulanger (2007), and Kramer (2008) correlating SPT blow counts with apparent shear strengths. Based on the calculated mean of the residual strength from the three methods, the average residual internal friction angle of soils below the water table during liquefaction could be about 15 degrees for in situ soil conditions.

5.5 Lateral Spreading

Lateral spreading is a phenomenon that can occur in loose to medium-dense, saturated, granular soils beneath even very gently sloping ground surfaces and on level ground near slopes (i.e. free faces) such as hill slopes, riverbanks or lakes. Lateral spreading occurs due to the softening and weakening of liquefied soil; it differs from flow-sliding in that it occurs in soils whose residual strength is exceeded by the shear stresses required for static equilibrium. As a result, lateral spreading deformations generally occur during the period of earthquake ground shaking and the deformations develop in an incremental manner.

Due to the relatively flat topography and distance from slopes, deep cuts, or channels created by both the Chilkoot Inlet and Chilkat Inlet, we believe the potential for lateral spreading at this site is moderate.

6 GEOTECHNICAL DISCUSSION

The geotechnical design and construction concerns at the site include loose, liquefiable, thawed, granular alluvial soils and compressible, frost-susceptible fine-grained soils.

We developed soil parameters to analyze primary and long-term consolidation settlement under the maintenance facility foundation. We conducted a settlement analysis using the Settle3 software developed by Rocscience (2021). Settle3 analyzes immediate settlement, and primary and secondary consolidation settlement under proposed foundations assuming various loading conditions. The analysis relies on soil layering, loading conditions, groundwater conditions, and soil stiffness to estimate settlement. In our analysis, we assumed allowable settlement of less than 2 inches.

The recommendations presented below have been developed to address these concerns. In summary:

- a surcharge load can be used to reduce the magnitude of total consolidation settlement after construction,
- a 4-foot-thick section of fill is recommended to attenuate some of the potential seismically induced differential settlement, and improve foundation performance as long-term consolidation settlement occurs,
- a reinforced mat foundation is recommended to improve foundation performance.

7 SITE PREPARATION RECOMMENDATIONS

We recommend different site preparation methods for the structures and pavement areas, as discussed below.

7.1 Surcharge

A surcharge load equivalent to 1,400 psf (or approximately 12 feet of sand) *can be* used to cause some consolidation settlement to occur prior to construction. Our analysis indicates that surcharging the site for a 12-month period will reduce the total anticipated consolidation (static) settlement to about 1.5 inches, and that surcharging the site for a 24-month period will reduce the total anticipated consolidation (static) settlement to less than 1 inch. We selected a surcharge load greater than the allowable soil bearing pressure for our analysis, which should limit the amount of anticipated consolidation settlement after construction.

We recommend constructing an approximately 12-foot-tall section of loose sand material with the top of the slope of the section beginning 10 feet outside the outside edge of the building footprint. The slope of the sand should follow all federal, state, and local government regulations.

We recommend monitoring the settlement by placing a steel plate on the ground surface prior to placing any sand. The plate should have a steel pipe connected perpendicularly and be at least 3 feet taller than the anticipated sand height. We recommend a professional surveyor obtain the horizontal location and elevation of the top of the pipe before placing sand, after placement of all sand, and at the completion of the surcharge loading timeframe at a minimum.

We note that the use of wick drains would accelerate the consolidation process and may increase the effective depth. Wick drains were not recommended; however, due to the increased expense to install compared to the potential benefit to the project.

Surcharging the site is not a requirement, but we believe it will provide long term benefit to the project in that it will reduce some of the potential future consolidation settlement. Shannon & Wilson should be retained to determine whether the settlement observed is consistent with the recommendations in this report.

7.2 Maintenance Facility and Refueling Station Site Preparation

Based on the soil conditions encountered in our borings, we recommend the maintenance facility be founded on a reinforced concrete mat foundation. Plastic clay and silt were observed to depths of up to 17 feet bgs and we anticipate the removal of this material would require excavating several feet below the water table which would likely require the use of shoring and dewatering. Our recommended approach limits the excavation depth, and does not require dewatering.

We recommend excavation to the minimum depth required to place 4 feet of compacted structural fill below the bottom of the foundation. Soft, saturated soils were encountered just below the existing fill, which ranged in depth from 4 feet to 6 feet below existing ground surface. We recommend leaving at least 12 inches of the existing fill in place; therefore, finished grade of the area may need to be raised 1 to 2 feet above the existing elevation. The base of the excavation should be planar and should extend at least 4 feet beyond the outside edge of the foundation. All wood debris and organics (i.e. rootlets, peat, organic soil) should be removed from the base of the excavation.

Seasonally frozen soils should be thawed at least 2 feet below the bottom of the excavation prior to proof rolling, compacting, or placing fill. We recommend a geotechnical engineer from our firm observe the bottom of excavation prior to placing fill, to determine whether soils in the base of excavation meet the intent of our recommendations.

Frost transitions should be used between the structure excavation and the pavement area excavations for utilities, and between new and existing pavement sections per the recommendations provided below.

After existing soils are removed and before placement of the structural fill, the base of the excavation should be uniformly and systematically proof rolled with a large, self-propelled compactor. Due to the fine-grained nature of anticipated subgrade soils, proof rolling should be done in static mode (i.e. without the use of vibratory compactors). If the subgrade is not adequately compacted, it will be difficult for the contractor to meet compaction requirements in the first lift of fill; however, due to natural changes in the gradation and composition of the subgrade, we do not recommend a specific percent compaction, as it is difficult to accurately test and verify.

We recommend installing geotextile separator fabric consistent with the recommendations in Section 9.6 on the subgrade prior to placing the first lift of structural fill.

7.3 Gravel Parking Areas and Driveways

Site preparation for the gravel parking areas and driveways should include excavating to a depth sufficient to establish the recommended pavement section (Exhibit 8-1). All organics, wood debris, or deleterious materials encountered in the base of excavation should be removed.

The base of the excavation should be uniformly and systematically proof rolled with a large, self-propelled compactor. There is no specific compaction requirement for the subgrade proof rolling; however, compaction should be sufficient so the compaction requirement for the aggregate fill is met for the first lift and all subsequent lifts. If soils are soft, wet, or pumping, proof rolling should be accomplished in static mode (without the use of vibration).

Seasonally frozen soils should be thawed at least 2 feet below the bottom of the excavation prior to proof rolling, compacting, or placing fill. We recommend a geotechnical engineer from our firm observe the bottom of excavations prior to placing fill, to determine whether soils in the base of excavation meet the intent of our recommendations. Compressible organic soil, organic debris (wood or roots), and deleterious materials should be removed from the base of the excavation.

We recommend installing geotextile separator fabric consistent with the recommendations in Section 9.6 on the compacted subgrade prior to placing the first lift of structural fill.

7.4 Excavation Slopes

Excavation slopes should be the responsibility of the contractor and their Competent Person because they are on site every day and have control over the work. We recommend all excavations be sufficiently sloped or shored to provide a stable bank. The work should be accomplished in general accordance with applicable local, state, and federal standards. For planning purposes, we recommend you assume unsupported excavation slopes will be no steeper than 1.5 horizontal to 1 vertical (1.5H:1V). In loose or soft, silty and organic soils below the water table, slopes may need to be flatter than 4H:1V for stability. It is also important to note temporary excavation slopes may initially stand steep but slough and cave as they dry out, particularly when equipment is operated nearby. Similarly, steep cuts made in seasonally frozen ground can become unstable upon thawing.

7.5 Frost Transitions

Beneath pavement structures, we recommend gradual transitions between the sections of structural fill and utility trench crossings and adjacent frost-susceptible subgrade soils. These transitions serve to spread out differential movements due to frost-heaving. Frost transitions should be provided by sloping the interface between structural fill and adjacent frost-susceptible subgrade soils no steeper than 5H:1V. Where the top of direct-bury utility line bedding is less than one foot below the bottom of a pavement structure, and adjacent subgrade soils outside the trench are non-frost-susceptible, frost transitions shall begin at the base of the trench, as seen in Exhibit 7-1. All other frost transitions shall start at the base of the structural fill (subbase or prepared subgrade) or a maximum depth of 60 inches, whichever is less. We do not recommend constructing frost transitions where the thickness of adjoining pavement structures, or pavement structures and utilities, differ by less than 6 inches.

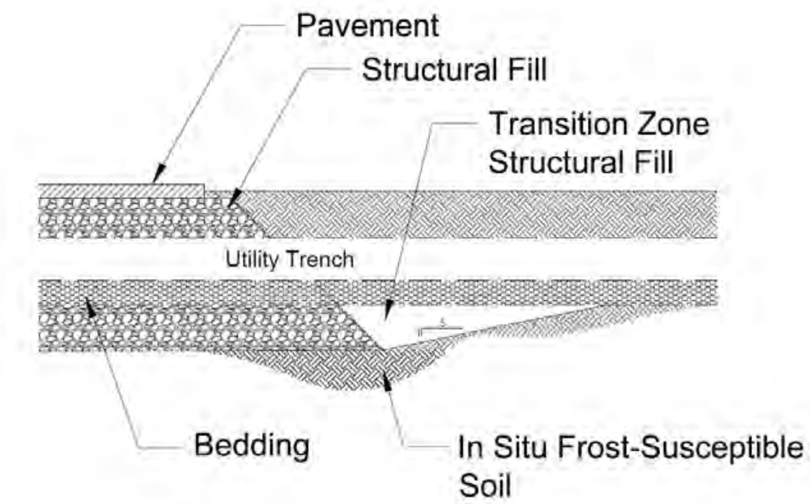

Exhibit 7-1: Utility Trench Transition

7.6 Drainage and Grading

During the construction of the project, the ground surface near the open excavation should be sloped away to reduce the water flowing into the excavation. The addition of water to soils in the excavation may reduce the stability of the slopes, as well as raise the moisture content of the subgrade to a point where it is difficult to compact to the required density.

Final grading should be designed to limit infiltration of water into the soils beneath the foundation and structure.

8 RECOMMENDATIONS

For purposes of our recommendations, it was necessary for us to assume that the results of the explorations are representative of conditions across the site. However, subsurface conditions should be expected to vary. We may need to revise our recommendations during construction if different conditions are encountered.

We have prepared our design recommendations for the structure foundation based on:

- The limitations of our approved scope, schedule, and budget described in our proposal dated January 17, 2023.
- Our understanding of the project and information provided by RESPEC and DOT&PF.
- Site and subsurface conditions we observed during our site visit and in the borings as they existed during our subsurface explorations.
- The result of testing performed on samples we collected from the borings.

When the designer develops additional information about final foundation configurations or other factors, the recommendations presented herein may need to be revised. Shannon & Wilson should be made aware of the revised or additional information so that we can revise our recommendations if necessary.

Our primary design concern for the foundation is settlement associated with consolidation of deep, soft, and compressible fine-grained soil. It is our opinion that the site may also experience differential settlement during or after seismic activity due to liquefaction of the loose, granular, and saturated soils encountered in our borings.

8.1 Modulus of Subgrade Reaction

Modulus of subgrade reaction can be used to evaluate the support of the subgrade soil underlying a pavement structure, or a structure foundation. An in-situ modulus of subgrade reaction can be obtained by conducting a field plate bearing test. In the absence of on-site testing, empirical values are commonly used. For this project, we referenced Unified Facilities Criteria (UFC) 3-260-02 dated 30 June 2001, which is the U.S. Army Corps of Engineers guidance for airfield pavement design. This reference was used based on the extent of available data and long history of acceptance on public projects.

For pavement, we assumed the subgrade soil will consist of silts and high plasticity clay. Per Table 6-1 of UFC 3-260-02, a typical range for this soil type is 50 pounds per cubic inch (pci) to 150 pci. In the absence of site-specific testing, the UFC recommends an assumed value of 50 pci.

For structures, we assume they will be founded on a section of compacted structural fill. The section of structural fill supporting the building will be at least 48 inches thick, and reinforced with geotextiles; therefore, we believe an improved modulus of subgrade reaction is appropriate for the structure. We believe assuming the modified modulus will be consistent with that of silty and clayey gravels is conservative for this site. Table 6-1 of the UFC recommends an assumed value of 250 pci for these types of soils.

8.2 Maintenance Facility Foundation Recommendations

Fine-grained, soft, and potentially compressible soils will remain below the planned structural fill section. As mentioned above, surcharging the site will reduce the magnitude of anticipated consolidation settlement after construction. Additionally, a thick section of reinforced gravel fill and a continuously reinforced foundation will improve building performance. Additionally, because the site soils are relatively poor, a reinforced mat foundation is planned for the new building.

The proposed maintenance facility can be founded on a reinforced mat foundation bearing on a minimum 4-foot section of compacted structural fill. Structural fill should be placed consistent with the recommendations in Section 9.1. Geotextile reinforcement meeting the requirements of Section 9.7 should be placed every 12 to 16 inches or as needed to install no less than two layers between the separator fabric and the top of structural fill. The top layer should be at least 18 inches below the top of the structural fill section to allow for placement of the thickened-edge slab.

We recommend the foundation bear a minimum 12 inches below adjacent finished grade elevation. The allowable bearing capacity is limited based on allowable settlement of less than 2 inches. We recommend a maximum allowable static bearing pressure of 1,200 pounds per square foot (psf).

The recommended allowable bearing capacity will not eliminate seismically induced settlements but should limit the total magnitude should an earthquake of sufficient magnitude and/or duration cause liquefaction during the life of the structure(s).

Along exterior walls, we recommend placing a minimum 2 inches of rigid-board insulation suitable for direct burial against the vertical portion of the foundation. The minimum thermal resistance in hour-feet-degrees Fahrenheit per British thermal unit (hr-ft-°F/Btu) of the insulation should be 5 per inch, or the insulation should be thickened to maintain equal or greater total thermal resistance values.

Insulation will rapidly degrade if subjected to fuel, motor oil, solvents, and other petroleum-based chemicals. Therefore, a protective membrane should be installed over and around the insulation to protect the insulation from petroleum-based chemicals if there is a possibility that it may come into contact with these chemicals in the future.

Due to the frost-susceptible nature of the subgrade soils, drainage away from the structures and prevention of surface water ponding or infiltration into soils below the structures is necessary.

Figure 3 presents our foundation recommendations.

8.3 Refueling Station Foundation Recommendations

We understand the tank will be above ground and a concrete slab foundation is preferred. We recommend the concrete slab be constructed on a minimum 4-foot section of compacted structural fill. We recommend the base of the concrete foundation be founded at least 12 inches below the ground surface. Surficial fill should be sloped to drain away from the foundation.

A reinforced concrete slab can be designed for an allowable bearing capacity of 1,500 psf. The allowable bearing capacity is limited to prevent a punching-type failure during liquefaction of the saturated soils underlying the site. We anticipate long-term consolidation settlement may approach 2.5 inches, all of which could be differential. The reinforced concrete slab and the connections to the tank should be designed to accommodate movement due to potential differential settlement.

The magnitude of anticipated consolidation settlement could be reduced by surcharging the area, and/or by reducing the bearing pressure.

8.4 Pavement Recommendations

We understand most of the driveway and parking areas will be aggregate surfaced. We also anticipate some drive surfaces (i.e. building apron, refueling area) may be paved. We developed pavement recommendations for aggregate, asphalt, and Portland cement surfaces.

We evaluated pavement section requirements using the Alaska Flexible Pavement Design Software, Version 2.0 developed by DOT&PF, along with the United States Army Corps of Engineers (USACE) Pavement-Transportation Computer Assisted Structural Engineering (PCASE) Version 7.0.4 design software. We conducted the analyses using soil parameters developed from our understanding of subsurface conditions, function, and potential traffic assumed for the analysis.

The pavement section recommendations were developed using the reduced subgrade strength (RSS) method due to the natural silty soils encountered at the site, anticipated deep seasonal frost penetration, and the potential for subgrade variability.

Exhibit 8-1 summarizes the pavement sections we developed for this project.

Exhibit 8-1: Minimum Recommended Pavement Sections

Pavement Area	Type	PCC (inches)	AC (inches)	E-1 (inches)	D-1 Base (inches)	Subbase[1] (inches)
Driveways / Parking	Aggregate	--	--	6	--	12
Driveways / Parking	AC	--	4	--	4	24
Building Vehicle Aprons	PCC	6	--	--	--	24

NOTES:

a NFS material meeting the requirements for structural fill except that the largest particle size should be 2 inches, placed in maximum 8-inch lifts (after compaction) compacted to 95% of modified proctor.

