

SUBMITTED TO:

RESPEC

2700 Gambell Street, Suite 500
Anchorage, Alaska 99503

BY:

Shannon & Wilson, Inc.
5430 Fairbanks Street
Anchorage, Alaska 99518

(907) 561-2120

www.shannonwilson.com

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GEOTECHNICAL ENGINEERING REPORT

Yakutat Airport Water Line Extension

YAKUTAT, ALASKA

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Submitted To: RESPEC
2700 Gambell Street, Suite 500
Anchorage, Alaska 99503
Attn: Ms. Karen Brady, P.E.

Subject: DRAFT GEOTECHNICAL ENGINEERING REPORT, YAKUTAT AIRPORT
WATER LINE EXTENSION, YAKUTAT, ALASKA

Shannon & Wilson prepared this report and participated in this project as a subconsultant to RESPEC Company, LLC (RESPEC) in support of upgrades to the City and Borough of Yakutat's (CBY) public water facilities. Our original geotechnical scope of services was specified in Tasks 3 and 4 of our March 21, 2024, proposal which was authorized via a signed subconsultant agreement on May 9, 2024. Our additional scope of services was specified in Task 11 of our July 19, 2024, proposal and Tasks 14 and 15 of our September 25, 2024, proposal. These additional services were authorized via signed subconsultant agreements on August 30 and November 5, 2024, respectively. Note that two of our proposed borings were not advanced due to conflicts with underground utilities. This report presents the results of our geotechnical explorations and engineering recommendations and was prepared by the undersigned. A draft environmental report summarizing environmental field activities and analytical testing results has been provided under separate cover. A wetlands delineation report has been submitted under separate cover.

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, INC.

Russell
Hepner, EIT
Digitally signed by
Russell Hepner, EIT
Date: 2025.04.15
18:28:53 -08'00'

Russell Hepner, EIT
Geotechnical Staff

FED/RCH:KLB



Kyle Brennan, PE
Vice President

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1 INTRODUCTION

This report presents the results of geotechnical engineering studies conducted by Shannon & Wilson in support of proposed public water system upgrades in Yakutat, Alaska. These upgrades generally include extending the municipal water supply to the Yakutat Airport, and construction of new water utility facilities, including a holding tank, pump house with an associated leach field, and water well. The purpose of this geotechnical study was to gather subsurface geotechnical data and provide geotechnical engineering recommendations to support design and construction of these proposed improvements. To accomplish this, 29 borings were advanced in the project area. Presented in this report are descriptions of the site and project, subsurface exploration and laboratory test results, an interpretation of subsurface conditions, and our geotechnical recommendations from our engineering studies.

2 SITE AND PROJECT DESCRIPTION

The project is in the City and Borough of Yakutat (CBY), Alaska, which is located along the southeastern shore of Yakutat Bay. Generally, the CBY area is a large (approximately 7,650 square miles), natively forested, intermittently hilly, relatively flat glacial drainage basin. The terrain can be characterized as marshy, populated with numerous wetlands, lakes, braided streams, creeks and a shallow water table. Vegetation consisting of large spruce trees (roughly 3 to 5 feet in diameter) are located throughout the project area. The proposed site of the new tank and pump house has been previously logged, but large diameter trees remain fallen throughout that area. Further details on the project setting are provided under separate cover in our November 2024 *Aquatic Resource Delineation Report, Yakutat Airport Water Line Extension, Yakutat, Alaska*. A vicinity map indicating the general project location is presented as Figure 1.

The CBY area is largely undeveloped, with development of the city itself primarily concentrated along the shore of Monti Bay. The city center connects to the nearby airport via a two-lane asphalt paved road (Airport Road) which is approximately 3.5 miles long, and it is through this development corridor where the water line is proposed to extend. The project area generally slopes slightly down from Ophir Creek to the Yakutat Airport; however, there is a roughly 15- to 20-foot-high hill on the townsite side of Ophir Creek. See Figure 2 (3 sheets) for an overview of prominent site features and the approximate boring locations.

Our understanding of the project scope is that the municipal water service is to be extended from the townsite to the Yakutat Airport. The proposed alignment is generally along/underneath the shoulder of existing roadways and is anticipated to be installed utilizing both routine trenching and trenchless techniques. Occasional waterway (i.e., Ophir Creek) and road crossings are anticipated to utilize horizontal directional drilling (HDD) techniques. In addition to the construction of new water line facilities, this project proposes constructing a new 500,000-gallon water tank, water well, and approximately 1,400-square foot pump house with leach field. The total proposed alignment length is approximately 20,000 feet.

3 SUBSURFACE EXPLORATIONS

Subsurface explorations for this study consisted of drilling and sampling 29 borings, designated Borings B-01 through B-29, in the project area between December 4 and 14, 2024. One additional shallow boring for a percolation test, designated PERC, was advanced on December 9, 2024. The general boring locations were selected by Shannon & Wilson to provide relatively even coverage along the alignment of the proposed water line and to selectively test the soils near the airport for the potential presence of per- and polyfluoroalkyl substances (PFAS). The actual boring locations were adjusted by our representative in the field to avoid conflicts with buried utilities. Note that two of our proposed boring locations near the airport were not drilled due to utility conflicts. The boring locations, shown in Figure 2, were measured as GPS coordinates that are generally considered accurate to within approximately 20 feet horizontally. It should be noted that GPS accuracy may be affected by geographic features, tree canopies, and atmospheric anomalies. The surface elevations shown on the boring logs were estimated from nearby survey control points provided by RESPEC in a survey control plan set. Therefore, the boring locations shown on the site plan and the elevations reported on the boring logs should be considered approximate.

Drilling services for this project were provided by Discovery Drilling of Anchorage, Alaska, using a track-mounted Geoprobe 6712DT drill rig. Representatives from our firm were present during drilling to locate the borings, observe drill action, collect geotechnical and environmental soil and water samples, log subsurface conditions, observe groundwater conditions, and perform a percolation test. We coordinated with the Call Locate Center and local authorities to clear the boring locations of buried public utilities prior to drilling. A dig permit from the Yakutat Airport manager for borings on airport property and a Right-of-Way (ROW) permit from the Alaska Department of Transportation and Public Facilities (DOT&PF) for borings advanced along Airport Road were also required prior to drilling.

The borings were advanced with 3 ¼-inch inner diameter (ID), continuous flight, hollow-stem augers to a typical depth of approximately 17 below the ground surface (bgs). Borings adjacent to Ophir Creek and at the proposed pump house location were advanced to approximately 32 feet bgs, and the boring at the proposed water well site was advanced to approximately 60 feet bgs. The boring for the percolation test was advanced to approximately 1.6 feet bgs. As the borings were advanced, samples were recovered using modified penetration test (MPT) methods at 2.5-foot intervals for the first 10 feet, and then generally at 5-foot intervals thereafter to the bottom of the borings. In the MPT method, samples are recovered by driving a 3-inch outer diameter (OD) split-spoon sampler into the bottom of the advancing hole with blows of a 340-pound hammer free falling 30 inches onto the drill rod.

For each sample, the number of blows required to drive the sampler the middle 12 inches of a 24-inch penetration (MPT) into undisturbed soil is recorded. Blow counts are shown graphically on the boring log figures as “penetration resistance” and are displayed adjacent to sample depth. The penetration resistance values give a measure of the relative density (compactness) or consistency (stiffness) of cohesionless or cohesive soils, respectively. Where the sampler did not penetrate the full 24 inches (i.e., the blow count for a 6-inch penetration exceeded 50 blows or when after 10 hammer drops the sampler had not moved), the penetration resistance is recorded as “sampler refusal” on the boring logs, in place of penetration resistance, and our log reports the blow count and corresponding total penetration in inches of the sample where refusal occurred. In addition to the split spoon samples, a grab sample of the near-surface soils was collected from the auger cuttings in the upper 1 to 2 feet of each boring.

The soil samples recovered during drilling were observed and described in the field in general accordance with the classification system described by ASTM International (ASTM) D2488. Selected samples recovered during drilling were tested in our laboratory to refine our soil descriptions in general accordance with the Unified Soil Classification System (USCS) described in Appendix A (Figure A-1, 3 sheets). Frost classifications were also estimated for samples based on laboratory testing (sieve analyses and hydrometer) and are shown on the boring logs. The frost classification system is presented in Appendix A as Figure A-2. Summary logs of the borings are presented in Appendix A as Figures A-3 through A-31.

3.1.1 Field Screening

Soil samples recovered above the water table during drilling were ‘screened’ for volatile organic vapors using a photoionization detector (PID) and an Alaska Department of Environmental Conservation (ADEC)-approved headspace-screening technique. The PID

was calibrated before screening activities with 100 parts per million (ppm) isobutylene standard gas. Field screening was conducted by collecting samples in re-sealable plastic bags, heating the bags to a minimum of 40 degrees Fahrenheit (F), and inserting the PID probe into the air space above the soil in the bag within 60 minutes of sample collection. One sample (Boring B-27 Sample 2 2.5' bgs) with a PID value exceeding 10 ppm was collected and submitted to Eurofins Seattle using chain-of-custody procedures. Field screening results are presented on the boring logs in Appendix A and are discussed under separate cover in our February 2025 *Draft Environmental Summary Report, Yakutat Water Line Expansion, Yakutat, Alaska* (Draft ESR).

3.1.2 PFAS Sample Collection

Six soil samples were collected for PFAS analysis; one each from Borings B-20 through B-25. The samples were generally collected as close as practicable to the soil/groundwater interface and submitted to Eurofins Sacramento using chain-of-custody procedures for analysis of PFAS by Environmental Protection Agency (EPA) 537M. One Class A water well sample was collected from Boring B-01 on December 11, 2024, as per the August 2024 *Exploration Phase – Sampling and Analysis Plan, Yakutat Airport Water Line Extension, Yakutat, Alaska*. Details and discussion of the analytical results are provided under separate cover in our Draft ESR.

3.1.3 Groundwater Monitoring

After drilling, Boring B-01 was completed by installing a 2-inch polyvinyl chloride (PVC) well casing to facilitate observation of groundwater levels and water sample collection at a later date. The 2-inch PVC well casing featured a 10-foot section of 'prepack' screened PVC from approximately 20 feet to 30 feet bgs. Borings B-02, B-03, B-08, B-12, B-16, B-24, B-27 and B-29 were completed with 1-inch diameter 'piezometer' PVC casings with hand-slotted tips. The annular space between the borehole walls and casing were backfilled with cuttings produced during drilling activity. Steel flush-mount monuments were installed to protect the top of the casings installed along Airport Road, whereas borings outside road extents (B-02, B-03, B-24, and B-29) were completed with a few feet of PVC 'stick up' above the ground surface. Borings that were not completed with piezometers were backfilled with drill cuttings produced during drilling and hand tamped level at the ground surface. Installation details for the observation wells are shown on the corresponding boring logs.

3.1.4 Percolation Testing

A percolation test boring was advanced adjacent to Boring B-02 to a depth of approximately 1.6 feet bgs. Note that this approximate depth was chosen based on conversations with RESPEC about the surface organic thickness and measured depth to groundwater within a

few days of drilling Borings B-01 and B-02. After drilling to the approximate percolation test depth, a 4-inch inner diameter (ID), open-ended PVC casing was placed in the bottom of the test hole with the end of the casing planted firmly in the bottom of the borehole. A representative from our firm was present to observe percolation casing installation and conduct percolation testing.

Percolation testing was generally performed as described in the 'Falling Head Percolation Test Procedure' described in the U.S. Environmental Protection Agency (EPA 1980) *Design Manual for Onsite Wastewater Treatment and Disposal Systems*. In summary, water is poured into the installed casing to a total water column height of 12 inches. The water is then allowed to drain and absorb into the soil under the bottom of the hole. If the water drains completely away in 10 minutes or less, twice consecutively, the percolation test can proceed immediately. If the water does not drain completely in 10 minutes or less, a soaking period is required. During the soaking period, water is poured into the casing and the water column height is maintained at 12 inches for at least 4 hours. Percolation testing then proceeds 15 to 30 hours following the soaking period, if required. To conduct the percolation test, the water level inside the casing is adjusted to a depth of approximately 8 inches above the bottom of the hole. The percolation rate is based on recording water drawdown in the casing at regular time intervals. The test is repeated as needed by re-establishing the water level at 8 inches above the bottom of the casing and measuring the draw down rate. Results from percolation testing are discussed in Section 5.3.

4 LABORATORY TESTING

Laboratory tests were performed on selected soil samples recovered from the borings to confirm our field classifications and to estimate the index properties of the typical materials encountered at the site. The laboratory testing was formulated with emphasis on determining gradation properties, natural water content, and frost characteristics.