8.5 Utilities

The project may include placement of or modification to underground sewage, water, and power lines. Subsurface conditions indicate conventional techniques and equipment could be used to construct direct-bury utilities. Trenches for utility lines must be wide enough and deep enough to allow placement and compaction of fill. We recommend a minimum trench width of 3 feet or 3 pipe diameters, whichever is greater. Direct-bury utility lines should be placed on a minimum of 6 inches of compacted fill meeting the gradation and compaction requirements for structural fill or bedding material. Bedding material should consist of granular, rounded to sub-rounded material meeting the gradation requirements in this report.

Prior to laying utility lines or placing fill, any seasonally frozen soils below the base of the trench should be removed or allowed to thaw a minimum of 2 feet below the base prior to backfilling. The base of the trench should then be uniformly and systematically proof-compacted with at least 4 passes of a large, self-propelled compactor. The equipment and number of passes shall be left to the contractor, so the contractor is able to achieve the required compaction in the first lift of fill.

After laying the pipe or conduit in the trench and establishing line and grade, the trench should be backfilled up to the pipe spring line, on both sides of the pipe at the same time, with bedding material sufficiently compacted so the pipe does not shift or lift. The trench should then be filled to 6 inches above the top of the pipe with additional bedding material. The surface of the bedding material should be compacted with a vibratory plate compactor. Compaction of the bedding material should be accomplished so as not to damage the pipe or pipe insulation.

We do not recommend a minimum compaction requirement for bedding material. Compaction of the bedding material should be left to the contractor, so the required compaction in the first lift of overlying fill is achievable. We do not recommend compaction testing within 1 foot of the pipe.

Abrupt changes in soil conditions should be avoided beneath paved areas to reduce the effects of frost-action. Beneath paved areas, native soils from the trench excavation should be used to fill the trench between the top of the trench bedding and the bottom of the pavement structure. Native fills should meet the requirements for, and be compacted in accordance with, recommendations for nonstructural fill in this report.

Where the top of bedding is less than 1 foot below the bottom of a pavement structure and adjacent soils in the trench are frost-susceptible, we recommend frost-transition zones as described in this report. Structural fill should be used to fill the trench between the top of the trench bedding and the base of the pavement structure. Pavement structures include the prepared subgrade, subbase, base, and surface courses.

9 MATERIALS

9.1 Structural Fill

Structural fill should consist of unfrozen non-frost susceptible (NFS) gravelly sand or sandy gravel meeting the following gradation limits, after compaction:

Exhibit 9-1: Structural Fill Gradation Limits

Size	Percent Passing
3-inch	100
No. 4 sieve	30-60
No. 200 sieve	0-6

We understand soils meeting this gradation requirement are available from local sources as pit-run sand and gravel and/or crushed or processed material. Structural fill should be well-graded, NFS material containing no more than 60 percent by weight passing the number 4 sieve, and not more than 6 percent by weight passing the number 200 sieve. NFS structural fill should contain little to no organics; less than 3 percent by weight of particles finer than 0.02 mm.

Structural fill, compacted with large, self-propelled vibratory rollers, should be placed in layers not exceeding 12 inches in loose lift height. Should small vibratory compactors or small plate compactors be used, the allowable loose lift height is 8 inches. The material in each layer should be compacted to achieve a density of at least 95 percent of the maximum dry density, or as recommended, based on the Modified Proctor moisture-density relationship consistent with ASTM D1557 *Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Modified Effort*. ASTM D6938, *Standard Test Method for In-Place Density and Water Content of Soil and Soil-Aggregate by Nuclear Methods (Shallow Depth)*, should be used to determine in-place densities.

The fill should consist of unfrozen materials and be placed at above-freezing air temperatures. If previously placed fill freezes, for instance overnight, the frozen material should be excavated and wasted or allowed to thaw and be recompacted prior to the placement of additional fill.

9.2 Aggregate Surface Course (E-1)

We recommend aggregate surface course meet the following DOT&PF gradation for E-1, after compaction:

Exhibit 9-2: Surface Course (E-1) Gradation Limits

Size	Percent Passing
1-inch	100
3/4-inch	70-100
3/8-inch	50-85
No. 4 sieve	35-65
No. 8 sieve	20-50
No. 50 sieve	15-30
No. 200 sieve	8-15

Surface material should consist of crushed stone or crushed gravel, free from clay balls, vegetative matter, or other deleterious material. Surface course, compacted with large, self-propelled vibratory rollers, should be placed in layers not exceeding 6 inches in loose lift height. The material in each layer should be compacted to achieve a density of at least 98 percent of the maximum dry density, or as recommended, based on the ASTM D1557. ASTM D6938 should be used to determine in-place densities.

Aggregate for Surface Course should meet the requirements of Table 703-1 found in the 2020 DOT&PF Standard Specifications:

Exhibit 9-3: Aggregate Quality Properties for Base and Surface Course

Property	Base Course	Surface Course	Test Method
L.A. Wear, %	50, max	45, max	AASHTO T 96
Degradation Value	45, min	45, min	ATM 313
Fracture, %	70, min	70, min, 1 face	ATM 305
Liquid Limit	--	35, max	ATM 204
Plastic Index	6, max	10, max	ATM 205
Sodium Sulfate Loss, %	9, max (5 cycles)	9, max (5 cycles)	AASHTO T 104

9.3 Base Course (D-1)

Base course material should meet the DOT&PF specification for D-1 base course and have the following gradation limits after compaction:

Exhibit 9-4: Base Course (D-1) Gradation Limits

Size	Percent Passing
1-inch	100
3/4-inch	70-100
3/8-inch	50-80
No. 4 sieve	35-65
No. 8 sieve	20-50
No. 50 sieve	6-30
No. 200 sieve	0-6

Base course material should be placed in lifts not exceeding 6 inches in loose lift height. Crushed aggregate base course should be compacted to at least 98% of the maximum density at optimum moisture content as determined by Modified Proctor ASTM D1557. ASTM D6938 should be used to determine in-place densities.

Aggregate for base course should meet the requirements of Table 703-1 found in the 2020 DOT&PF Standard Specifications, presented above in Exhibit 9-3.

9.4 Nonstructural Fill and Backfill

Nonstructural fill may be used to fill or shape unpaved areas for landscaping, or as backfill above utilities outside foundation bearing zones. Nonstructural fill may consist of fine-grained soils from the excavation; however, the fill should not contain topsoil, organics, or deleterious matter. Maximum loose lift height for nonstructural fill should not exceed 12 inches, although with fine-grained soils, a maximum 8-inch lift thickness may be necessary to achieve the desired compaction with smooth drum rollers or small plate compactors. Nonstructural fill should be compacted to at least 90 percent of the maximum dry density obtained from ASTM D1557. ASTM D6938 should be used to determine in-place densities. Drying or wetting of the soil may be necessary to obtain compaction.

The compaction of sandy fine-grained soils with small vibratory compactors, particularly small hand-operated equipment, is expected to be difficult. If hand-operated compactors (jumping jacks or walk-behind plate compactors) are used to compact excavated materials, the loose lift thickness should not exceed 6 inches.

9.5 Bedding Material

Bedding material should consist of unfrozen, granular, rounded to sub-rounded material meeting the following gradation limits:

Exhibit 9-5: Bedding Material Gradation Limits

Size	Percent Passing
1-inch	100
No. 200 sieve	0-6

Clean sand and gravel from on-site building excavations or borrow sources meeting these gradation requirements may also be used as bedding material. The use of angular or crushed gravel is not recommended for bedding. Bedding material should be placed and compacted in accordance with the recommendations given in the utilities section of this report.

9.6 Geotextile Separator

We recommend the geotextile separator fabric conform to the requirements of the American Association for State Highway and Transportation Officials (AASHTO) M288-00 for a Class 2 geotextile with an elongation greater than or equal to 50 percent. The Class 2 geotextile should conform to the requirements of Table 3 Separation Geotextile Property Requirements in AASHTO M 288, except the minimum permittivity of the fabric should be 0.05 per second. The geotextile separator should also have an apparent opening size equal to or between the No. 70 and No. 100 U.S. Standard Sieve as determined by ASTM D4751 *Standard Test Method for Determining Apparent Opening Size of a Geotextile*. Class 2 geotextile may be joined either by sewing or by overlapping. If the material is joined by overlapping, the material should be overlapped a minimum of 24 inches. The installation of the geotextile fabric should conform to the requirements of Appendix A3 of AASHTO M288.

9.7 Geotextile Reinforcement

We recommend geotextile reinforcement meet the requirements of DOT&PF 2020 Standard Specifications Table 729-1, Type 1. Reinforcement geotextile shall meet the survivability requirements presented in Table 729-2, and the physical requirements shown in Table 729-3. The tables are provided below, for reference.

Exhibit 9-6: DOT&PF Table 729-1

Property	Test Method	Units	Type 1 Requirement[a]
Grab Tensile	ASTM D4632	lb	200/200
Grab Elongation	ASTM D4632	% (MD)	10
Wide Width Tensile	ASTM D4595	lb/in (ultimate)	200/200
Wide Width Tensile	ASTM D5495	lb/in (@ 5% strain)	100/100
Seam Breaking Strength	ASTM D4632	lb/in	180
Puncture	ASTM D6241	lb	500
Trapezoidal Tear	ASTM D4533	lb	100
AOS	ASTM D4751	U.S. sieve size	#30[b]
Permittivity	ASTM D4491	sec^{-1}	0.20
Flow Rate	ASTM D4491	$gal/min/ft^2$	10

NOTES:

a Minimum Average Roll Values (MARV) in machine direction (MD) / cross-machine direction (XD) unless otherwise specified.

b Maximum average roll value.

Exhibit 9-7: DOT&PF Table 729-2

Property	Test Method	Units	Type 1 Requirement
Ultimate Multi-Rib Tensile Strength[a]	ASTM D6637	lb/ft	1230
Junction Strength[a]	ASTM D7737	lb	25
Ultraviolet Stability (Retained Strength)	ASTM D4335	%	50% after 500 hours of exposure

NOTES:

a Minimum Average Roll Values (MARV) in any rib direction.

Exhibit 9-8: DOT&PF Table 729-3

Property	Test Method	Units	Type 1 Requirement[a]
2% Tensile Strength[a]	ASTM D6637	lb/ft	≥400
5% Tensile Strength[a]	ASTM D6637	lb/ft	≥800
Percent Open Area	COE, CW-02215	%	50-80
Aperture Size[b]	Direct measure	in	0.5-3.0

NOTES:

b Minimum Average Roll Values (MARV) in machine and cross-machine directions.

c Measured as the spacing between parallel ribs.

10 CLOSURE AND LIMITATIONS

This report was prepared for the exclusive use of our client and their representatives for evaluating the site as it relates to the geotechnical aspects discussed herein. The conclusions and interpretation contained in this report are based on site conditions as they existed during our site visit and/or explorations. It is assumed that the exploratory borings are representative of the subsurface conditions throughout the site, i.e., the subsurface conditions everywhere are not significantly different from those disclosed by the explorations.

If, during construction, subsurface conditions different from those encountered in these explorations are observed or appear to be present, Shannon & Wilson, Inc. should be advised at once so that these conditions can be reviewed. If there is a substantial lapse of time between the submittal of this report and the start of work at the site, or if conditions have changed due to natural causes or construction operations at or adjacent to the site, it is recommended that this report be reviewed to determine the applicability of the conclusions considering the changed conditions and time lapse.

Unanticipated soil conditions are commonly encountered and cannot fully be determined by merely taking soil samples or advancing test holes. Such unexpected conditions frequently require that additional expenditures be made to attain a properly constructed project. Therefore, some contingency fund is recommended to accommodate such potential extra costs. Please read the Important Information section at the back of this report to understand how our services may help reduce project risks.

We recommend that we be retained to review those portions of the plans and specifications pertaining to earthwork to determine if they are consistent with our recommendations. In addition, we should be retained to observe construction, particularly site excavations, preparation of subgrade, compaction of structural fill, and also to make field observations as may be necessary.

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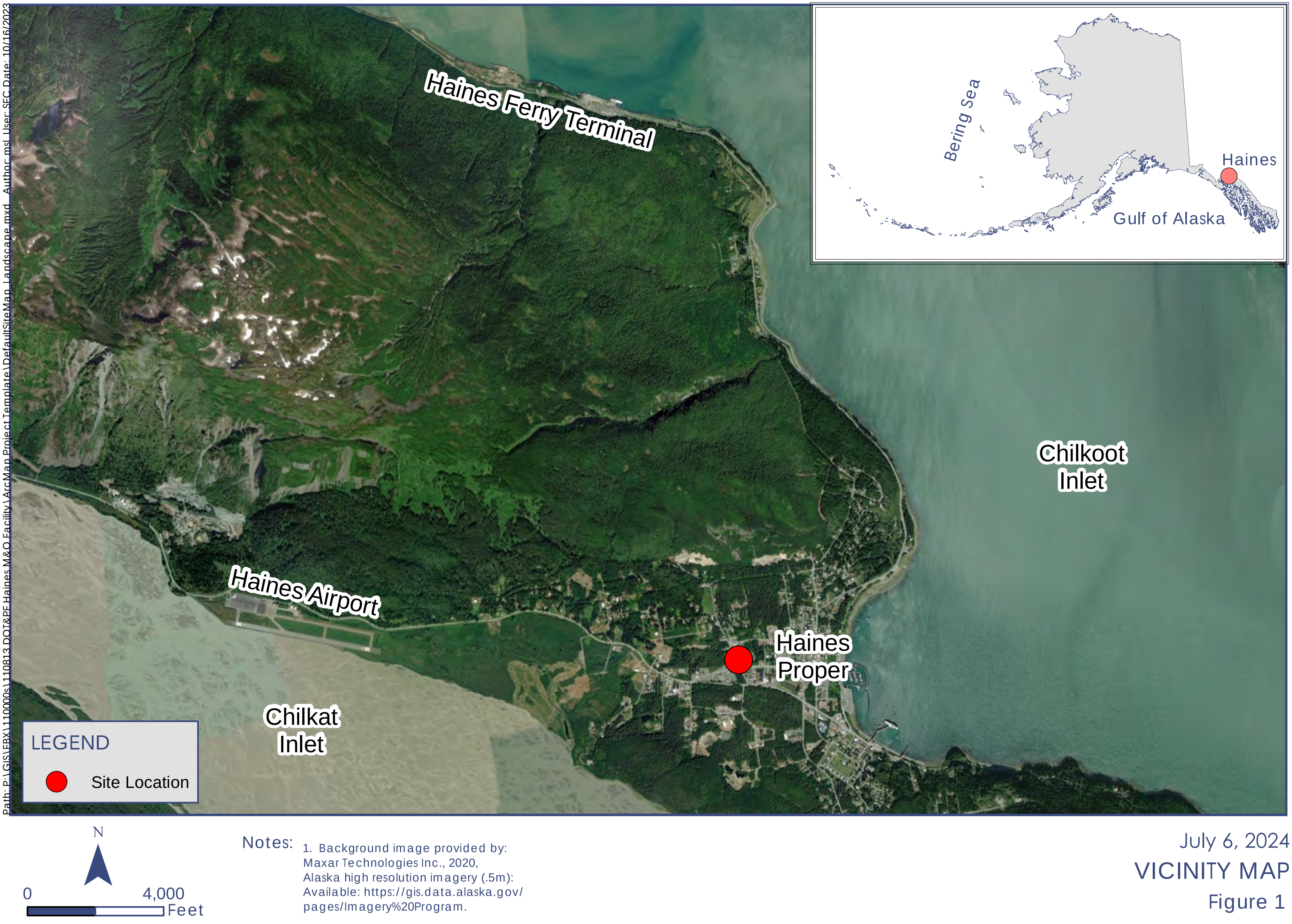

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Notes:
1. Background image provided by: Maxar Technologies Inc., 2020, Alaska high resolution imagery (.5m): Available: https://gis.data.alaska.gov/pages/Imagery%20Program.