Water content tests were performed in general accordance with ASTM D2216. The results of the water content measurements are presented graphically on the boring logs in Appendix A, Figures A-3 through A-31.

Grain size classification (gradation) testing was performed to estimate the particle size distribution of selected samples from the borings. The gradation testing generally followed the procedures described in ASTM C136 and D422, for testing without and with hydrometer analysis, respectively. The test results are presented in Figure A-32 (20 sheets) and summarized on the boring logs as percent gravel, percent sand, and percent fines. Percent fines on the boring logs are equal to the sum of the silt and clay fractions indicated by the

percent passing the No. 200 sieve. Note that sieve and hydrometer testing indicate particle size only and visual classification under USCS designates the entire fraction of soil finer than the No. 200 sieve as silt. Plasticity characteristics (Atterberg Limits results) are required to differentiate between silt and clay soils under USCS.

Selected soil analyses for non-PFAS analytes were submitted to Eurofins Seattle. Samples for PFAS analysis were submitted to Eurofins Sacramento. Details on analysis methods and results are provided under separate cover in our Draft ESR.

5 SUBSURFACE CONDITIONS

The subsurface soil and groundwater conditions encountered in our explorations are described in the following subsections and presented graphically on the boring logs in Appendix A, Figures A-3 through A-31.

5.1 Soil Conditions

In general, soil conditions were relatively similar throughout the project area with relatively clean (i.e., low fines contents) sands and gravels. Groundwater was encountered at relatively shallow depths to the south of Ophir Creek and generally became shallower in depth as borings progressed towards the airport. The following subsections break the project into three separate areas for more detailed descriptions.

5.1.1 Ophir Creek, Pump House, and Well Site Borings

This section discusses the soil conditions encountered in borings advanced for the proposed well, pump house, and adjacent to Ophir Creek (Borings B-01 through B-04, B-26, B-27, and B-29). Where advanced off an existing road embankment, these borings encountered approximately 3 to 12 inches of predominantly organic-rich soil at the ground surface, underlain with complexly interbedded and intermingled, relatively clean sand and gravel layers to the bottom of the borings. However, Boring B-27 encountered intermingled peat and wood debris within the interpreted depth of the road embankment as Airport Road climbed the hill on the north side of Ophir Creek. Fill soils near the Ophir Creek bridge abutments included lenses of peat and what appeared to be creosote wood debris; these soils were generally loose. Cobbles were inferred from drill action to varying depths. Cobble contents noted on the boring logs are estimated by percent volume, based on drill action and samples recovered from the borings. These percentages are difficult to estimate and varied by location, but were generally interpreted to be less than about 20 percent. Fines contents were generally low to moderate, averaging around 5 percent, with occasional higher values of up to 10 percent in the samples tested.

Based on penetration resistance values ranging from 3 to over 50 blows per foot (bpf), the granular soils encountered during drilling measured as loose to very dense. Based on observations of drill action, the soils could generally be categorized as loose to medium dense, with higher penetration resistance likely resulting from driving oversize rock (i.e., cobbles). Samples recovered below approximately 10 feet bgs would occasionally exhibit heaving or flowing sands in the sample spoons, which can bias blow count measurements to be high relative to in-situ soil density when the sampler becomes overfull. Overall, the soils encountered in this area generally exhibited characteristics consistent with glacial outwash sands and gravels, predominantly coarse-grained, with relatively low density and low fines contents. PID readings on all borings were low except the creosote wood in B-26 samples S2 and S3 with 7.9 and 8.4 ppm, respectively. Additional details are provided on the boring logs in Appendix A.

5.1.2 Airport Road Borings

This section discusses the soil conditions encountered in borings advanced through the shoulder of Airport Road between Ophir Creek and Colorado Road (B-05 through B-19) and near the Community Health Center (B-28). Airport Road borings (B-05 through B-19) generally encountered approximately 2 to 7 feet of road embankment fill overlying the natural ground surface at the toe of the embankment slopes. Due to similarities between the road embankment material and underlying native soils, boundaries between fill and native gravelly soil deposits were difficult to ascertain. The embankment height varied, although it generally became thinner heading from Ophir Creek towards the airport. Fill material generally consisted of grayish brown, relatively clean sands and subrounded gravels. Occasionally the fill material was underlain (or in other cases intermixed) with trace to few amounts of organics, wood debris, or peat. Organic soils of this type were observed at depths of up to approximately 8 feet bgs, though this was atypical. Native soils below organics and fill were typically comprised of relatively clean sands and gravels. Fines contents in fill and native soil were generally low, averaging around 4 percent. Note that roadway between the Ophir Creek bridge (Borings B-26 and B-27) and community health center (Boring B-28) had thicker deposits of fill (up to approximately 10 feet) as Airport Road climbed the hill to the north.

The native, granular soils encountered during drilling were generally loose to medium dense with typical penetration resistance values ranging from about 8 to 20 bpf. Outlier penetration resistance values, likely caused by driving oversize rock, skew higher than this range. Many samples recovered below the groundwater table would typically overfill samplers with heaved sand. Overall, the native soils encountered in this area were similar to soils described in Section 5.1.1. PID readings on these borings were less than 10 ppm

when field screened. Additional details are provided in the boring logs presented in Appendix A.

5.1.3 Colorado Road, Endicott Road, and Airport Access Road

This section discusses soil conditions encountered in borings advanced along the proposed water line alignment through airport property. Borings B-20 through B-25 were advanced through the shoulders of gravel access roads and occasionally off the road area and through adjacent native ground. Fill soils, where encountered, were generally thin with an approximate depth of between 2 and 2.5 feet, except for near the intersection of Colorado Road and Airport Road where fill soils were interpreted to nearly 5 feet bgs. Soils with intermixed organics and peat layers, where encountered, were observed to depths of up to 7 feet bgs.

Fill material was generally consistent in gradation characteristics with Airport Road fill, as described in Section 5.1.2. Saturated and wet soils below approximately 2 to 7 feet bgs would typically heave sands into the sample spoon and augers during drilling. Native soils were generally loose to medium dense based on blow values between 4 and 20 bpf. These native soils consisted of intermixed sands and gravels with varying amounts of silt. Fines contents ranged from approximately 3 to 5 percent in the samples tested in the laboratory. These soils were consistent with soils from similar depths in other sections of the alignment.