July 6, 2024

VICINITY MAP

Figure 1

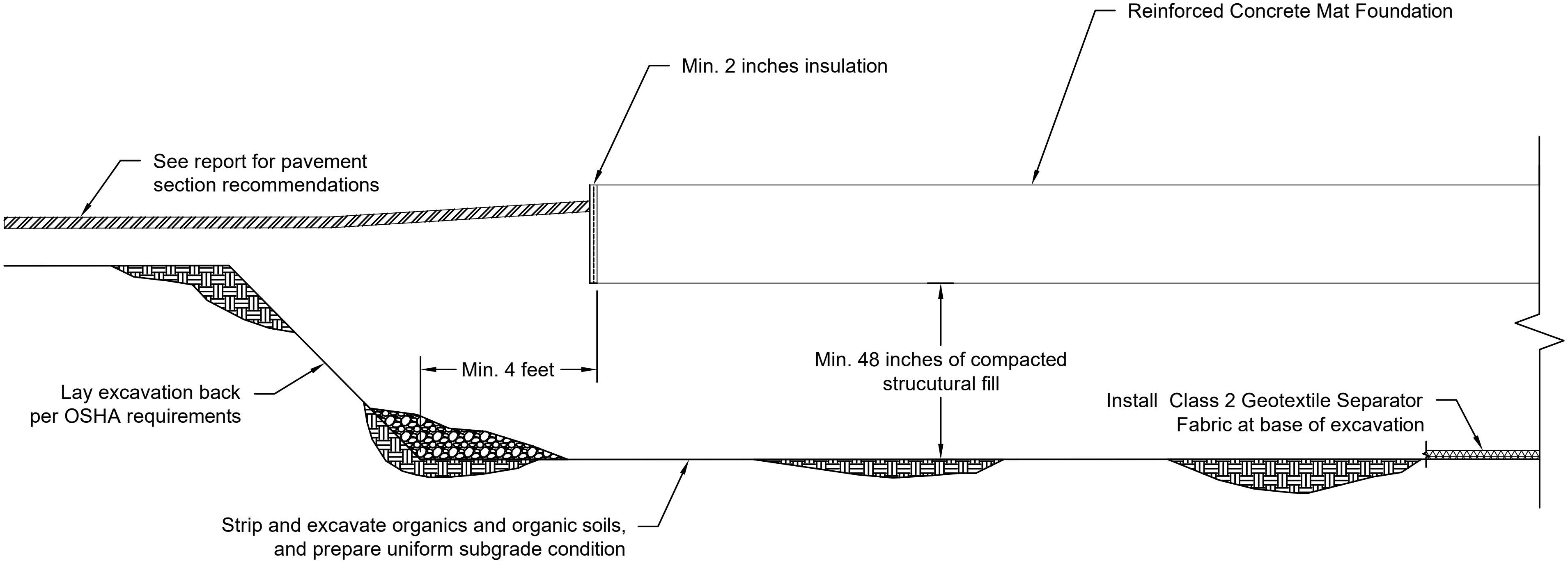
N
0
Feet

Notes:

1. Imagery provided by Maxar Products. Dynamic Mosaic © 2020 Maxar Technologies Inc., Alaska Geospatial Office, USGS, available: https://geoportal.alaska.gov/portal/home/item.html?id=b6cdda a872a8458085e99defa723f168, accessed January 2023

July 6, 2024
SITE MAP
Figure 2

Path: P:\GIS\FBX\110000s\110813 DOT&PF Haines M&O Facility\ArcMap Project Template\DefaultSiteMap.mxd Author: msl User: WAP Date: 3/8/2024



Legend

- Structural Fill (see report for details)
- Nonstrucural Fill or Ashpalt Pavement Section (see report for details)
- In Situ Soils
- Insulation (see report for details)
- Class 2 Geotextile Separator Fabric (see report for details)

Not to Scale

Haines ADOT&PF M&O Facility
Haines, Alaska

FROST-PROTECTED FOUNDATION RECOMMENDATIONS

July 6, 2024 110813-001

SHANNON & WILSON, INC. GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

Figure 3

Figure 3

APPENDIX A: SOIL CLASSIFICATION AND BORING LOGS

Appendix A
Soil Classification and Boring Logs

CONTENTS

- Figure A-1 Soil Description and Log Key
- Figures A-2 through A-7 Logs of Boring 23-01 through 23-06

Shannon & Wilson, Inc. (S&W), uses a soil identification system modified from the Unified Soil Classification System (USCS). Elements of the USCS and other definitions are provided on this and the following pages. Soil descriptions are based on visual-manual procedures (ASTM D2488) and laboratory testing procedures (ASTM D2487), if performed.

S&W INORGANIC SOIL CONSTITUENT DEFINITIONS

CONSTITUENT[2]	FINE-GRAINED SOILS (50% or more fines)[1]	COARSE-GRAINED SOILS (less than 50% fines)[1]
Major	***Silt, Lean Clay, Elastic Silt,*** or ***Fat Clay***[3]	***Sand*** or ***Gravel***[4]
Modifying (Secondary) Precedes major constituent	30% or more coarse-grained: ***Sandy*** or ***Gravelly***[4]	More than 12% fine-grained: ***Silty*** or ***Clayey***[3]
Minor Follows major constituent	15% to 30% coarse-grained: ***with Sand*** or ***with Gravel***[4]	5% to 12% fine-grained: ***with Silt*** or ***with Clay***[3]
	30% or more total coarse-grained *and* lesser coarse-grained constituent is 15% or more: ***with Sand*** or ***with Gravel***[5]	15% or more of a second coarse-grained constituent: ***with Sand*** or ***with Gravel***[5]

[1]All percentages are by weight of total specimen passing a 3-inch sieve.
[2]The order of terms is: *Modifying Major with Minor*.
[3]Determined based on behavior.
[4]Determined based on which constituent comprises a larger percentage.
[5]Whichever is the lesser constituent.

MOISTURE CONTENT TERMS

Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, from below water table

STANDARD PENETRATION TEST (SPT) SPECIFICATIONS

Hammer:	140 pounds with a 30-inch free fall. Rope on 6- to 10-inch-diam. cathead 2-1/4 rope turns, > 100 rpm NOTE: If automatic hammers are used, blow counts shown on boring logs should be adjusted to account for efficiency of hammer.
Sampler:	10 to 30 inches long Shoe I.D. = 1.375 inches Barrel I.D. = 1.5 inches Barrel O.D. = 2 inches
N-Value:	Sum blow counts for second and third 6-inch increments. Refusal: 50 blows for 6 inches or less; 10 blows for 0 inches.

NOTE: Penetration resistances (N-values) shown on boring logs are as recorded in the field and have not been corrected for hammer efficiency, overburden, or other factors.

PARTICLE SIZE DEFINITIONS

DESCRIPTION	SIEVE NUMBER AND/OR APPROXIMATE SIZE
FINES	< #200 (0.075 mm = 0.003 in.)
SAND Fine Medium Coarse	 #200 to #40 (0.075 to 0.4 mm; 0.003 to 0.02 in.) #40 to #10 (0.4 to 2 mm; 0.02 to 0.08 in.) #10 to #4 (2 to 4.75 mm; 0.08 to 0.187 in.)
GRAVEL Fine Coarse	 #4 to 3/4 in. (4.75 to 19 mm; 0.187 to 0.75 in.) 3/4 to 3 in. (19 to 76 mm)
COBBLES	3 to 12 in. (76 to 305 mm)
BOULDERS	> 12 in. (305 mm)

RELATIVE DENSITY / CONSISTENCY

COHESIONLESS SOILS		COHESIVE SOILS	
N, SPT, BLOWS/FT.	RELATIVE DENSITY	N, SPT, BLOWS/FT.	RELATIVE CONSISTENCY
< 4	Very loose	< 2	Very soft
4 - 10	Loose	2 - 4	Soft
10 - 30	Medium dense	4 - 8	Medium stiff
30 - 50	Dense	8 - 15	Stiff
> 50	Very dense	15 - 30	Very stiff
		> 30	Hard

WELL AND BACKFILL SYMBOLS

Bentonite Cement Grout	Surface Cement Seal
Bentonite Grout	Asphalt or Cap
Bentonite Chips	Slough
Silica Sand	Inclinometer or Non-perforated Casing
Perforated or Screened Casing	Vibrating Wire Piezometer

PERCENTAGES TERMS [1, 2]

Trace	< 5%
Few	5 to 10%
Little	15 to 25%
Some	30 to 45%
Mostly	50 to 100%

[1]Gravel, sand, and fines estimated by mass. Other constituents, such as organics, cobbles, and boulders, estimated by volume.

[2]Reprinted, with permission, from ASTM D2488 - 09a Standard Practice for Description and Identification of Soils (Visual-Manual Procedure), copyright ASTM International, 100 Barr Harbor Drive, West Conshohocken, PA 19428. A copy of the complete standard may be obtained from ASTM International, www.astm.org.

Haines Maintenance & Operations Station
Project No. 57183-B
Haines, Alaska

SOIL DESCRIPTION AND LOG KEY

March 2024 110813-001

SHANNON & WILSON, INC.
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FIG. A-1
Sheet 1 of 3

UNIFIED SOIL CLASSIFICATION SYSTEM (USCS)
(Modified From USACE Tech Memo 3-357, ASTM D2487, and ASTM D2488)

MAJOR DIVISIONS			GROUP/GRAPHIC SYMBOL	TYPICAL IDENTIFICATIONS
COARSE-GRAINED SOILS *(more than 50% retained on No. 200 sieve)*	Gravels *(more than 50% of coarse fraction retained on No. 4 sieve)*	Gravel *(less than 5% fines)*	GW	Well-Graded Gravel; Well-Graded Gravel with Sand
			GP	Poorly Graded Gravel; Poorly Graded Gravel with Sand
		Silty or Clayey Gravel *(more than 12% fines)*	GM	Silty Gravel; Silty Gravel with Sand
			GC	Clayey Gravel; Clayey Gravel with Sand
	Sands *(50% or more of coarse fraction passes the No. 4 sieve)*	Sand *(less than 5% fines)*	SW	Well-Graded Sand; Well-Graded Sand with Gravel
			SP	Poorly Graded Sand; Poorly Graded Sand with Gravel
		Silty or Clayey Sand *(more than 12% fines)*	SM	Silty Sand; Silty Sand with Gravel
			SC	Clayey Sand; Clayey Sand with Gravel
FINE-GRAINED SOILS *(50% or more passes the No. 200 sieve)*	Silts and Clays *(liquid limit less than 50)*	Inorganic	ML	Silt; Silt with Sand or Gravel; Sandy or Gravelly Silt
			CL	Lean Clay; Lean Clay with Sand or Gravel; Sandy or Gravelly Lean Clay
		Organic	OL	Organic Silt or Clay; Organic Silt or Clay with Sand or Gravel; Sandy or Gravelly Organic Silt or Clay
	Silts and Clays *(liquid limit 50 or more)*	Inorganic	MH	Elastic Silt; Elastic Silt with Sand or Gravel; Sandy or Gravelly Elastic Silt
			CH	Fat Clay; Fat Clay with Sand or Gravel; Sandy or Gravelly Fat Clay
		Organic	OH	Organic Silt or Clay; Organic Silt or Clay with Sand or Gravel; Sandy or Gravelly Organic Silt or Clay
HIGHLY-ORGANIC SOILS	Primarily organic matter, dark in color, and organic odor		PT	Peat or other highly organic soils (see ASTM D4427)

NOTE: No. 4 size = 4.75 mm = 0.187 in.; No. 200 size = 0.075 mm = 0.003 in.

NOTES

1. Dual symbols *(symbols separated by a hyphen, i.e., SP-SM, Sand with Silt)* are used for soils with between 5% and 12% fines or when the liquid limit and plasticity index values plot in the CL-ML area of the plasticity chart. Graphics shown on the logs for these soil types are a combination of the two graphic symbols (e.g., SP and SM).
2. Borderline symbols *(symbols separated by a slash, i.e., CL/ML, Lean Clay to Silt; SP-SM/SM, Sand with Silt to Silty Sand)* indicate that the soil properties are close to the defining boundary between two groups.

Haines Maintenance & Operations Station
Project No. 57183-B
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SOIL DESCRIPTION AND LOG KEY

March 2024 110813-001

SHANNON & WILSON, INC.
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FIG. A-1
Sheet 2 of 3

GRADATION TERMS

Poorly Graded	Narrow range of grain sizes present or, within the range of grain sizes present, one or more sizes are missing (Gap Graded). Meets criteria in ASTM D2487, if tested.
Well-Graded	Full range and even distribution of grain sizes present. Meets criteria in ASTM D2487, if tested.

CEMENTATION TERMS[1]

Weak	Crumbles or breaks with handling or slight finger pressure.
Moderate	Crumbles or breaks with considerable finger pressure.
Strong	Will not crumble or break with finger pressure.

PLASTICITY[2]

DESCRIPTION	VISUAL-MANUAL CRITERIA	APPROX. PLASTICITY INDEX RANGE
Nonplastic	A 1/8-in. thread cannot be rolled at any water content.	< 4
Low	A thread can barely be rolled and a lump cannot be formed when drier than the plastic limit.	4 to 10
Medium	A thread is easy to roll and not much time is required to reach the plastic limit. The thread cannot be rerolled after reaching the plastic limit. A lump crumbles when drier than the plastic limit.	10 to 20
High	It takes considerable time rolling and kneading to reach the plastic limit. A thread can be rerolled several times after reaching the plastic limit. A lump can be formed without crumbling when drier than the plastic limit.	> 20

ADDITIONAL TERMS

Mottled	Irregular patches of different colors.
Bioturbated	Soil disturbance or mixing by plants or animals.
Diamict	Nonsorted sediment; sand and gravel in silt and/or clay matrix.
Cuttings	Material brought to surface by drilling.
Slough	Material that caved from sides of borehole.
Sheared	Disturbed texture, mix of strengths.

PARTICLE ANGULARITY AND SHAPE TERMS[1]

Angular	Sharp edges and unpolished planar surfaces.
Subangular	Similar to angular, but with rounded edges.
Subrounded	Nearly planar sides with well-rounded edges.
Rounded	Smoothly curved sides with no edges.
Flat	Width/thickness ratio > 3.
Elongated	Length/width ratio > 3.

ACRONYMS AND ABBREVIATIONS

ATD	At Time of Drilling
Diam.	Diameter
Elev.	Elevation
ft.	Feet
FeO	Iron Oxide
gal.	Gallons
Horiz.	Horizontal
HSA	Hollow Stem Auger
I.D.	Inside Diameter
in.	Inches
lbs.	Pounds
MgO	Magnesium Oxide
mm	Millimeter
MnO	Manganese Oxide
NA	Not Applicable or Not Available
NP	Nonplastic
O.D.	Outside Diameter
OW	Observation Well
pcf	Pounds per Cubic Foot
PID	Photo-Ionization Detector
PMT	Pressuremeter Test
ppm	Parts per Million
psi	Pounds per Square Inch
PVC	Polyvinyl Chloride
rpm	Rotations per Minute
SPT	Standard Penetration Test
USCS	Unified Soil Classification System
q_u	Unconfined Compressive Strength
VWP	Vibrating Wire Piezometer
Vert.	Vertical
WOH	Weight of Hammer
WOR	Weight of Rods
Wt.	Weight

STRUCTURE TERMS[1]

Interbedded	Alternating layers of varying material or color with layers at least 1/4-inch thick; singular: bed.
Laminated	Alternating layers of varying material or color with layers less than 1/4-inch thick; singular: lamination.
Fissured	Breaks along definite planes or fractures with little resistance.
Slickensided	Fracture planes appear polished or glossy; sometimes striated.
Blocky	Cohesive soil that can be broken down into small angular lumps that resist further breakdown.
Lensed	Inclusion of small pockets of different soils, such as small lenses of sand scattered through a mass of clay.
Homogeneous	Same color and appearance throughout.

[1]Reprinted, with permission, from ASTM D2488 - 09a Standard Practice for Description and Identification of Soils (Visual-Manual Procedure), copyright ASTM International, 100 Barr Harbor Drive, West Conshohocken, PA 19428. A copy of the complete standard may be obtained from ASTM International, www.astm.org.

[2]Adapted, with permission, from ASTM D2488 - 09a Standard Practice for Description and Identification of Soils (Visual-Manual Procedure), copyright ASTM International, 100 Barr Harbor Drive, West Conshohocken, PA 19428. A copy of the complete standard may be obtained from ASTM International, www.astm.org.