Boring B-23 was found to have PID values ranging up to 26.7 ppm in Sample S2, collected from 2.5 to 4.5 feet bgs. Soils at and below this depth exhibited an oily sheen and a hydrocarbon odor. Sample S2 from this boring was collected for analytical testing. Additionally, one sample at the soil/groundwater interface was collected from each boring within this section of the alignment for PFAS testing. See our Draft ESR for details and results of these tests. Additional details are provided in the boring logs, Appendix A.

5.2 Groundwater Conditions

Yakutat receives high annual precipitation, estimated at approximately 140 inches. During our explorations, we observed prevalent surface water in roadside ditches, streams, and ponds. Daily precipitation levels during our drilling effort averaged between a half and three quarters of an inch. Precipitation manifested as a mix of rain and snow showers. It should be noted that groundwater levels may fluctuate by several feet seasonally, especially during periods of high precipitation and rapid snow melt. Observed and measured groundwater levels during field explorations are provided on the following page:

Exhibit 5-1: Groundwater Summary

Boring ID	GW Depth While Drilling (bgs)	Drilling Date	Static GW Depth on 12/15/24 (bgs)
B-01	10	12/4/24	8
B-02	7.5	12/6/24	4.6
B-03	5	12/7/24	4.1
B-04	7.5	12/7/24	--
B-05	7.3	12/9/24	--
B-06	9.7	12/9/24	--
B-07	7.3	12/9/24	--
B-08	14.7	12/10/24	10.7
B-09	10	12/10/24	--
B-10	9.7	12/10/24	--
B-11	7.5	12/10/24	--
B-12	4.7	12/10/24	2.9
B-13	4.7	12/10/24	--
B-14	4.7	12/11/24	--
B-15	4.7	12/11/24	--
B-16	7.5	12/11/24	13.9
B-17	7.5	12/11/24	--
B-18	4.7	12/11/24	--
B-19	4.7	12/11/24	--
B-20	7.2	12/12/24	--
B-21	4.7	12/12/24	--
B-22	4.7	12/12/24	--
B-23	4	12/12/24	--
B-24	4.7	12/12/24	4.9
B-25	2.2	12/12/24	--
B-26	10	12/13/24	--
B-27	15	12/13/24	14.7
B-28	--	12/13/24	--
B-29	30	12/13/24	29.7

5.3 Percolation Test Results

Percolation testing was conducted using the procedures described in Section 3.1.4 on December 11, 2024, adjacent to Boring B-02. The soils did not meet the criteria for an immediate percolation test; therefore, the soils at the bottom of the hole were pre-soaked before beginning the test, as specified by the EPA. The soil sample collected at the depth of the percolation test (approximately 1.6 feet bgs) was classified as a *Silty Gravel with Sand* (GM) via laboratory testing, with a fines content of 12.6 percent. Percolation testing yielded a percolation rate of approximately 1.9 minutes per inch (min/in).

6 SEISMIC CONDITIONS AND CONSIDERATIONS

Assuming soil conditions in the top 100 feet of the soil column are similar to those encountered by our explorations at the tank and pump house site and the site is prepared as described below, the site class according to ASCE 7-16 would be D for a stiff soil profile based on the blow count (N) method with in the 15 to over 50 blows per foot range. However, note that because our liquefaction analysis found the native soils at the water tank and pump house site to be potentially liquefiable under the design earthquake, a site response analysis is required unless the exception is met as described in Section 20.3.1 (Exception Number 1). For structures that have fundamental periods of vibration equal to or less than 0.5 s, site response analysis is not required to determine spectral accelerations for liquefiable soils. Rather, a site class is permitted to be determined in accordance with Section 20.3 and the corresponding values of F_a and F_v determined from Tables 11.4-1 and 11.4-2.

Based on the ground motions contained in ASCE 7-16 for Risk Category IV, S_s and S_1 for the Maximum Considered Earthquake were estimated at 1.63 and 0.76, respectively. The site-specific modifying coefficients for the short period spectral response acceleration is $F_A = 1.0$. The SMS was calculated to be 1.63 and the computed SDS is 1.086. Note that because S_1 is greater than 0.2, a site response analysis is required unless the exception is met as described in Section 11.4.8 (Exception Number 2). For Site Class D, the Seismic Response Coefficient (C_s) must fall within the required parameters as described to qualify for the exception. Calculating C_s will require SDS, the appropriate Response Modification Coefficient (R) from Table 12.2-1 (Design Coefficients and Factors for Seismic Force-Resisting Systems), and the Importance Factor determined based on Risk Category.

The calculated peak ground acceleration (PGA) for the site as determined by using software developed by the United States Geological Survey (USGS) was found to be 0.734g. This value is roughly equivalent to what would be calculated using probabilistic estimates of

ground motions with a 2 percent probability of exceedance in 50 years (2,475-year return period).

Based on the soil and groundwater conditions found in our borings, we believe that the native soils have a relatively low risk for slope failure or surface rupture resulting from the effects of seismically induced settlement and strength loss. However, a liquefaction analyses was conducted for the borings near the proposed pump house location (Borings B-01 and B-02); Youd, et al (2001), Cetin et al. (2004), and Idriss and Boulanger (2014). Liquefaction is generally associated with loose, saturated, cohesionless soils. The methods above are specifically intended for cohesionless soils, which are generally granular in nature. However, some fine-grained soils exhibit cohesionless or “sand-like” behavior. Soft, cohesive layers may be subject to strength loss from ground shaking; however, if they exhibit cohesionless behavior, they could be considered liquefiable.

We conducted our analyses using ground motion parameters of 0.734g PGA and an 8.2 Magnitude (M) earthquake. Soils are considered potentially liquefiable when the factor of safety (FS) is generally equal to or less than 1.0 against liquefaction. Based on the analyses, localized zones of liquefaction may be experienced during the design seismic event. We believe that the native soils at the site have a potential for liquefaction with estimated seismically induced settlement and strength loss of approximately 2 to 6-inches or more. Note that it is our opinion that all of this settlement could be differential over very short distances (less than 5 feet). Therefore, we recommend that buried pipes consist of flexible material, such as high-density polyethylene (HDPE) to the extent practicable to reduce the risk of seismically induced settlement damage. We also recommend incorporating a geogrid material into the planned building pad that the tank and pump house will be constructed over. See Section 7.7 for geogrid material recommendations.