Haines Maintenance & Operations Station
Project No. 57183-B
Haines, Alaska

SOIL DESCRIPTION AND LOG KEY

March 2024 110813-001

SHANNON & WILSON, INC.
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FIG. A-1
Sheet 3 of 3

Total Depth:	*41.5 ft.*	Northing:	~	Drilling Method:	*Hollow Stem Auger*	Hole Diam.:	*8 in.*
Top Elevation:	~	Easting:	~	Drilling Company:	*GeoTek Alaska, Inc.*	Rod Diam.:	
Vert. Datum:		Station:	~	Drill Rig Equipment:	*Geoprobe 7718DT*	Hammer Type:	*Automatic*
Horiz. Datum:	*WGS84*	Offset:	~	Other Comments:			

SOIL DESCRIPTION

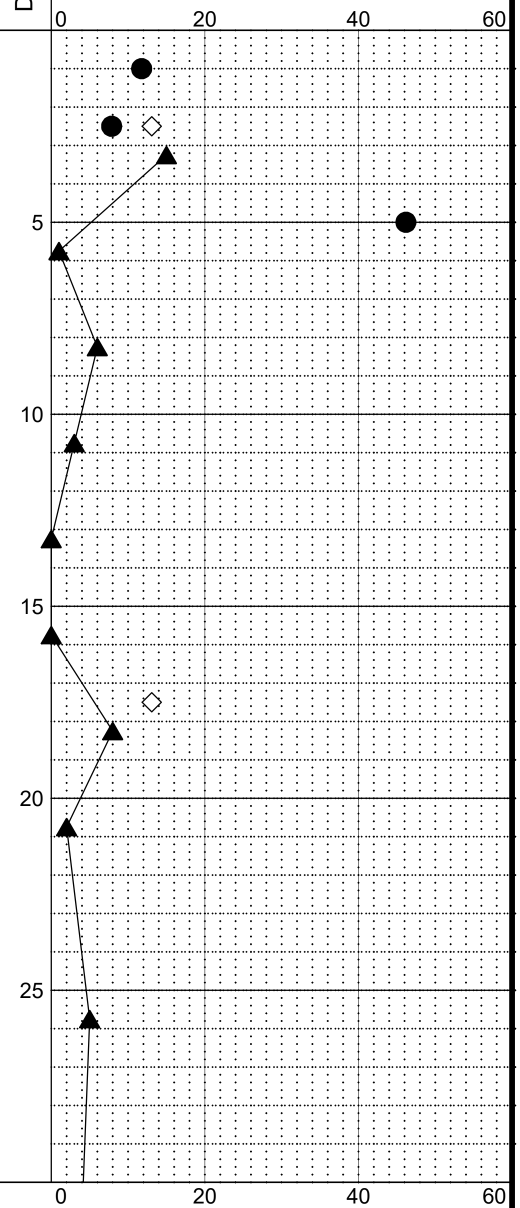
Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between material types, and the transition may be gradual.

Soil Description	Depth, ft.	Samples	Ground Water
Medium-dense, brown, *Poorly Graded Gravel with Silt and Sand and Cobbles (GP-GM)*; moist; subangular to subrounded gravel; few organics, wood debris. FILL	4.0	1 (G), 2	
Very soft, gray, *Elastic Silt (MH)*; moist; few sand; medium plasticity; few black organics, 3-inch wood debris at 6.2 feet bgs.	7.0	3	During Drilling ∇
Soft to medium-stiff, dark brown, *Silt with Sand (ML)*; wet; some organics, wood debris; organic odor.	12.0	4, 5	
Very soft, brown to gray, *Elastic Silt (MH)*; wet; medium plasticity; some organics, wood debris, rootlets; organic odor.	17.0	6, 7	
Loose to very loose, gray, *Silty Sand with Gravel (SM)*; wet; subangular to subrounded gravel, gravel decreasing with depth; sand coarser with depth; few organics, wood debris, rootlets; organic odor.	22.5	8, 9	
Loose, gray, *Clayey Sand (SC)*; wet; few gravel; trace organics.	27.5	10	
Soft, gray; *Lean Clay (CL)*; trace gravel; medium plasticity.			

CONTINUED NEXT SHEET

PENETRATION RESISTANCE (blows/foot)

▲ Hammer Wt. & Drop: *340 lbs / 30 inches*



◇ % Fines (<0.075mm)
● % Water Content

LEGEND

- * Sample Not Recovered
- G Grab Sample
- 3" O.D. Split Spoon Sample
- ∇ Ground Water Level ATD

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.
4. The hole location was measured from existing site features and should be considered approximate.

Haines Maintenance & Operations Station
Project No. 57183-B
Haines, Alaska

LOG OF BORING 23-01

March 2024 110813-001

SHANNON & WILSON, INC.
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FIG. A-2
Sheet 1 of 2

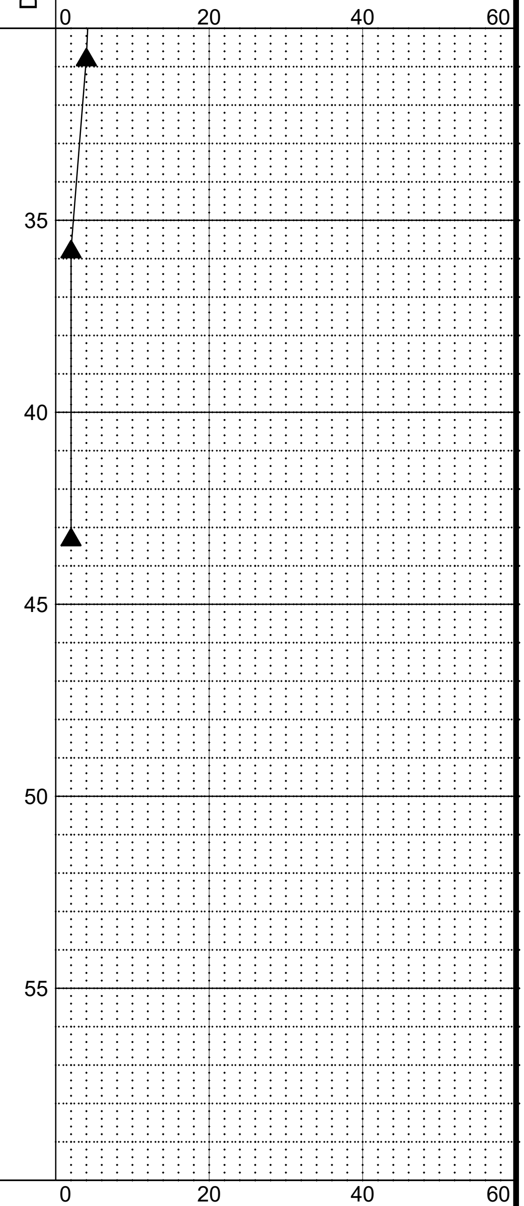
REV 3 - Approved for Submittal

Total Depth:	41.5 ft.	Northing:	~	Drilling Method:	Hollow Stem Auger	Hole Diam.:	8 in.
Top Elevation:	~	Easting:	~	Drilling Company:	GeoTek Alaska, Inc.	Rod Diam.:	
Vert. Datum:		Station:	~	Drill Rig Equipment:	Geoprobe 7718DT	Hammer Type:	Automatic
Horiz. Datum:	WGS84	Offset:	~	Other Comments:			

SOIL DESCRIPTION *Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between material types, and the transition may be gradual.*	Depth, ft.	Symbol	Samples	Ground Water
Soft, gray; *Lean Clay (CL)*; trace gravel; medium plasticity.			11, 12, 13	
BOTTOM OF BORING COMPLETED 10/9/2023	44.0			

PENETRATION RESISTANCE (blows/foot)

▲ Hammer Wt. & Drop: 340 lbs / 30 inches



LEGEND

- * Sample Not Recovered
- Grab Sample
- 3" O.D. Split Spoon Sample
- Ground Water Level ATD

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.
4. The hole location was measured from existing site features and should be considered approximate.

Haines Maintenance & Operations Station
Project No. 57183-B
Haines, Alaska

LOG OF BORING 23-01

March 2024 — 110813-001

SHANNON & WILSON, INC.
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FIG. A-2
Sheet 2 of 2

REV 3 - Approved for Submittal

Total Depth:	41.5 ft.	Northing:	~	Drilling Method:	Hollow Stem Auger	Hole Diam.:	8 in.
Top Elevation:	~	Easting:	~	Drilling Company:	GeoTek Alaska, Inc.	Rod Diam.:	
Vert. Datum:		Station:	~	Drill Rig Equipment:	Geoprobe 7718DT	Hammer Type:	Automatic
Horiz. Datum:	WGS84	Offset:	~	Other Comments:			

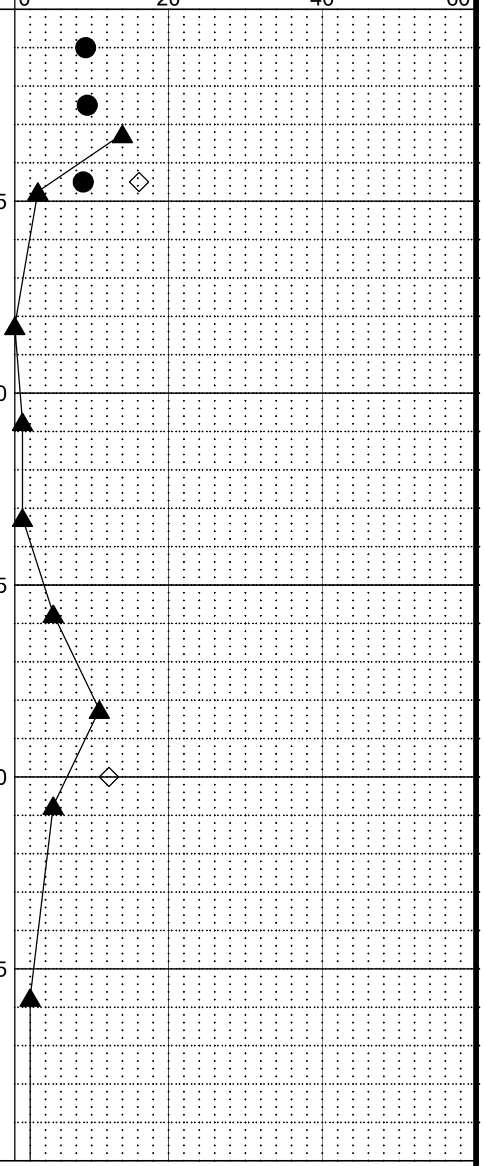
SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between material types, and the transition may be gradual.

Soil Description	Depth, ft.	Samples	Ground Water
Medium-dense to very loose, brown, *Poorly Graded Gravel with Sand and Cobbles (GP)*; moist; subangular to subrounded gravel, fractured gravel; sand decreasing with depth; trace organics; fractured cobbles. FILL	5.0	1 (G), 2, 3a	
Very soft, brown to gray, *Elastic Silt (MH)*; wet; some organics, wood debris, peat; trace black organics; iron staining.	14.5	3b, 4, 5, 6	During Drilling
Loose, dark gray to gray, *Clayey Gravel with Sand (GC)*; wet; subangular to subrounded gravel.	17.0	7	
Medium-dense, gray, *Poorly Graded Sand with Gravel (SP)*; wet; subangular to subrounded gravel; coarse sand.	19.5	8	
Loose, dark gray to gray, *Clayey Gravel with Sand (GC)*; wet; subangular to subrounded gravel; clay increasing with depth.	22.5	9	
Soft, gray, *Lean Clay (CL)*; wet; medium plasticity; trace gravel, gravel increasing with depth.		10	

PENETRATION RESISTANCE (blows/foot)

▲ Hammer Wt. & Drop: 340 lbs / 30 inches



CONTINUED NEXT SHEET

◇ % Fines (<0.075mm)
● % Water Content

LEGEND

- * Sample Not Recovered
- G Grab Sample
- 3" O.D. Split Spoon Sample
- ▽ Ground Water Level ATD

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.
4. The hole location was measured from existing site features and should be considered approximate.

Haines Maintenance & Operations Station
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LOG OF BORING 23-02

March 2024 110813-001

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FIG. A-3
Sheet 1 of 2

REV 3 - Approved for Submittal

Total Depth:	*41.5 ft.*	Northing:	~	Drilling Method:	*Hollow Stem Auger*	Hole Diam.:	*8 in.*
Top Elevation:	~	Easting:	~	Drilling Company:	*GeoTek Alaska, Inc.*	Rod Diam.:	
Vert. Datum:		Station:	~	Drill Rig Equipment:	*Geoprobe 7718DT*	Hammer Type:	*Automatic*
Horiz. Datum:	*WGS84*	Offset:	~	Other Comments:			

SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between material types, and the transition may be gradual.

Depth, ft. | Symbol | Samples | Ground Water | Depth, ft.

PENETRATION RESISTANCE (blows/foot)

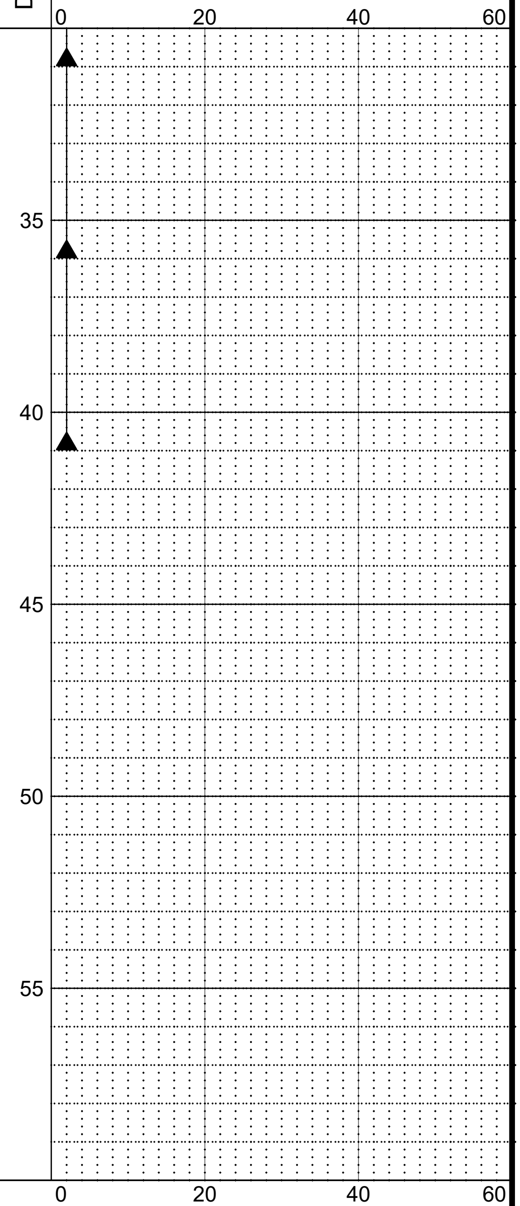
▲ Hammer Wt. & Drop: *340 lbs / 30 inches*

Soft, gray, *Lean Clay (CL)*; wet; medium plasticity; trace gravel, gravel increasing with depth.

Samples: 11, 12, 13

41.5

BOTTOM OF BORING
COMPLETED 10/9/2023



◇ % Fines (<0.075mm)
● % Water Content

LEGEND

* Sample Not Recovered
Ground Water Level ATD
Grab Sample
3" O.D. Split Spoon Sample

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.
4. The hole location was measured from existing site features and should be considered approximate.

Haines Maintenance & Operations Station
Project No. 57183-B
Haines, Alaska

LOG OF BORING 23-02

March 2024 110813-001

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FIG. A-3
Sheet 2 of 2

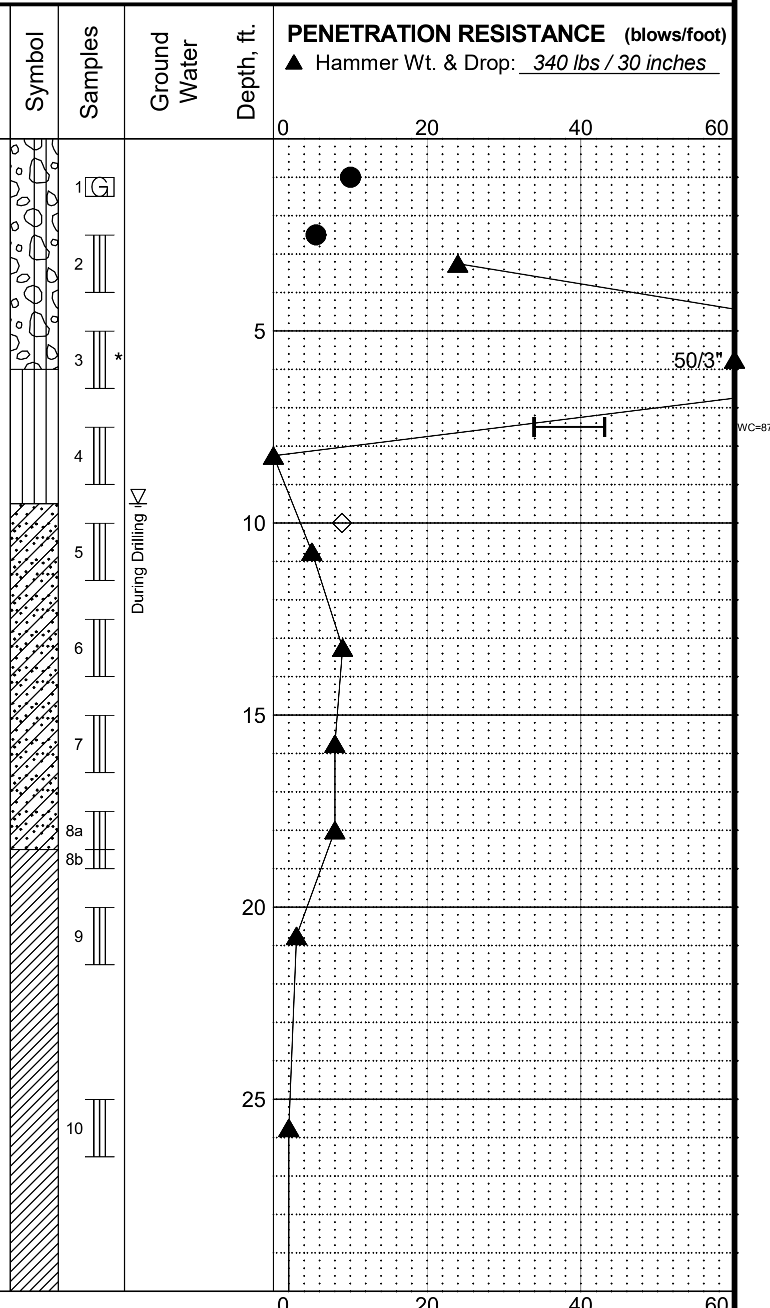
REV 3 - Approved for Submittal

Total Depth:	41.5 ft.	Northing:	~	Drilling Method:	Hollow Stem Auger	Hole Diam.:	8 in.
Top Elevation:	~	Easting:	~	Drilling Company:	GeoTek Alaska, Inc.	Rod Diam.:	
Vert. Datum:		Station:	~	Drill Rig Equipment:	Geoprobe 7718DT	Hammer Type:	Automatic
Horiz. Datum:	WGS84	Offset:	~	Other Comments:			

SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between material types, and the transition may be gradual.

Soil Description	Depth, ft.
Medium-dense, brown, *Poorly Graded Gravel with Silt and Sand (GP-GM)*; moist; subangular to subrounded gravel, fractured gravel; trace organics; fractured cobbles. FILL	6.0
Very soft, gray-brown, *Silt (ML)*; moist; medium plasticity; few organics.	9.5
Loose, brown to gray, *Clayey Sand with Gravel (SC)*; wet; fine gravel, subangular to subrounded gravel, gravel increasing with depth; sand coarser with depth; organic odor.	18.5
Soft, gray, *Lean Clay (CL)*; wet; trace gravel, gravel decreasing with depth; trace sand, sand decreasing with depth; medium plasticity.	



CONTINUED NEXT SHEET

LEGEND

- * Sample Not Recovered
- Grab Sample
- 3" O.D. Split Spoon Sample
- Ground Water Level ATD

◇ % Fines (<0.075mm)
● % Water Content
Plastic Limit ├─●─┤ Liquid Limit
Natural Water Content

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.
4. The hole location was measured from existing site features and should be considered approximate.

Haines Maintenance & Operations Station
Project No. 57183-B
Haines, Alaska

LOG OF BORING 23-03

March 2024 110813-001

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FIG. A-4
Sheet 1 of 2

REV 3 - Approved for Submittal

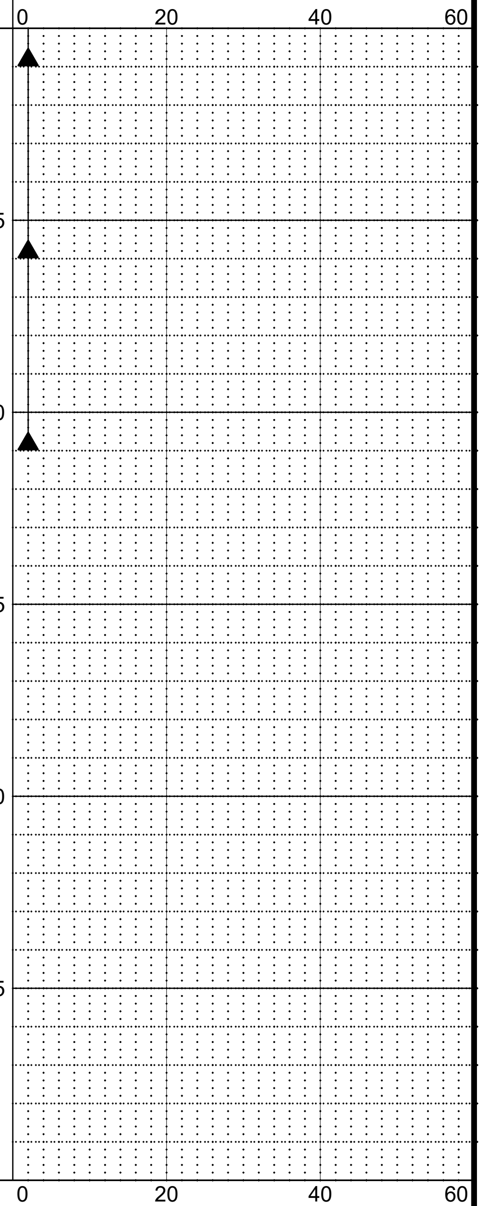
Total Depth:	*41.5 ft.*	Northing:	~	Drilling Method:	*Hollow Stem Auger*
Top Elevation:	~	Easting:	~	Drilling Company:	*GeoTek Alaska, Inc.*
Vert. Datum:		Station:	~	Drill Rig Equipment:	*Geoprobe 7718DT*
Horiz. Datum:	*WGS84*	Offset:	~	Other Comments:	

Hole Diam.:	*8 in.*
Rod Diam.:	
Hammer Type:	*Automatic*

SOIL DESCRIPTION	Depth, ft.	Symbol	Samples	Ground Water	Depth, ft.
Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between material types, and the transition may be gradual.					
Soft, gray, *Lean Clay (CL)*; wet; trace gravel, gravel decreasing with depth; trace sand, sand decreasing with depth; medium plasticity.			11		
			12		35
			13		40
BOTTOM OF BORING COMPLETED 10/10/2023	41.5				
					45
					50
					55

PENETRATION RESISTANCE (blows/foot)

▲ Hammer Wt. & Drop: *340 lbs / 30 inches*



◇ % Fines (<0.075mm)

● % Water Content

Plastic Limit |—●—| Liquid Limit

Natural Water Content

LEGEND

* Sample Not Recovered

G Grab Sample

3" O.D. Split Spoon Sample

Ground Water Level ATD

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.
4. The hole location was measured from existing site features and should be considered approximate.

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Project No. 57183-B
Haines, Alaska

LOG OF BORING 23-03

March 2024 110813-001

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FIG. A-4
Sheet 2 of 2

REV 3 - Approved for Submittal

Total Depth:	21.5 ft.	Northing:	~	Drilling Method:	Hollow Stem Auger	Hole Diam.:	8 in.
Top Elevation:	~	Easting:	~	Drilling Company:	GeoTek Alaska, Inc.	Rod Diam.:	
Vert. Datum:		Station:	~	Drill Rig Equipment:	Geoprobe 7718DT	Hammer Type:	Automatic
Horiz. Datum:	WGS84	Offset:	~	Other Comments:			

SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between material types, and the transition may be gradual.

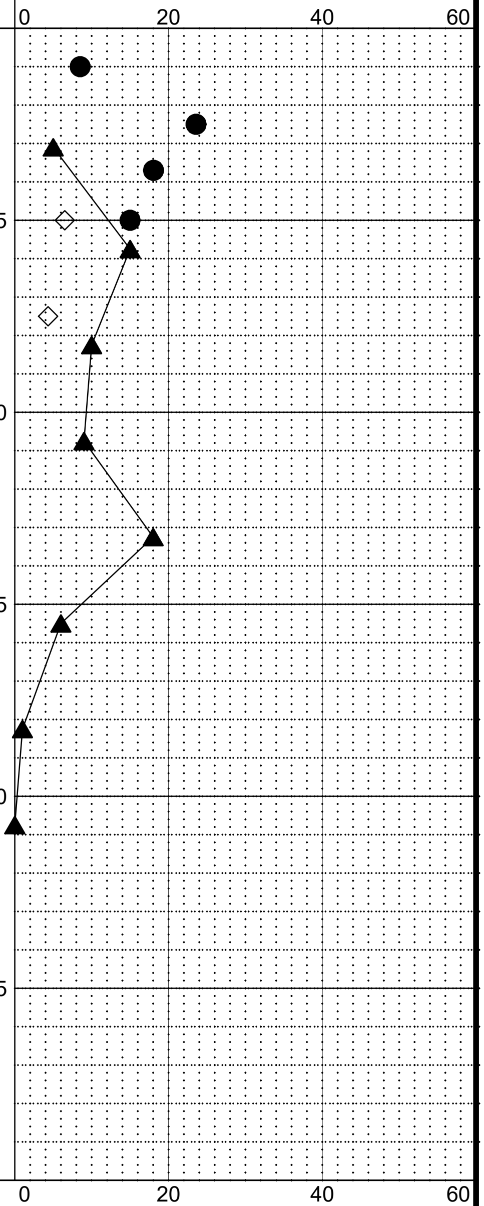
Soil Description	Depth, ft.	Samples
Brown, *Poorly Graded Gravel with Silt and Sand (GP-GM)*; moist; fine gravel. FILL	2.0	1 (G)
Medium-stiff, gray-brown, *Elastic Silt with Gravel (MH)*; moist; fractured gravel; few organics, peat.	3.7	2a, 2b
Medium-dense to loose, brown to red-brown to gray, *Poorly Graded Sand with Silt (SP-SM)*; moist to 4.5 feet bgs, then wet; subangular gravel, gravel increasing with depth.	16.0	3, 4, 5, 6, 7a, 7b
Very soft, gray, *Lean Clay (CL)*; wet; medium plasticity.	21.5	8, 9

BOTTOM OF BORING
COMPLETED 10/10/2023

Ground Water: During Drilling ∇

PENETRATION RESISTANCE (blows/foot)

▲ Hammer Wt. & Drop: 340 lbs / 30 inches



◇ % Fines (<0.075mm)
● % Water Content

LEGEND

- * Sample Not Recovered
- ∇ Ground Water Level ATD
- G Grab Sample
- 3" O.D. Split Spoon Sample

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.
4. The hole location was measured from existing site features and should be considered approximate.

Haines Maintenance & Operations Station
Project No. 57183-B
Haines, Alaska

LOG OF BORING 23-04

March 2024 110813-001

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FIG. A-5

REV 3 - Approved for Submittal

Total Depth:	26.5 ft.	Northing:	~	Drilling Method:	Hollow Stem Auger	Hole Diam.:	8 in.
Top Elevation:	~	Easting:	~	Drilling Company:	GeoTek Alaska, Inc.	Rod Diam.:	
Vert. Datum:		Station:	~	Drill Rig Equipment:	Geoprobe 7718DT	Hammer Type:	Automatic
Horiz. Datum:	WGS84	Offset:	~	Other Comments:			

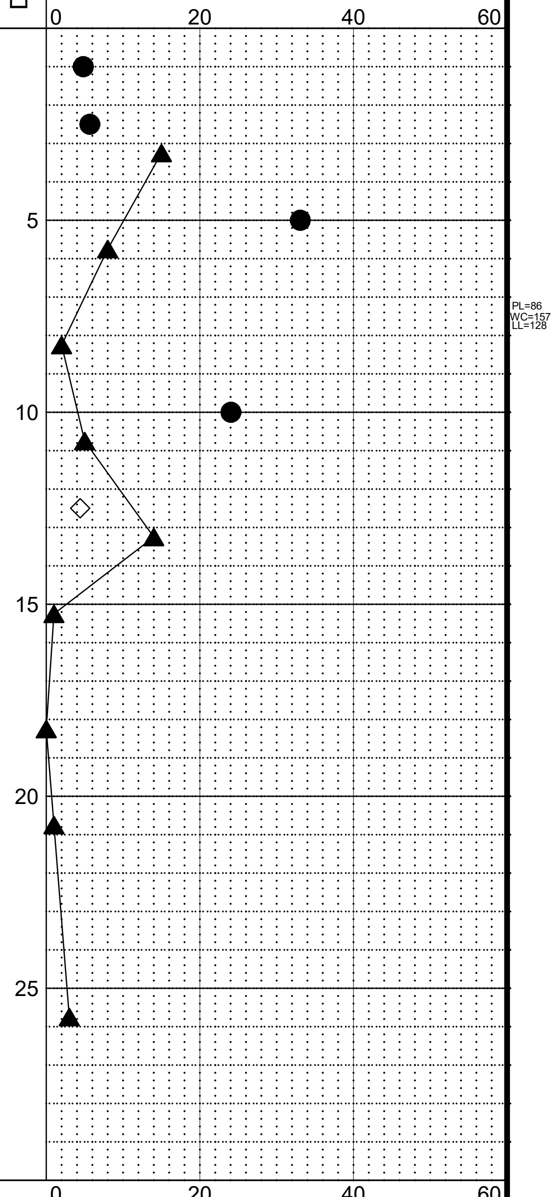
SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between material types, and the transition may be gradual.

Soil Description	Depth, ft.	Samples	Ground Water
Medium-dense to loose, brown, *Silty Sand with Gravel and Cobbles (SM)*; moist; subrounded gravel, gravel increasing with depth; trace organics, wood debris, peat; organics increasing with depth. FILL		1, 2	
Soft, brown, *Elastic Silt (MH)*; moist; medium plasticity; some organics, wood debris.	5.5	3, 4	
Very loose to medium-dense, gray, *Clayey Sand with Gravel (SC)*; moist to 12 feet bgs, then wet; few gravel, subangular to subrounded gravel, fractured gravel; sand increasing with depth; 6-inch wood debris at 10 feet bgs.	9.5	5, 6, 7a	During Drilling
Very soft to soft, gray, *Lean Clay (CL)*; wet; trace gravel; trace sand; medium plasticity.	15.5	7b, 8, 9, 10	
BOTTOM OF BORING COMPLETED 10/10/2023	26.5		

PENETRATION RESISTANCE (blows/foot)

▲ Hammer Wt. & Drop: 340 lbs / 30 inches



◇ % Fines (<0.075mm)

● % Water Content

Plastic Limit ├─●─┤ Liquid Limit

Natural Water Content

LEGEND

- * Sample Not Recovered
- G Grab Sample
- 3" O.D. Split Spoon Sample
- ⊻ Ground Water Level ATD

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.
4. The hole location was measured from existing site features and should be considered approximate.

Haines Maintenance & Operations Station
Project No. 57183-B
Haines, Alaska

LOG OF BORING 23-05

March 2024 110813-001

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FIG. A-6

REV 3 - Approved for Submittal

MASTER_LOG_E_ALASKA 110813-001.GPJ SHAN_WIL.GDT 3/8/24 Log: SFC Rev: WAP Typ: SFC

Total Depth:	21.5 ft.	Northing:	~	Drilling Method:	Hollow Stem Auger	Hole Diam.:	8 in.
Top Elevation:	~	Easting:	~	Drilling Company:	GeoTek Alaska, Inc.	Rod Diam.:	
Vert. Datum:		Station:	~	Drill Rig Equipment:	Geoprobe 7718DT	Hammer Type:	Automatic
Horiz. Datum:	WGS84	Offset:	~	Other Comments:			

SOIL DESCRIPTION

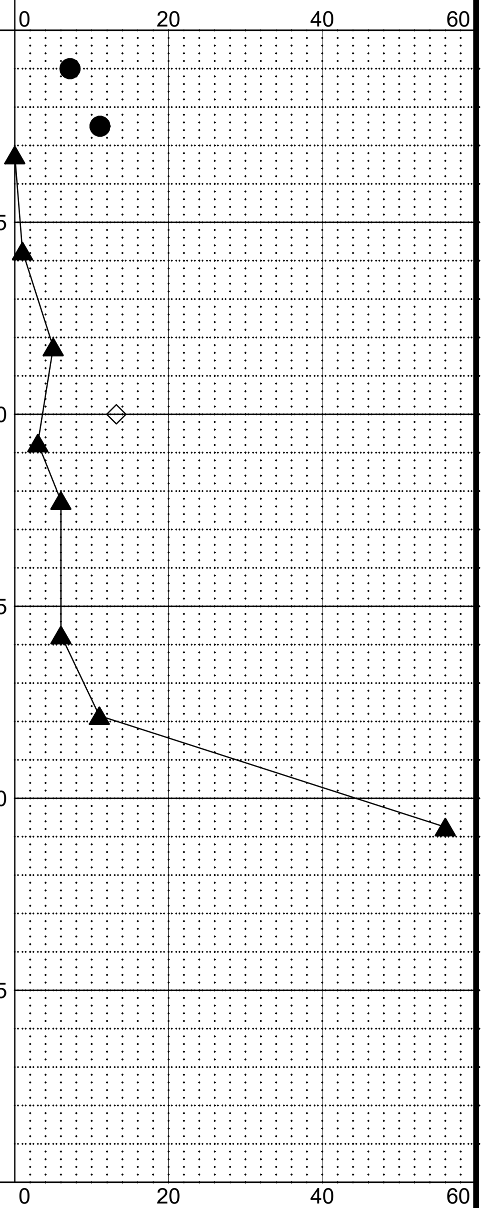
Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between material types, and the transition may be gradual.