7 ENGINEERING RECOMMENDATIONS

Geotechnical considerations associated with this project consist of general earthwork, controlling excavation slopes, controlling construction drainage, planning for potential dewatering needs, expected settlements, backfill and compaction, developing pipe bedding, pavement repairs, and the effects of seasonal frost action around the new structures and water line. Furthermore, the proposed pump house building and water tank foundations must consider the allowable bearing capacity of the support soils, lateral earth pressures, and addressing potential seismically induced settlements.

Based on the conditions encountered by borings in the project area, the soils generally consist of a surficial (approximately 1-foot-thick) organic rich soil layer or road embankment

underlain with native outwash sands and gravels with relatively low to moderate fines contents. Soils along Airport Road were advanced through approximately 2 to 10 feet of embankment fill, with fill depths generally being thickest near Ophir Creek Bridge and thinning as the alignment approaches the airport terminal. Frost classifications ranged between NFS and F2. Proper control of excavation (including construction dewatering) and backfilling activities will be paramount in achieving a well-constructed project.

7.1 Water Line Construction

We understand that the preliminary plan is to bury the new water line at a depth of approximately 5 feet below the surrounding ground surface. We also understand that the new pipe may consist of a combination of HDPE and ductile iron pipe, and that it will generally be located adjacent to the existing asphalt, within the edge of the existing road embankments. Thermal conditions (i.e., seasonal frost depths) also need to be considered to ensure that the water within the pipe does not freeze.

Borings advanced along Airport Road first encountered embankment fill which generally had similar gradation ratios of sands and gravels as native soils, though typically with a slightly higher fines content. Based on the conditions encountered by our borings, the granular material at the approximate burial depth of the proposed water line varied from loose to medium dense. In our opinion, the soils at this depth should be adequate to support the proposed water main improvements, but isolated areas of peat or organic-rich soil will need to be over-excavated and replaced with compacted Selected Material Type A. Likewise, loose mineral soils should either be compacted or excavated and replaced with compacted Selected Material Type A to create a firm, unyielding base to construct the water line over. If loose or organic-rich soils remain in place under the new pipe, potentially damaging differential settlements could develop along the length of the project area. Proper control of excavation (including construction dewatering) and backfilling activities will be paramount in achieving a well-constructed project.

Thermal modelling was performed for the project using the BERG2 computer program to estimate the frost penetration below the ground surface based on subsurface conditions and climate data for the Yakutat/Juneau area. Based on our explorations, we assumed average ground cover over the new water line would consist of 2 inches of asphalt over gravelly soils that became saturated at a depth of approximately 5 feet bgs. Based on these assumptions, the frost depth was estimated to be approximately 5.7 feet bgs, which is deeper than the preliminarily planned burial depth of 5 feet bgs. Therefore, we recommend that the new water line is either buried at a minimum depth so that the top of pipe has at least 6 feet of cover, the pipe is insulated or arctic pipe is used, or insulation is placed in the trench over the water line to reduce the risk of water freezing in the pipe during the winter.

Note that for the purposes of this report we assume that deeper burial depths are not desired due to the relatively shallow depth to groundwater over portions of the alignment and insulated or arctic pipe may be too expensive for the project budget, so we assume that insulation may be placed over the pipe as the utility trench is backfilled.

In general, we recommend that the insulation within the trench, if used, results in an R-value of at least 8.0 hr-ft² °F/BTU. Trench insulation should be placed a minimum of 18 inches below the ground surface and at least 2 feet above the water line, but also at a depth that is anticipated to remain above the depth of groundwater. Based on groundwater depths observed during our explorations, this could be as shallow as approximately 2 feet bgs (groundwater encountered at approximately 2.2 feet bgs in Boring B-25). We recommend that road crossings under Airport Road are designed so that the new water line utility has a minimum of 6 feet of cover and insulation is not required to avoid problems associated with insulated pavements. Also note that further analysis by the pipe manufacturer or others could further evaluate the risk of water freezing in the pipe based on the temperature of water along the length of the water line, flow rates, and the thermal properties of the selected pipe itself.

7.1.1 Trenchless Excavations

We understand that trenchless techniques (i.e., HDD) are preferred for advancing the water line beneath Ophir Creek and potentially at various other road crossings, drainage ditches, or smaller streams along the proposed alignment. We understand that the exact locations of the excavations are not known at this time and will depend on the details of the trenchless excavation and location of new pipe features that will be tied into. Geotechnically, HDD-installed pipe will need to consider the presence of groundwater and excavation requirements for sending and receiving pits. With the sending and receiving pits anticipated to be located adjacent to waterways, potential groundwater inundation into the pits should be planned for. Cobbles were also encountered in borings adjacent to Ophir Creek (i.e., Borings B-03 and B-26) at depths of up to 12.5 feet bgs. Note that B-26 was advanced through embankment fill on Airport Road, with an estimated 10 to 15-foot elevation differential between the road surface and the thalweg of Ophir Creek. The recommendations provided for trench excavation and backfill (Sections 7.1.2 and 7.6, respectively) can generally be applied to trenchless installation, but at a minimum, we recommend that the size of the sending and receiving pit excavations be limited to the extent practicable.

7.1.2 Trench Excavations

Trenches, or excavations needed to access the burial depth of the proposed utility, will be required to install the new water line. Buried utilities exist in relatively close proximity to

the proposed water line in portions of the alignment. We recommend that utilities encountered near an excavation should be adequately supported and braced during construction. Trenches should generally be constructed as presented in Figure 6. Note that near vertical walls may be utilized in roughly the bottom 1 foot of the trench for constructability purposes directly adjacent to the existing paved roadway.

We understand that DOT&PF recently repaved Airport Road and may require that where damaged, the pavement structural section is repaired to meet their standards for structural section composition and compaction requirements. Based on the 65% design drawings for typical water main sections along Airport Road, we understand that utility trench side slopes for the new water main are generally planned to extend to the edge of existing asphalt. In our opinion, excavations made through road embankments for this project will increase the risk of longitudinal cracks developing within existing pavements. This risk will increase the closer that the excavation gets to the existing pavement. While noticeable disturbance to the soils directly underlying the existing pavement may not be observed, we believe that excavation, utility installation, and trench backfilling vibrations will likely loosen the embankment material to the point where these longitudinal cracks may develop over time. Anticipating future conversations with DOT&PF on this topic, we recommend that a contingency is set up to accommodate such potential extra costs associated with repairing existing pavements.