Soil Description	Depth, ft.	Samples
Brown, *Silty Gravel with Sand (GM)*; moist; subangular to subrounded gravel; few organics. FILL	3.0	1 (G), 2
Very soft to medium-stiff, brown, *Elastic Silt (MH)*; moist to 6 feet bgs, then wet; trace gravel; some organics, wood debris, peat; 6-inch wood debris at 8.5 feet bgs and 12.0 feet bgs.	12.5	3, 4, 5, 6a
Loose to medium-dense, gray, *Poorly Graded Sand with Gravel (SP)*; wet; subangular to subrounded gravel, gravel decreasing with depth; fine sand; trace organics; 3-inch wood debris at 15 feet bgs.	18.2	6b, 7, 8a
Stiff, gray, *Lean Clay with Gravel (CL)*; wet; gravel increasing with depth.	19.5	8b
Dense, gray, *Poorly Graded Sand with Gravel (SP)*; wet; subangular to subrounded gravel, fractured gravel; trace organics.	21.5	9

BOTTOM OF BORING
COMPLETED 10/9/2023

Ground Water: During Drilling ▽

PENETRATION RESISTANCE (blows/foot)
▲ Hammer Wt. & Drop: 340 lbs / 30 inches



◇ % Fines (<0.075mm)
● % Water Content

LEGEND

- * Sample Not Recovered
- G Grab Sample
- 3" O.D. Split Spoon Sample
- ▽ Ground Water Level ATD

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.
4. The hole location was measured from existing site features and should be considered approximate.

Haines Maintenance & Operations Station
Project No. 57183-B
Haines, Alaska

LOG OF BORING 23-06

March 2024 110813-001

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FIG. A-7

MASTER_LOG_E_ALASKA 110813-001.GPJ SHAN_WIL.GDT 3/8/24 Log: SFC Rev: WAP Typ: SFC

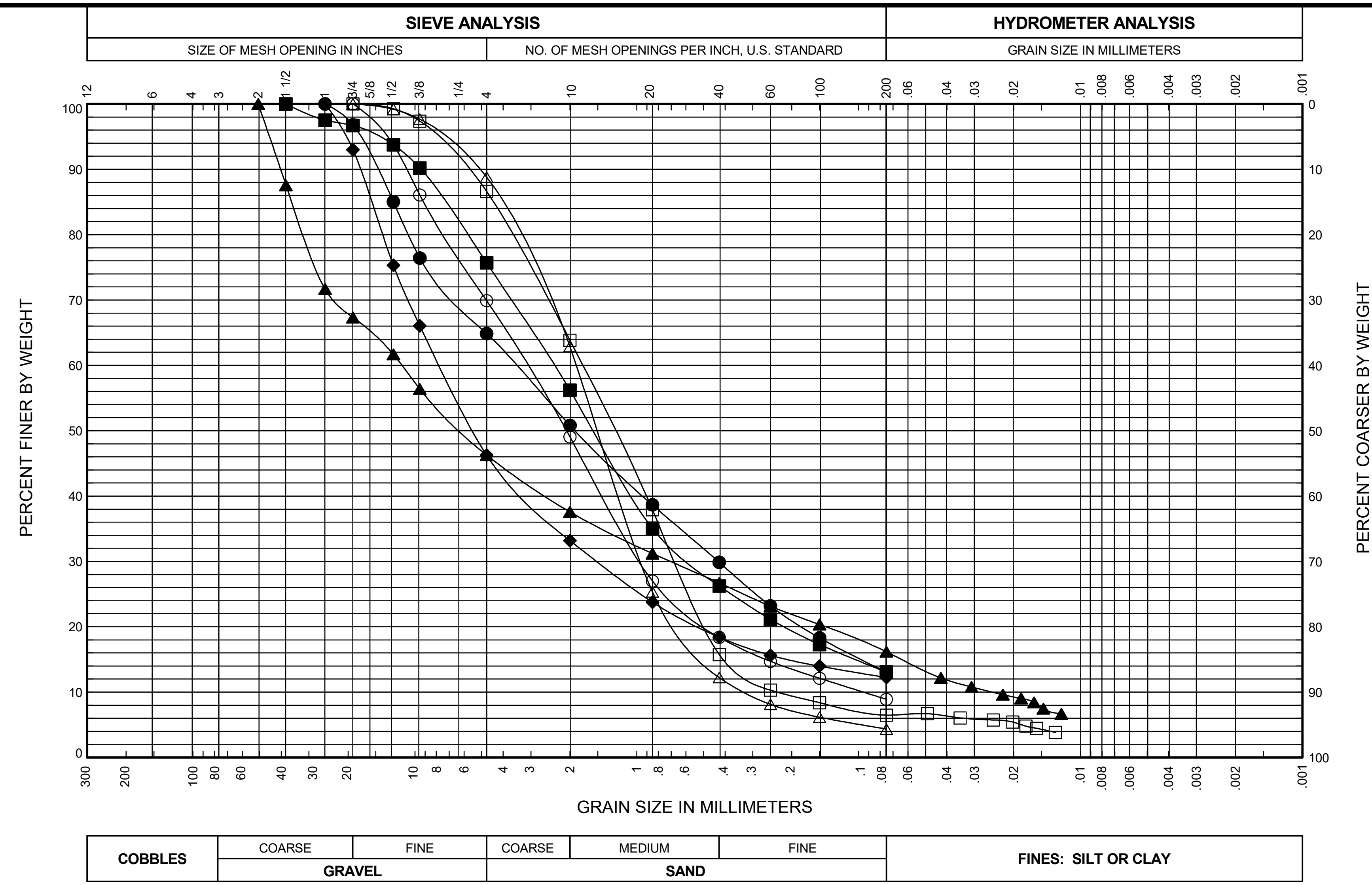
REV 3 - Approved for Submittal

APPENDIX B: LABORATORY TEST RESULTS

Appendix B

Laboratory Test Results

CONTENTS



BORING AND SAMPLE NO.	DEPTH (feet)	U.S.C.S. SYMBOL	SOIL CLASSIFICATION	COBBLE %	GRAVEL %	SAND %	FINES %	NAT. W.C. %	TEST BY	REVIEW BY	ASTM STD
● 23-01, S-2*	2.5	SM	Silty Sand with Gravel		35	52	13.1	7.9	AKL	LDW	C136
■ 23-01, S-8*	17.5	SM	Silty Sand with Gravel		24	63	13.1		AKL	LDW	C136
▲ 23-02, 3a*	4.5	GM	Silty Gravel with Sand		54	30	16.2	8.9	SPW	LDW	D422
◆ 23-02, S-9*	20.0	GM	Silty Gravel with Sand		54	34	12.2		AKL	LDW	C136
○ 23-03, S-5*	10.0	SW-SM	Well-Graded Sand with Silt and Gravel		30	61	8.9		AKL	LDW	C136
□ 23-04, S-3	5.0	SW-SM	Well-Graded Sand with Silt		13	80	6.5	15.0	SPW	LDW	D422
△ 23-04, S-4	7.5	SP	Poorly Graded Sand		11	84	4.3		AKL	LDW	C136

** Sample specimen weight did not meet required minimum mass for ASTM test method.*

Haines Maintenance & Operations Station
Project No. 57183-B
Haines, Alaska

GRAIN SIZE DISTRIBUTION

March 2024 110813-001

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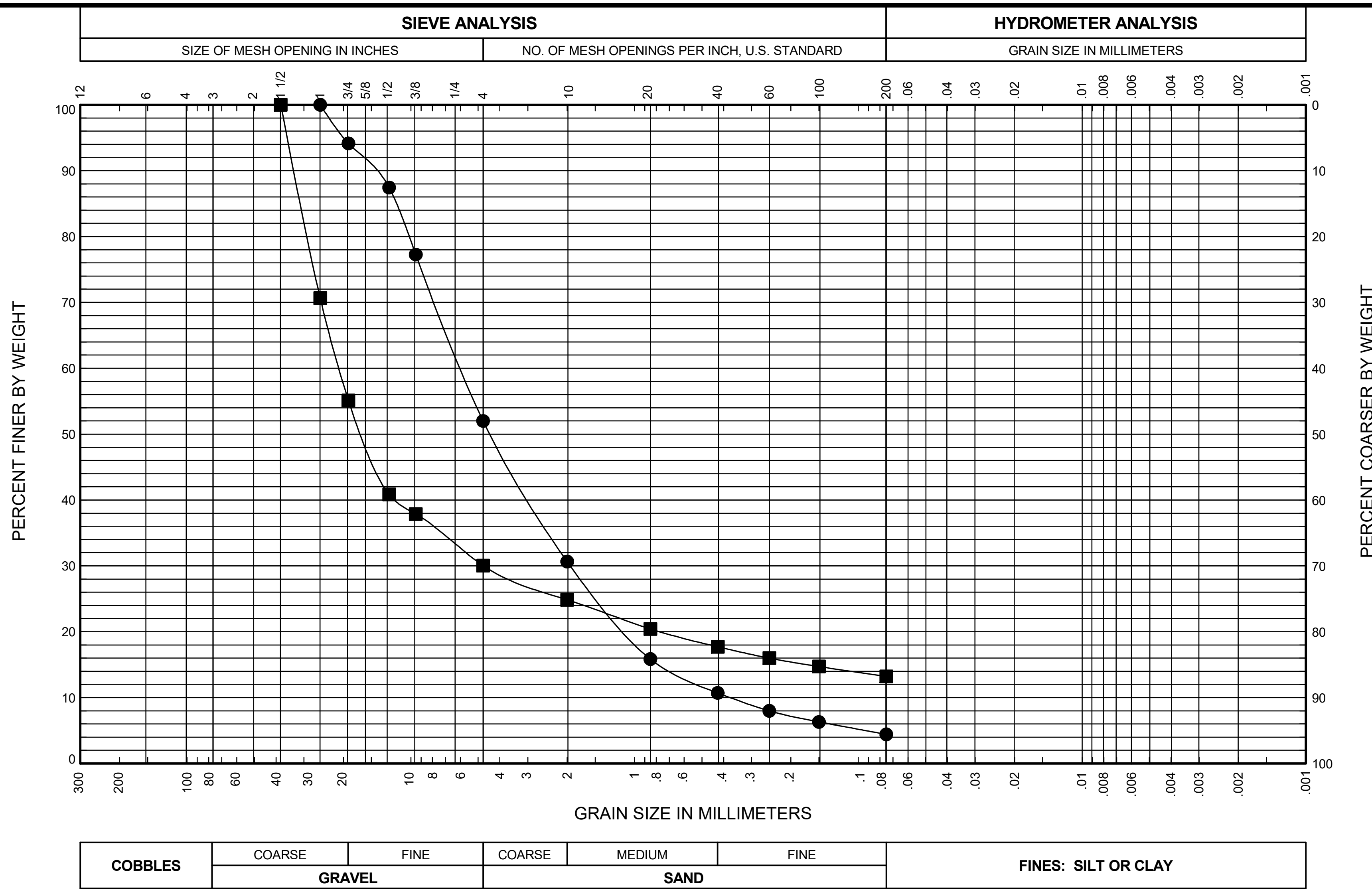
FIG. B-1
Sheet 1 of 2

RESPEC

FIG. B-1

AASHTO GSA MAIN 110813-001.GPJ SHAN WIL.GDT 3/8/24



BORING AND SAMPLE NO.	DEPTH (feet)	U.S.C.S. SYMBOL	SOIL CLASSIFICATION	COBBLE %	GRAVEL %	SAND %	FINES %	NAT. W.C. %	TEST BY	REVIEW BY	ASTM STD
● 23-05, S-6*	12.5	GW	Well-Graded Gravel with Sand		48	48	4.4		AKL	LDW	C136
■ 23-06, S-5*	10.0	GM	Silty Gravel with Sand		70	17	13.2		AKL	LDW	C136

* *Sample specimen weight did not meet required minimum mass for ASTM test method.*

Haines Maintenance & Operations Station
Project No. 57183-B
Haines, Alaska

GRAIN SIZE DISTRIBUTION

March 2024 110813-001

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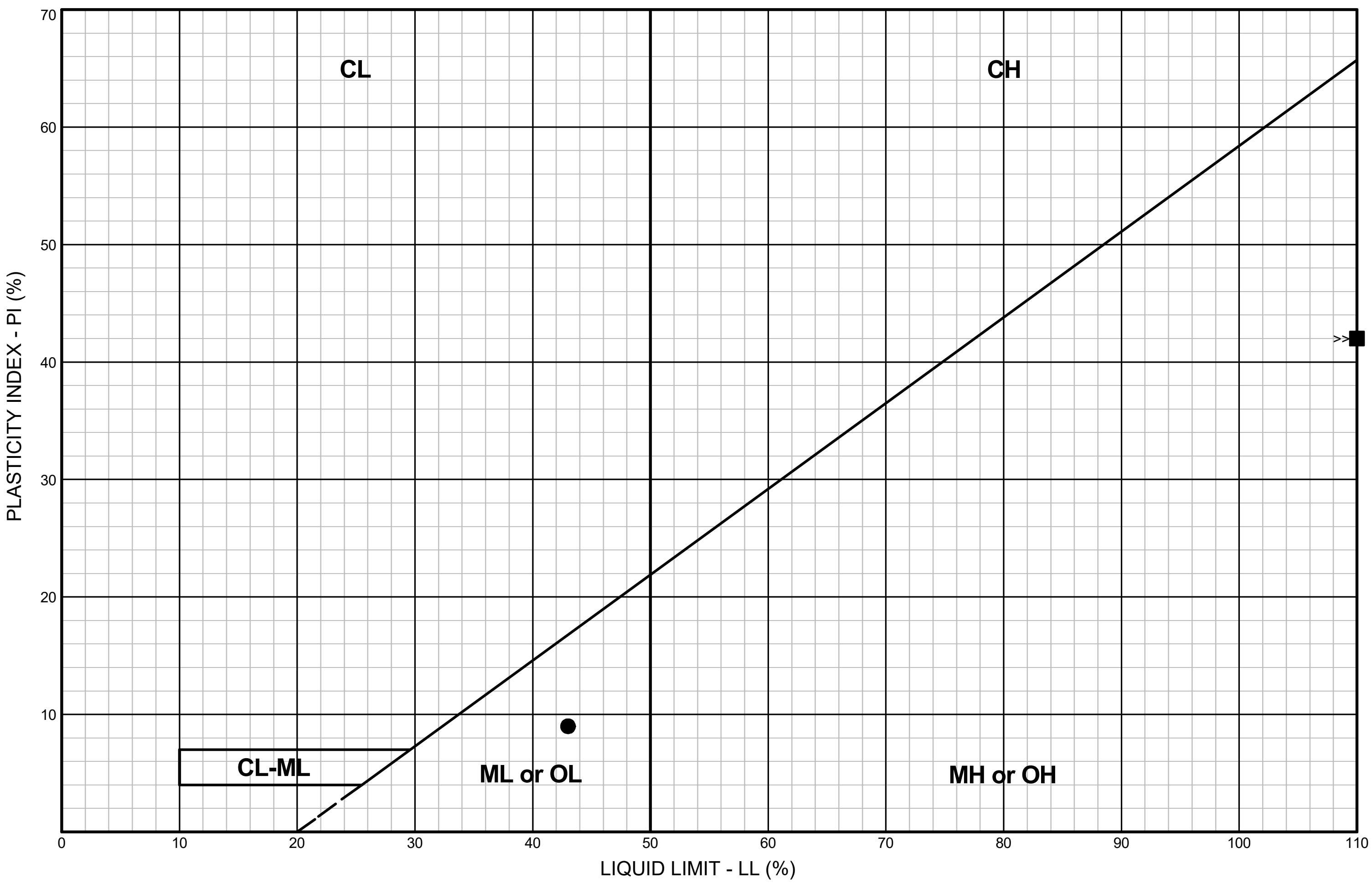
FIG. B-1
Sheet 2 of 2

RESPEC

FIG. B-1



LEGEND

CL: Low plasticity inorganic clays; sandy and silty clays

CH: High plasticity inorganic clays

ML: Inorganic silts and clayey silts of low plasticity

MH: Inorganic silts and clayey silts of high plasticity

CL-ML: Silty clays and clayey silts

OL: Organic silts and clays of low plasticity

OH: Organic silts and clays of high plasticity

LL: Liquid limit

PL: Plastic limit

PI: Plasticity index; PI=LL-PL

NP: Nonplastic

⩔ : Nonplastic

^, >>: Test value exceeds limit of graph

FINES %: Percentage of specimen mass passing the No. 200 sieve

NAT WC%: Natural water content

< 2 um %: Percentage of soil particles finer than 2 micrometers (0.002 mm); clay-size fraction