During excavation, mineral soils above the groundwater table may initially tend to stand relatively steeply, due to the apparent cohesion associated with the soil moisture. However, as the soils dry, they will tend to ravel and slough to their natural angle of repose, which for planning purposes is estimated at about 1.5 horizontal (H) to 1 vertical (V). Granular soils below the groundwater table in this area were generally sandy such that undrained soils below the groundwater table could run, slough, or ravel to slopes of 3H to 1V or less. The actual slope and excavation bottom conditions should be made the responsibility of the contractor, who will be present on a day to day basis and can adjust efforts to obtain the needed stability and meet the applicable Alaska and Federal (OSHA) safety regulations.

Groundwater in borings advanced between about Ophir Creek and the airport was generally encountered between approximately 2 and 10 feet bgs during drilling and was later measured at similar depths where observation wells were installed after drilling. Based on our observations during drilling, and the gradation and compactness of the materials encountered, moderate groundwater inflow is anticipated where excavations penetrate the water table, and running ground conditions may occur which make excavation side slopes difficult to maintain if groundwater is not controlled. Recommendations for construction drainage and dewatering are discussed in Section 7.4.

If wet conditions persist at the trench bottom, crushed aggregate may be used to stabilize the trench bottom (i.e., provide a firm unyielding surface on which to support the new pipe) and porous backfill material (see Figure 7) may be used as a substitute for pipe bedding material. This should only be done if it is too wet to compact mineral soils, as porous backfill material may be placed in relatively wet conditions and can be compacted with hand equipment.

We recommend that the contractor be required to submit an excavation plan once the utility layout and depths have been determined. The excavation plan should describe the methods and sequencing for excavation as well as additional information for dewatering and shoring as necessary. The plan should highlight areas that may require dewatering and include details for the type or types of dewatering that will be undertaken (including, but not limited to pumping methods and rates, discharge locations, treatment, etc.). The excavation plan should also include the types and locations of shoring to be used and engineered plans for the shoring if required. Also note that excavations near the existing Ophir Creek Bridge and associated abutment structures may require significantly greater shoring measures to support foundation loading, depending on proximity to these features. We recommend that we be retained to review the excavation plan prior to authorizing work to proceed at the site to ensure that the plan contains the necessary information and is appropriate for the conditions at the site.

7.1.3 Trench Backfill

Where water main trenches require roadway repair, pipe bedding and excavation backfill should be placed in maximum 12-inch loose lifts and compacted to at least 95 percent of the Modified Proctor maximum dry density, as discussed in Section 7.6 below, or per manufacturer recommendations to support and hold the pipe firmly in place. In areas where no road work is planned (undeveloped areas), less compaction is required and material may be placed in thicker lifts (up to 18 inches) and moderately compacted to achieve at least 90 percent compaction. As noted in Figure 6, we recommend bedding the pipe in a minimum of 6 inches of bedding material, or bedding material recommended by the pipe manufacturer. Gradation recommendations are provided in Figure 7.

Utility trenches and excavations should be backfilled with existing inorganic native soils placed in original stratigraphic sequence as much as practicable between the top of pipe bedding and the ground surface or roadway subgrade as discussed in Section 7.5. This procedure limits the contrast between trench backfill and the surrounding soil conditions that can lead to adverse settlement or frost heave behavior. Bulking of backfill into trenches should be discouraged as this can cause variable subgrade support or voids and lead to irregular surface settlements. If existing inorganic soils become difficult to compact during

construction due to the material properties or additional backfill is needed, the contractor may utilize imported Selected Material Type A classified fill, or better, as defined by the DOT&PF Standard Specifications for Highway Construction (SSHC), at the discretion of the Project Engineer.

7.1.4 Estimated Utility Settlements

The magnitudes of the settlements that will develop around the new utilities are dependent upon the applied loads, the gradation properties of the bedding and fill material, and the care with which the bedding and backfill are placed and compacted. Assuming that the loose, organic, or otherwise unsuitable soils are removed from the water line footprint and replaced with compacted Selected Material Type A as described above, it is estimated that total static settlements will generally be limited to the elastic deflection of the bedding, or about ½-inch or less, of which all of it may be differential over a length of approximately 50 lineal feet. However, as discussed in Section 6, we believe the native soils in the project area have a potential for liquefaction with estimated seismically induced settlement of approximately 2 to 6-inches or more. It is our opinion that all of this settlement could be differential over distances of less than 5 feet. Therefore, we recommend that buried pipes consist of flexible material, such as HDPE, to reduce the risk of seismically induced settlement damage to the new water line.

7.2 Foundation Recommendations

The design of the proposed new tank, pump house, and other possibly associated structure foundations must consider the bearing support capabilities of the support soils, expected settlements, and the effects of seasonal frost action around the new structures. Based on preliminary discussions with the structural team for this project, we understand that conventional shallow foundations and frost-protected shallow foundations (FPSFs) are both being considered. Based on the subsurface information acquired during our explorations, we believe that both of these foundation systems are appropriate provided that they are founded on firm, unyielding, native soils that are free from organics or on compacted structural fill placed over firm, unyielding material.

Note that we assume the pump house will be heated continuously (year-round) to a minimum temperature of 63 degrees F, but that the water tank foundation will not be heated to that extent, so it will be considered an unheated foundation. The exposed grade and structural fill placed beneath footings should be placed and compacted in accordance with the recommendations included in Section 7.6.

7.2.1 Site Preparation

Site preparation for structures will be needed at a minimum of within the proposed footprint of the gravel pad that the 500,000-gallon water tank, pump house, leach field, associated ancillary structures, and along portions of the water line alignment. Prior to construction, all surface organics should be cleared and grubbed and removed from the site within the building and gravel pad footprints. In general, all loose, soft, organic, silt, or otherwise unsuitable soils should be removed from beneath these footprints to expose firm, unyielding, native soils. Note that significant clearing and grubbing of existing vegetation, including hemlock and large felled and standing spruce trees, will be required prior to excavation for the tank and pump house area. Additionally, deeper deposits of surficial organic matter beneath the current ground surface may be encountered and need to be removed. If unsuitable soils are observed at the proposed depth of structure foundations, the unsuitable materials should be over-excavated and replaced with compacted Selected Material Type A. The horizontal extents of the over-excavation should be defined by a line extending down and out from the perimeter of the exterior footings or outer edge of horizontal insulation (if a FPSF is used) at a slope of 1H to 1V. In our opinion, unsuitable soils that are excavated should be hauled away from the site or they may be used for landscaping if it is not too wet to properly stockpile and place.

Once firm, unyielding mineral soils are exposed throughout the area of the building footprint and over-excavation, the ground surface at the bottom excavations should be proof rolled and observed by an experienced geotechnical professional for unsuitable conditions, including localized areas of soil that may have become soft or loose from disturbance during construction. If required, we recommend backfilling with well-graded sand and gravel to raise portions of the site up to the design foundation subgrade elevation. Backfill should generally consist of classified fill that is placed and compacted with moisture density control as described below in Section 7.6. Classified fill should generally consist of materials that meet DOT&PF SSHC specifications for Selected Material Type A fill. Further details on gradation characteristics are provided in Figure 7.

We understand that a large gravel pad is proposed to be constructed at the site of the water tank and pump house to support those and other associated structures. We recommend that once firm, unyielding mineral soils are exposed throughout the footprint of the proposed gravel pad, a layer of biaxial geogrid should be placed within the exposed footprint of the pad as described in Section 7.7. The gravel pad embankment should then be constructed using Selected Material Type A that is placed and compacted in 12-inch maximum lifts (see Section 7.6) up to the design embankment elevation. In our opinion, a single layer of geogrid placed at the bottom of the gravel pad and an additional layer placed either 12 inches above the base layer of geogrid or a minimum of 6 inches below concrete

footings of structures would be sufficient to support the proposed structures. However, additional layers of geogrid may be placed between additional layers of the fill pad to further increase the strength of the pad and reduce the risk of differential, seismically induced settlements that may develop during a design earthquake at this site. Note that we recommend that there is a minimum of 6 inches of Selected Material Type A placed between the bottom of the foundation and the upper layer of geogrid. As described in Section 7.5, a 4-inch surface course (i.e., E-1) is presented as an optional driving surface. We also recommend establishing a crown or sloping the gravel pad surface at a minimum slope of 3 percent in all directions to encourage drainage of water off the gravel pad surface.

Care should be taken to minimize disturbance of the excavation bottom by digging/excessive tracking by equipment in areas that are below the groundwater level and/or may be moisture sensitive. In addition, if existing soils become disturbed and or wet, construction could be difficult if the contractor is not able to control and compact fills that are placed.

7.2.2 Conventional Shallow Foundations

We recommend that continuous footings are sized such that the width is at least 16 inches and that spread footings should be a minimum of 2 feet wide. Footings should be buried sufficiently to prevent structural damage resulting from frost action. We recommend that perimeter footings in the heated (pump house) building be placed a minimum of 42 inches below the finish grade. For interior footings in heated areas, footings may be placed directly below the floor slab such that embedment is 18 inches or more below the finished floor elevation. If portions of the proposed building are to be unheated, the minimum burial depth for footings should be increased to 5 feet bgs for frost protection.

Based on the recommended footing dimensions, depths, and site preparation recommendations, we recommend that foundations for the proposed building be designed with an allowable soil bearing pressure of 3,000 pounds per square foot (psf). Localized loose or soft areas, whether resulting from existing conditions or disturbance during construction must be corrected prior to casting footings or damaging differential settlements could occur. The above bearing value may be increased by 1/3 for short-term wind or seismic loading. Figure 3 presents a conventional floor slab and footing detail.

7.2.3 Frost-Protected Shallow Foundations

We understand that thickened edge slabs are also being considered for the water tank and pump house foundations. For geotechnical purposes, we refer to this foundation system as FPSF because the shallow burial depths of the slabs typically require insulation to protect the structure from frost-related movements (i.e., frost heaving). We recommend that

buildings supported by a FPSF bear on compacted structural fill placed over firm, unyielding material. The exposed grade and structural fill placed beneath footings should be placed and compacted in accordance with the recommendations included in Section 7.6.

Our recommendations and allowable bearing pressures are based on support soils consisting of loose to medium dense granular materials, and assume the site is prepared and in accordance with Section 7.2.1. We used the design guidance from the Structural Engineering Institute/American Society of Civil Engineers (SEI/ASCE) 32-01, which provides prescribed design criteria regardless of the supporting soil. It is required with FPSF construction that the foundations be placed on NFS fill with a thickness equal to or greater than the design frost depth for the locale, or the outer edges of the slab must be protected from frost with a layer of insulation along the outside vertical edge of the stem wall, and horizontally away from the bottom of the thickened edge slab. In Yakutat, based on our BERG2 thermal analysis, we estimate frost penetration may extend to a maximum depth of approximately 6 feet bgs in areas without an organic mat that are kept clear of snow. Based on laboratory testing, existing fill and native soils may be non- to moderately frost susceptible with frost classifications ranging from NFS to F2. Therefore, we recommend protecting the foundations with insulation in accordance with ASCE 32-01 where NFS fill (i.e., Selected Material Type A) does not extend to a depth of at least 6 feet bgs.

Insulation should be protected in accordance with Section 4.1.4 of the SEI/ASCE 32-01. This states that vertical insulation placed externally on foundation walls or along edges of slab-on-grade floors should have an opaque and weather-resistant protective covering to mitigate degradation of thermal performance from ultraviolet radiation, physical damage, or other sources of deterioration. Protective coverings or coatings should be compatible with the insulation material and extend a minimum of 6 inches below finished grade. Polystyrene insulation should not be exposed to petroleum-based products. Horizontal insulation placed less than 12 inches below the ground surface, or any portion extending outward more than 24 inches from the foundation edge, should be protected against damage by concrete or asphalt pavement on the ground surface directly above the insulation by cementitious board or plywood rated for below-ground use, or by other approved materials placed directly on the top surface of the insulation.

Assuming the site is prepared according to the recommendations in Section 7.2.1, and considering the recommendations described above, we recommend that the footings or thickened edge portion of the slab be designed using an allowable bearing capacity of 2,500 psf. This bearing value may be increased by 1/3 for short-term wind and seismic loading. Embedment for interior footings and the thickened edge slab should be at least 12 inches

below finished grade. Minimum footing dimensions should be 16 inches for perimeter footings and 2 feet for spread footings.