ASTM DES: ASTM International test standard designation

BORING AND SAMPLE NO.	DEPTH (feet)	U.S.C.S. SYMBOL	SOIL CLASSIFICATION	LL %	PL %	PI %	NAT WC %	FINES %	< 2 μm %	TEST BY	REVIEW BY	ASTM DES
● 23-03, S-4	7.5	ML	Silt	43	34	9	87.5			SFC		D4318
■ 23-05, S-4	7.5	MH	Elastic Silt	128	86	42	157.2			SFC		D4318

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Haines, Alaska

PLASTICITY CHART

March 2024 110813-001

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FIG. B-2
Sheet 1 of 1

RESPEC

FIG. B-2

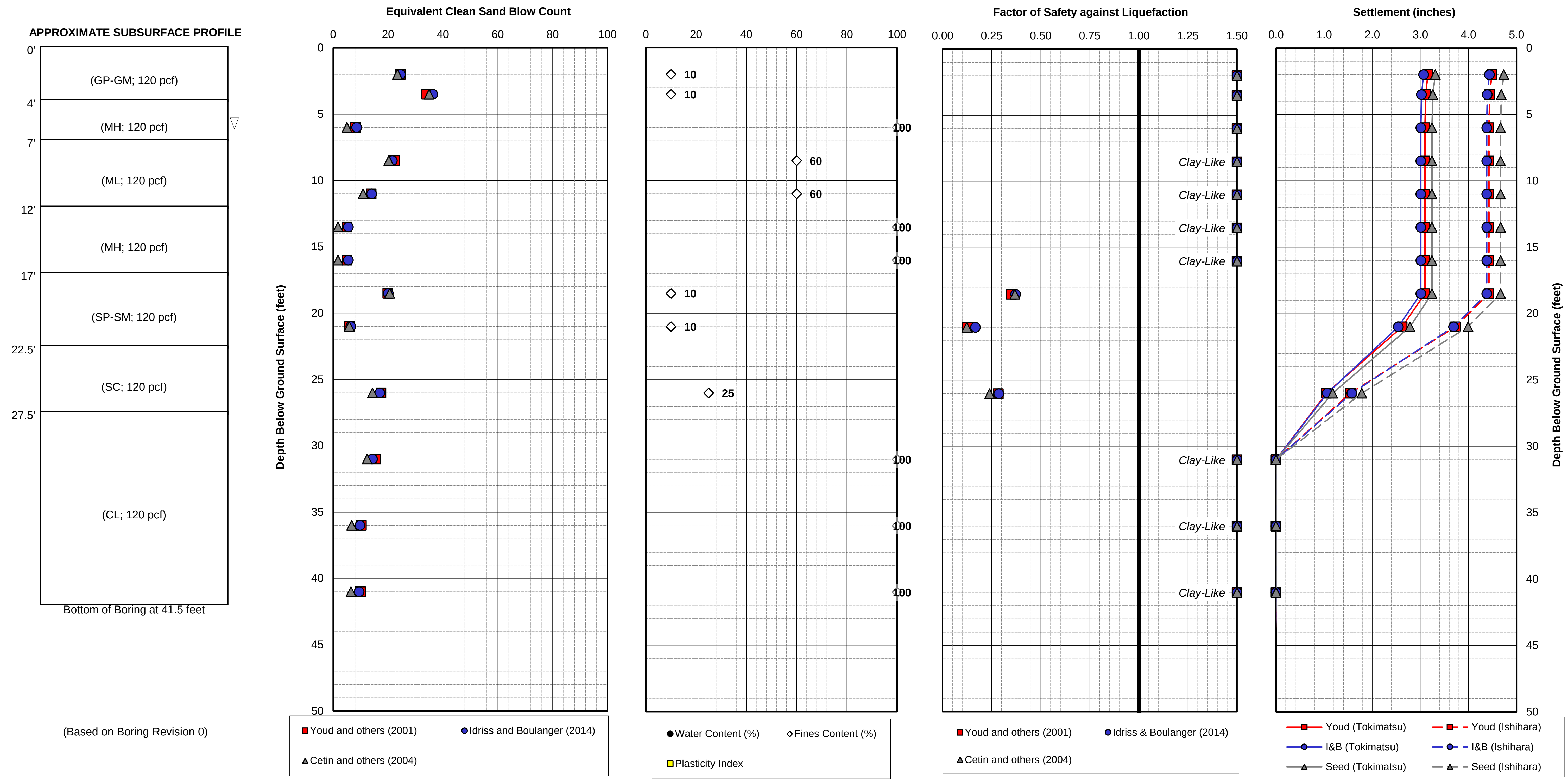
APPENDIX C: CALCULATIONS

Appendix C

Calculations

CONTENTS

- Figure C-1 through C-3 Liquefaction Analysis of Boring 23-01 through 23-03
- Pavement Calculations



NOTES

1. See main text for references.
2. The liquefaction resistance of a soil is based on its density and fines content. We used the results of the standard penetration testing to estimate the density, and the results of selected laboratory tests to estimate the fines content.
3. We estimated the seismic settlement below the water table using Tokimatsu and Seed (1987) and Ishihara and Yoshimine (1992). We estimated the settlement above the water table using Pradel (1998).
4. Average shear wave velocity in upper 30 feet = 385 fps
   Hammer energy ratio = 80%
   Groundwater table depth = 7 feet
5. pcf = pounds per cubic foot
   g = gravitational acceleration
   PGA = peak ground acceleration

Haines Maintenance & Operations Station
Project No. 57183-B
Haines, Alaska

LIQUEFACTION ANALYSIS SUMMARY
BORING 23-01
M = 7.8, PGA = 0.60g

October 2023 110813-001

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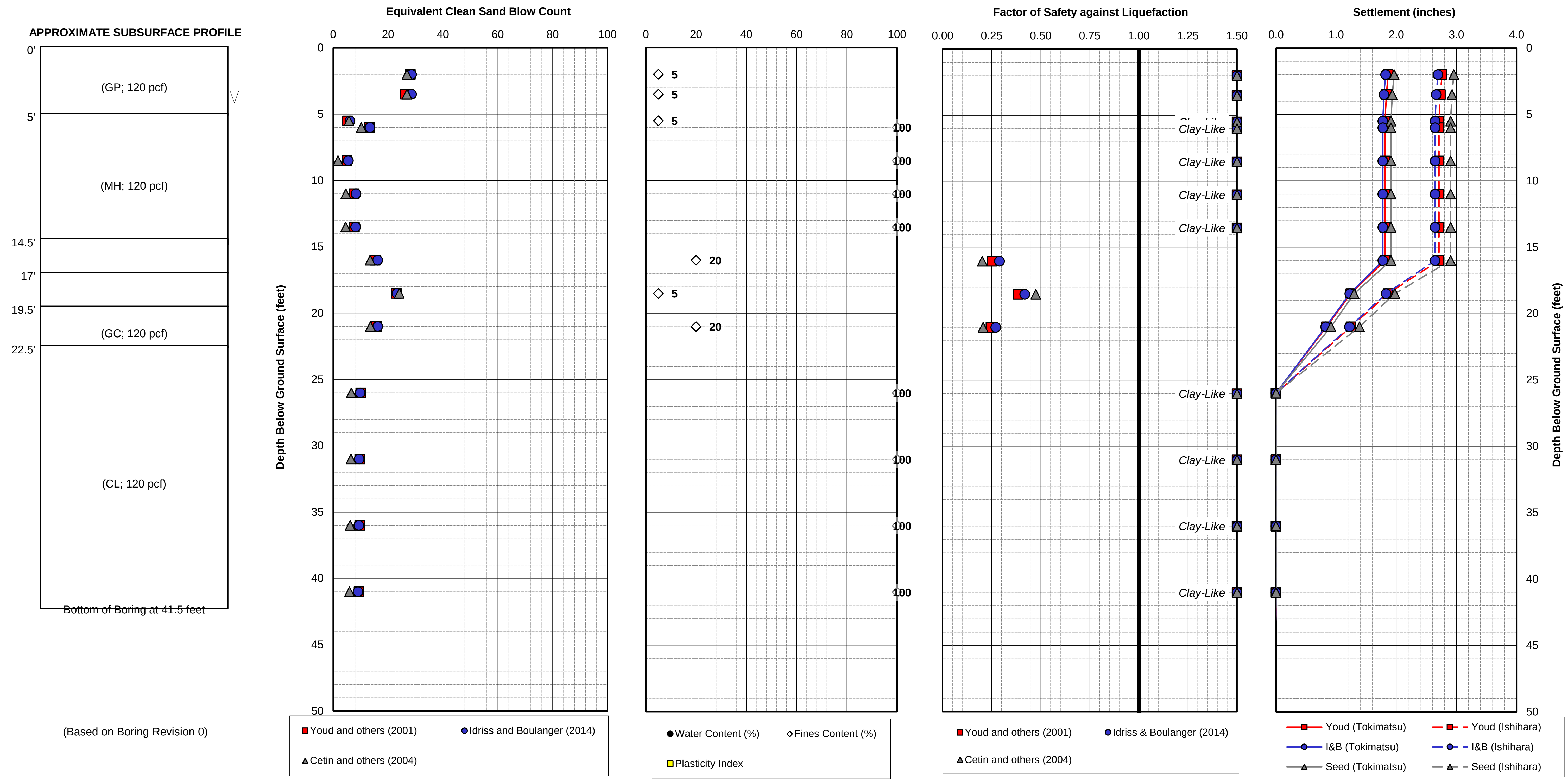
FIG. C-1



(Based on Boring Revision 0)

NOTES

1. See main text for references.
2. The liquefaction resistance of a soil is based on its density and fines content. We used the results of the standard penetration testing to estimate the density, and the results of selected laboratory tests to estimate the fines content.
3. We estimated the seismic settlement below the water table using Tokimatsu and Seed (1987) and Ishihara and Yoshimine (1992). We estimated the settlement above the water table using Pradel (1998).
4. Average shear wave velocity in upper 30 feet = 386 fps
   Hammer energy ratio = 80%
   Groundwater table depth = 5 feet
5. pcf = pounds per cubic foot
   g = gravitational acceleration
   PGA = peak ground acceleration

Haines Maintenance & Operations Station
Project No. 57183-B
Haines, Alaska

LIQUEFACTION ANALYSIS SUMMARY
BORING 23-02
M = 7.8, PGA = 0.60g

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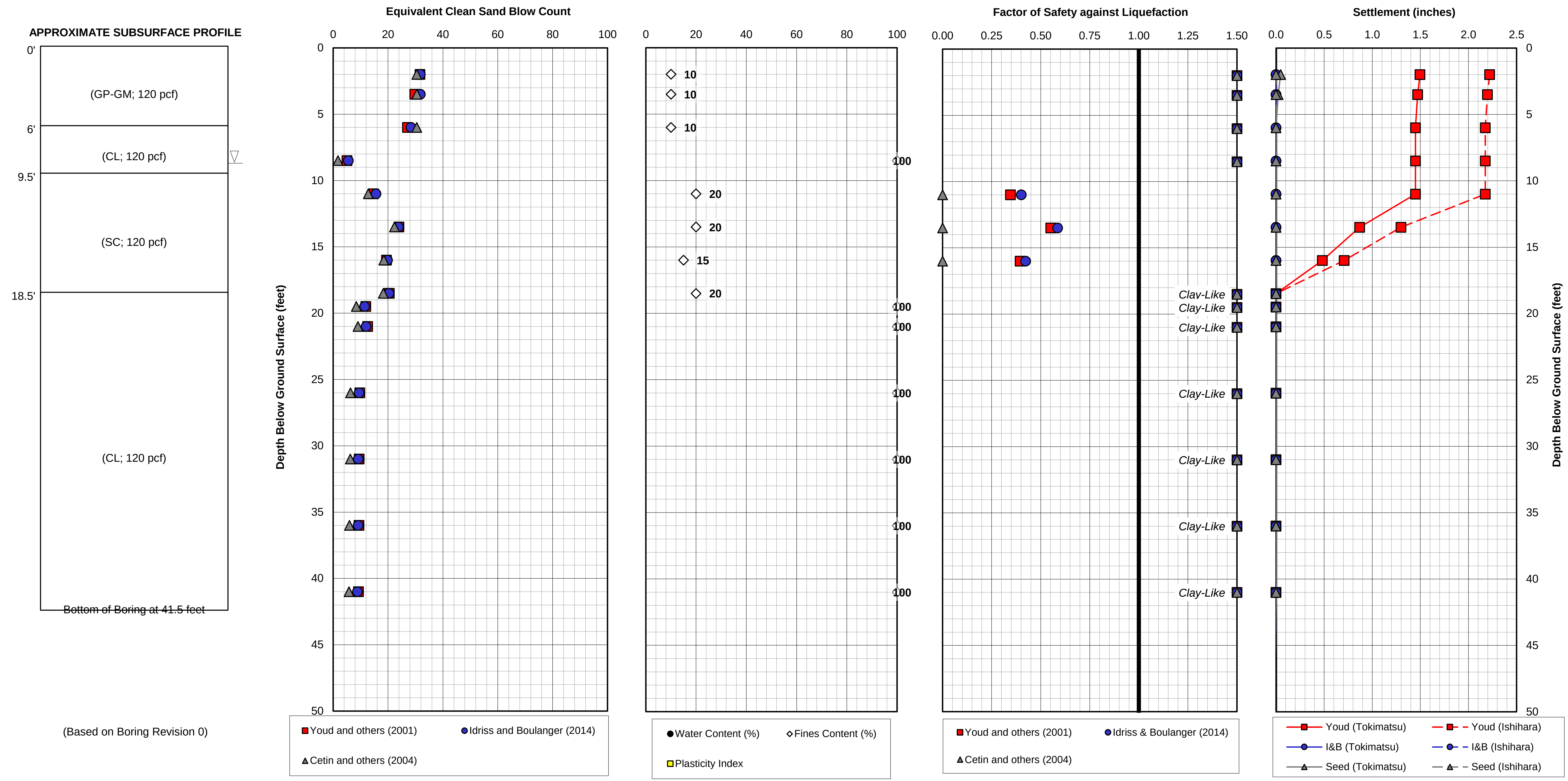
FIG. C-2



NOTES

1. See main text for references.
2. The liquefaction resistance of a soil is based on its density and fines content. We used the results of the standard penetration testing to estimate the density, and the results of selected laboratory tests to estimate the fines content.
3. We estimated the seismic settlement below the water table using Tokimatsu and Seed (1987) and Ishihara and Yoshimine (1992). We estimated the settlement above the water table using Pradel (1998).
4. Average shear wave velocity in upper 30 feet = 428 fps
   Hammer energy ratio = 80%
   Groundwater table depth = 9.5 feet
5. pcf = pounds per cubic foot
   g = gravitational acceleration
   PGA = peak ground acceleration

Haines Maintenance & Operations Station
Project No. 57183-B
Haines, Alaska

LIQUEFACTION ANALYSIS SUMMARY
BORING 23-03
M = 7.8, PGA = 0.60g

October 2023 | 110813-001

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FIG. C-3

Project Name: Haines M&O Facility			**Project Number:** 110813				**Analysis Date:** 11/14/2023			Project Status		
Design Type: New Design			Designer: Presler				Unit: US Customary			At least one layer damage is more than 100%.		
Project Location:	HAINES AP			Tire Load (lbs) 4500	Load Description:		ESAL					
					Load Loc (in)							
			Design	Tire Press. (psi)	X:	0	13.5					
Design AADT:	100		Loadings	110	Y:	0	0					
Spring%:	25		249,236		Eval Loc (in)							
Summer%:	42		418,717		X:	0	6.75					
Fall%:	17		169,481		Y:	0	0					
Winter%:	16		159,511									
Total%:	100		996,945									
	Layer	Critical Z Coordinate (in)	Asphalt Properties	Season	Modulus (Ksi)	Poisson's Ratio	Tensile Micro Strain	Compressive Stress (psi)	Million Cycles to Failure	Past Damage (%)	Future Damage (%)	Total Damage (%)
			Air%:	Spring	20	0.35		110.0	0.00		17688.88	17688.88
Thickness (in):	12	0.01	Asphalt%:	Summer	30	0.35		110.0	0.01		7333.05	7333.05
Name:	Aggregate Base P200<10%		ensity (kg/m3)	Fall	30	0.35		110.0	0.01		2968.14	2968.14
Use TAI:				Winter	50	0.35		110.0	0.03		528.34	528.34
									Total Damage:		28518.41	28518.41
			Air%:	Spring	20	0.40		15.1	0.92		27.16	27.16
Thickness (in):	24	12.01	Asphalt%:	Summer	30	0.40		15.0	3.80		11.03	11.03
Name:	Select A P200<10%		Density:	Fall	30	0.40		15.0	3.80		4.46	4.46
Use TAI:				Winter	50	0.40		14.9	20.53		0.78	0.78
									Total Damage:		43.43	43.43
			Air%:	Spring	5	0.45		1.8	5.11		4.88	4.88
Thickness (in):	0	36.01	Asphalt%:	Summer	5	0.45		1.4	10.25		4.09	4.09
Name:	Subgrade P200>30%		Density:	Fall	5	0.45		1.4	10.25		1.65	1.65
Use TAI:				Winter	5	0.45		1.1	26.66		0.60	0.60
									Total Damage:		11.22	11.22
			Air%:	Spring								
Thickness (in):			Asphalt%:	Summer								
Name:			Density:	Fall								
Use TAI:				Winter								
									Total Damage:			
				Spring								
Thickness (in):				Summer								
Name:				Fall								
				Winter								
									Total Damage:			

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PCASE Version:	7.0.4 2022-08-24
Design Name:	Aggregate Surface
Layer Model Name:	Aggregate Surface
Drainage Station:	Not selected
Frost Station:	USA-Alaska-Haines
Pavement Use:	Roadway
Design Type:	Unsurfaced
Traffic Area:	Parking Areas
Analysis Type:	CBR
Depth of Frost (in):	44.34
Wander Width (in):	33.35

Haines Maintenance & Operations Station
Project No. 57183-B
Haines, Alaska

Layer Information

Layer Type	Material Type	Frost Code	Moisture Content (%)	Dry Unit Weight (lb/ft^3)	Analysis	Non-frost Design Thickness	Reduced Subgrade Strength	Limited Subgrade Penetration	CBR
Unsurfaced	Unbound Aggregate	NFS	5	135	Compute	4			100
Subbase	Unbound Aggregate	NFS	5	135	Manual	12			50
Natural Subgrade	Cohesive Cut		18	100	Manual	4			6

Calc. Messages

Type	Message
Information	Frost design thicknesses were requested but no frost-susceptible layers were identified. RSS and LSFP results will therefore be the same as non-frost.

Traffic Information

Service	Army
Pattern Name:	Assumed Traffic

Vehicles	Load (lb)	Passes	Equivalent Passes
TRUCK, 3 AXLE	35000	52000	3432
TRUCK, 5 AXLE	80000	52000	52000
TRUCK, 5 AXLE	80000		55432

Estimated AASHTO ESALS: 0

Project Name: Haines M&O Facility	**Project Number:** 110813	**Analysis Date:** 11/14/2023	Project Status
Design Type: New Design	Designer: Presler	Unit: US Customary	All layer damages less than 100%.

		Design Loadings				
Project Location:	HAINES AP		Tire Load (lbs): 4500	Load Description:	ESAL	
				Load Loc (in)		
			Tire Press. (psi): 110	X:	0	13.5
Design AADT:	100			Y:	0	0
Spring%:	25	249,236		Eval Loc (in)		
Summer%:	42	418,717		X:	0	6.75
Fall%:	17	169,481		Y:	0	0
Winter%:	16	159,511				
Total%:	100	996,945				

Layer		Critical Z Coordinate (in)	Asphalt Properties		Season	Modulus (Ksi)	Poisson's Ratio	Tensile Micro Strain	Compressive Stress (psi)	Million Cycles to Failure	Past Damage (%)	Future Damage (%)	Total Damage (%)
			Air%:	5	Spring	350	0.30	301		0.81		30.86	30.86
Thickness (in):	4	3.99	Asphalt%:	5.5	Summer	300	0.30	266		1.38		30.26	30.26
Name:	sphalt Concrete (Unmodified Asph		Density (pcf)	148	Fall	300	0.30	266		1.38		12.25	12.25
Use TAI:	Yes				Winter	1200	0.30	105		9.05		1.76	1.76
										Total Damage:		75.13	75.13
			Air%:		Spring	40	0.35		30.0	1.01		24.66	24.66
Thickness (in):	4	4.01	Asphalt%:		Summer	50	0.35		36.1	1.14		36.79	36.79
Name:	Aggregate Base P200<6%		Density:		Fall	50	0.35		36.1	1.14		14.89	14.89
Use TAI:					Winter	100	0.35		25.8	32.80		0.49	0.49
										Total Damage:		76.83	76.83
			Air%:		Spring	20	0.40		13.6	1.29		19.28	19.28
Thickness (in):	24	8.01	Asphalt%:		Summer	30	0.40		16.1	3.02		13.88	13.88
Name:	Select A P200<10%		Density:		Fall	30	0.40		16.1	3.02		5.62	5.62
Use TAI:					Winter	50	0.40		12.1	40.70		0.39	0.39
										Total Damage:		39.17	39.17
			Air%:		Spring	5	0.45		1.6	7.35		3.39	3.39
Thickness (in):	0	32.01	Asphalt%:		Summer	5	0.45		1.4	12.25		3.42	3.42
Name:	Subgrade P200>30%		Density:		Fall	5	0.45		1.4	12.25		1.38	1.38
Use TAI:					Winter	5	0.45		0.9	51.28		0.31	0.31
										Total Damage:		8.50	8.50
					Spring								
Thickness (in):					Summer								
Name:					Fall								
					Winter								
										Total Damage:			

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Haines Maintenance & Operations Station
Project No. 57183-B
Haines, Alaska

AC Surface - Asphalt Surface

PCASE Version:	7.0.4 2022-08-24
Design Name:	AC Surface
Layer Model Name:	Asphalt Surface
Drainage Station:	Not selected
Frost Station:	USA-Alaska-Haines
Pavement Use:	Roadway
Design Type:	Flexible
Traffic Area:	Road Areas
Analysis Type:	CBR
Depth of Frost (in):	50.17
Wander Width (in):	33.35

Haines Maintenance & Operations Station
Project No. 57183-B
Haines, Alaska

Layer Information

Layer Type	Material Type	Frost Code	Moisture Content (%)	Dry Unit Weight (lb/ft^3)	Analysis	Non-frost Design Thickness	Reduced Subgrade Strength	Limited Subgrade Penetration	CBR
Asphalt Concrete	Asphalt Cement	NFS	0	140	Compute	4	4	4	
Base	Unbound Aggregate	NFS	5	135	Compute	4	4	7.37	80
Subbase	Unbound Aggregate	NFS	5	135	Manual	24	24	24	50
Natural Subgrade	Cohesionless Cut	F3F4	10	120	Manual	4			6

Traffic Information

Service	Army
Pattern Name:	Assumed Traffic

Vehicles	Load (lb)	Passes	Equivalent Passes
TRUCK, 3 AXLE	35000	52000	676
TRUCK, 5 AXLE	80000	52000	52000
TRUCK, 5 AXLE	80000		52676

Estimated AASHTO ESALS: 21590480

Haines Maintenance & Operations Station
Project No. 57183-B
Haines, Alaska

PCASE Version:	7.0.4 2022-08-24	
Design Name:	PCC Surface	
Layer Model Name:	PCC Surface	
Drainage Station:	Not selected	
Frost Station:	USA-Alaska-Haines	
Pavement Use:	Roadway	
Design Type:	Rigid	
Traffic Area:	Road Areas	
Analysis Type:	K	
Depth of Frost (in):	51.01	
Wander Width (in):	33.35	
% Load Transfer:	0	
% Steel:	0	
Joint Spacing:	12.5-15 ft.	Joint/dowel information based on RSS PCC thickness
Dowel Spacing:	12 in.	
Dowel Length:	16 in.	
Dowel Diameter:	0.75 in.	

Layer Information

Layer Type	Material Type	Frost Code	Moisture Content (%)	Dry Unit Weight (lb/ft^3)	Analysis	Non-frost Design Thickness	Reduced Subgrade Strength	Limited Subgrade Frost	Flexural Strength (psi)	Modulus (psi)	K (pci)	Effective K (pci)
Portland Cement Concrete	Portland Cement	NFS	0	145	Compute	6	6.15	5.37	650	4000000		
Subbase	Unbound Aggregate	NFS	5	135	Manual	24	24	30.73				315
Natural Subgrade	Cohesionless Cut	F3F4	10	120	Manual	4					100	100

Traffic Information

Service	Army
Pattern Name:	Assumed Traffic

Vehicles	Load (lb)	Passes	Equivalent Passes
TRUCK, 3 AXLE	35000	52000	1982
TRUCK, 5 AXLE	80000	52000	52000
TRUCK, 5 AXLE	80000		53982

Estimated AASHTO ESALS: 127549

Appendix D

Photo Report

APPENDIX D: PHOTO REPORT

Photo 1: Boring 23-01, Sample 2, 2.5-4.0 feet bgs.

Photo 2: Boring 23-01, Sample 3, 5.0-6.5 feet bgs.

Photo 3: Boring 23-01, Sample 9, 20.0-21.5 feet bgs.

Photo 4: Boring 23-01, Sample 11, 30.0-31.5 feet bgs.

Photo 5: Boring 23-02, Sample 2, 2.5-4.0 feet bgs.

Photo 6: Boring 23-02, Sample 5, 7.5-9.0 feet bgs.

Photo 7: Boring 23-02, Sample 7, 15.0-16.5 feet bgs.

Photo 8: Boring 23-02, Sample 10, 25.0-26.5 feet bgs.

Photo 9: Boring 23-03, Sample 2, 2.5-4.0 feet bgs.

Photo 10: Boring 23-03, Sample 6, 12.5-14.0 feet bgs.

Photo 11: Boring 23-03, Sample 9, 20.0-21.5 feet bgs.

Photo 12: Boring 23-03, Sample 12, 35.0-36.5 feet bgs.

Photo 13: Boring 23-04, Samples 2, 2.5-4.0 feet bgs.

Photo 14: Boring 23-04, Samples 7, 15.0-16.5 feet bgs.

Photo 15: Boring 23-04, Sample 9, 20.0-21.5 feet bgs.


Photo 16: Boring 23-05, Sample 2, 2.5-4.0 feet bgs.

Photo 17: Boring 23-05, Sample 4, 7.5-9.0 feet bgs.


Photo 18: Site Photo Near Boring 23-05.

Photo 19: Boring 23-06, Sample 3, 5.0-6.5 feet bgs.

Photo 20: Boring 23-06, Sample 4, 7.5-9.0 feet bgs.

Photo 21: Boring 23-06, Sample 9, 20.0-21.5 feet bgs.

Photo 22: Site Photo Near Boring 23-06.

Photo 23: Site Photo Looking South From Boring 23-03.

Photo 24: Site Photo Looking North From Boring 23-03.

IMPORTANT INFORMATION

Important Information

About Your Geotechnical Report

CONSULTING SERVICES ARE PERFORMED FOR SPECIFIC PURPOSES AND FOR SPECIFIC CLIENTS.

Consultants prepare reports to meet the specific needs of specific individuals. A report prepared for a civil engineer may not be adequate for a construction contractor or even another civil engineer. Unless indicated otherwise, your consultant prepared your report expressly for you and expressly for the purposes you indicated. No one other than you should apply this report for its intended purpose without first conferring with the consultant. No party should apply this report for any purpose other than that originally contemplated without first conferring with the consultant.

THE CONSULTANT'S REPORT IS BASED ON PROJECT-SPECIFIC FACTORS.

A geotechnical/environmental report is based on a subsurface exploration plan designed to consider a unique set of project-specific factors. Depending on the project, these may include the general nature of the structure and property involved; its size and configuration; its historical use and practice; the location of the structure on the site and its orientation; other improvements such as access roads, parking lots, and underground utilities; and the additional risk created by scope of service limitations imposed by the client. To help avoid costly problems, ask the consultant to evaluate how any factors that change subsequent to the date of the report may affect the recommendations. Unless your consultant indicates otherwise, your report should not be used (1) when the nature of the proposed project is changed (for example, if an office building will be erected instead of a parking garage, or if a refrigerated warehouse will be built instead of an unrefrigerated one, or chemicals are discovered on or near the site); (2) when the size, elevation, or configuration of the proposed project is altered; (3) when the location or orientation of the proposed project is modified; (4) when there is a change of ownership; or (5) for application to an adjacent site. Consultants cannot accept responsibility for problems that may occur if they are not consulted after factors that were considered in the development of the report have changed.

SUBSURFACE CONDITIONS CAN CHANGE.

Subsurface conditions may be affected as a result of natural processes or human activity. Because a geotechnical/environmental report is based on conditions that existed at the time of subsurface exploration, construction decisions should not be based on a report whose adequacy may have been affected by time. Ask the consultant to advise if additional tests are desirable before construction starts; for example, groundwater conditions commonly vary seasonally.

Construction operations at or adjacent to the site and natural events such as floods, earthquakes, or groundwater fluctuations may also affect subsurface conditions and, thus, the continuing adequacy of a geotechnical/environmental report. The consultant should be kept apprised of any such events and should be consulted to determine if additional tests are necessary.

MOST RECOMMENDATIONS ARE PROFESSIONAL JUDGMENTS.

Site exploration and testing identifies actual surface and subsurface conditions only at those points where samples are taken. The data were extrapolated by your consultant, who then applied

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judgment to render an opinion about overall subsurface conditions. The actual interface between materials may be far more gradual or abrupt than your report indicates. Actual conditions in areas not sampled may differ from those predicted in your report. While nothing can be done to prevent such situations, you and your consultant can work together to help reduce their impacts. Retaining your consultant to observe subsurface construction operations can be particularly beneficial in this respect.

A REPORT'S CONCLUSIONS ARE PRELIMINARY.

The conclusions contained in your consultant's report are preliminary, because they must be based on the assumption that conditions revealed through selective exploratory sampling are indicative of actual conditions throughout a site. Actual subsurface conditions can be discerned only during earthwork; therefore, you should retain your consultant to observe actual conditions and to provide conclusions. Only the consultant who prepared the report is fully familiar with the background information needed to determine whether or not the report's recommendations based on those conclusions are valid and whether or not the contractor is abiding by applicable recommendations. The consultant who developed your report cannot assume responsibility or liability for the adequacy of the report's recommendations if another party is retained to observe construction.

THE CONSULTANT'S REPORT IS SUBJECT TO MISINTERPRETATION.

Costly problems can occur when other design professionals develop their plans based on misinterpretation of a geotechnical/environmental report. To help avoid these problems, the consultant should be retained to work with other project design professionals to explain relevant geotechnical, geological, hydrogeological, and environmental findings, and to review the adequacy of their plans and specifications relative to these issues.

BORING LOGS AND/OR MONITORING WELL DATA SHOULD NOT BE SEPARATED FROM THE REPORT.

Final boring logs developed by the consultant are based upon interpretation of field logs (assembled by site personnel), field test results, and laboratory and/or office evaluation of field samples and data. Only final boring logs and data are customarily included in geotechnical/environmental reports. These final logs should not, under any circumstances, be redrawn for inclusion in architectural or other design drawings, because drafters may commit errors or omissions in the transfer process.

To reduce the likelihood of boring log or monitoring well misinterpretation, contractors should be given ready access to the complete geotechnical engineering/environmental report prepared or authorized for their use. If access is provided only to the report prepared for you, you should advise contractors of the report's limitations, assuming that a contractor was not one of the specific persons for whom the report was prepared, and that developing construction cost estimates was not one of the specific purposes for which it was prepared. While a contractor may gain important knowledge from a report prepared for another party, the contractor should discuss the report with your consultant and perform the additional or alternative work believed necessary to obtain the data specifically appropriate for construction cost estimating purposes. Some clients hold the mistaken impression that simply disclaiming responsibility for the accuracy of subsurface information always insulates them from attendant liability. Providing the best available information to contractors helps prevent costly construction problems and the adversarial attitudes that aggravate them to a disproportionate scale.

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READ RESPONSIBILITY CLAUSES CLOSELY.

Because geotechnical/environmental engineering is based extensively on judgment and opinion, it is far less exact than other design disciplines. This situation has resulted in wholly unwarranted claims being lodged against consultants. To help prevent this problem, consultants have developed a number of clauses for use in their contracts, reports, and other documents. These responsibility clauses are not exculpatory clauses designed to transfer the consultant's liabilities to other parties; rather, they are definitive clauses that identify where the consultant's responsibilities begin and end. Their use helps all parties involved recognize their individual responsibilities and take appropriate action. Some of these definitive clauses are likely to appear in your report, and you are encouraged to read them closely. Your consultant will be pleased to give full and frank answers to your questions.