7.2.3.1 Heated Structure

The thickened portion of the slab below heated structures should be embedded to a minimum depth of 16 inches bgs, and the horizontal insulation should be placed at this depth. This report presents the required R-values for each of the insulation locations around the building. This allows the contractor to select the type of insulation that they prefer. Provided that the burial depth prescribed above is followed, the horizontal insulation along the wall should have a required R-value of at least 1.7 hr-ft² °F/BTU with a minimum width of horizontal insulation along the wall of 12 inches. Additionally, the insulation should extend horizontally a minimum of 12 inches beyond the outer edge of any foundation element. The width of the horizontal insulation at the corners of the building should be increased to a minimum of 24 inches and the required R-value for horizontal insulation at the corners of the buildings is 4.9 hr-ft² °F/BTU. The 24-inch width and higher R-value should be used for horizontal insulation layers within 40 inches of the corners of the building.

It should be noted that the required R-values of horizontal insulation and the width of the insulation away from the building wall are interdependent. If the width of the horizontal insulation is different from the assumed 12 or 24 inches, Shannon & Wilson should be notified immediately so that the required R-values given in this report can be modified accordingly. Assuming that the vertical stick-up of the slab above the ground surface is no more than 12 inches, the required R-value in the vertical insulation is 6.7 hr-ft² °F/Btu. Figure 4 presents a typical heated building FPSF detail depicting the recommended insulation installation. These recommendations assume that the minimum average monthly interior temperature of the building will remain above 63 degrees F. If areas within the building are expected to be kept below an average temperature of 63 degrees F throughout the winter, the FPSF design in these areas should be evaluated for additional frost protection.

7.2.3.2 Unheated Structure

FPSF design for unheated structures requires a layer of NFS soil beneath the entire footprint of the structure and extending beyond the outer edge of the thickened portion of the foundation. An insulation layer is also required beneath the entire structure and extending beyond the outer edge of the foundation. The insulation for unheated structures should be placed over a minimum of 6 inches of bedding material and at least 12 inches beneath the bottom of the thickened edge slab, which will result in a minimum burial of 24 inches below the ground surface. The minimum horizontal extent of insulation beyond the outer edge of

the unheated slab is 51.5 inches in all directions. The required minimum R-value (R_g) for the horizontal ground insulation layer is $10.9 \text{ hr-ft}^2 \text{ }^\circ\text{F/Btu}$ assuming a minimum burial depth of 24 inches below the surrounding ground. Note that the insulation type selected by the contractor should have sufficient allowable bearing capacity to support the loads upon it including the soil, concrete, tank, and its contents. Figure 5 presents a typical detail of this configuration with the insulation installed. Note that the unheated FPSF configuration is appropriate for use under heated structures as well.

7.2.4 Floor Slab Support

We assume a concrete-on-grade floor slab will be used on the interior floor of the pump house and that the water tank will be supported by a concrete slab, directly beneath the bottom of the tank. Assuming the site is prepared in accordance with the recommendations in Section 7.2.1, we recommend that floor slabs be placed on a minimum 6-inch lift of bedding material founded on firm, unyielding native soil or compacted fill soils. Gradation requirements for bedding material are presented in Figure 7. All fill materials under the slab should be placed and compacted as recommended in Section 7.6. Provided the recommendations discussed above are adhered to by the contractor, a subgrade reaction modulus of at least 150 pounds per square inch per inch (psi/in) should be attainable for concrete on the recommended support soils. In areas to receive floor coverings, we recommend installing a vapor retarder directly beneath the concrete slab. Typical floor slab details for a conventional shallow foundation system and an FPSF system are presented in Figures 3 through 5.

7.2.5 Estimated Building Settlements

The magnitude of total settlements that will develop at the water tank and pump house site are dependent upon the actual loads that are applied, the footing sizes, density of support material, and the care with which structural fills are placed and compacted beneath foundations. Assuming the site is prepared as recommended and the subgrade beneath footings is protected from moisture while exposed, we estimate total settlements of about 1-inch or less could occur and that differential settlements will be about 1/2 of the total over a horizontal distance of 50 feet. We estimate that these settlements should develop almost elastically as the loads are applied such that long term differential settlements will be relatively small and well within tolerable limits. It is estimated that long term differential settlements of these structures should be about 1/4 inch or less.

As discussed in Section 6, we believe the native soils in the project area have a potential for liquefaction with estimated seismically induced settlement of approximately 2 to 6-inches or more. It is our opinion that all this settlement could be differential over distances of less than 5 feet. Therefore, we recommend that buried pipes consist of flexible material, such as

HDPE, and that these possible settlements are accounted for in foundation design to reduce the risk of seismically induced settlement damage to structures.

7.3 Temporary Shoring and Lateral Earth Pressures

Temporary shoring may be needed to support excavation sidewalls during construction, for foundation walls below ground, and retaining walls that support earth fills and floor slabs. The shoring and foundation elements should be designed to resist lateral earth pressures. For this project, we assume that the shoring would consist of a prefabricated, steel “trench box” or a cantilevered sheet pile wall structure. We assume a maximum height of 10 feet for trenches and HDD sending and receiving pits. Note that this information is intended to apply for excavations that are not load-affected by Ophir Creek Bridge or other features along the alignment. Depending on proximity to these features, active earth pressures and surcharge loading may be significantly greater than what is described below. The shoring plan for excavations near these features will require a design developed by experienced geotechnical and structural engineers.

Assuming an active earth pressure condition under static loading and drained conditions, an equivalent fluid weight of 37 pounds per cubic foot (pcf) is recommended for design of the shoring. This active earth pressure condition would also apply to walls that are allowed to deflect laterally or rotate an amount equal to about 0.001 times the height of the wall. For rigid walls that are restrained from deflecting at the top, an at-rest earth pressure condition would prevail and an equivalent fluid weight of 58 pcf is recommended. To simulate seismic loading, the active earth pressure should be increased with a uniformly distributed, rectangular pressure prism of 19 pounds per square foot (per linear foot of structure) of wall height should be applied to the below-grade walls. These values do not include a factor of safety.

Lateral forces may be resisted by passive earth pressures against the structure walls which can be estimated using an equivalent fluid weight of 350 pcf. This value includes a factor of safety of at least 1.5 on the full passive earth pressure, but should be reduced to 150 pcf if saturated conditions exist in the embedded portion of the shoring. Lateral resistance may also be developed in friction against sliding along the base of foundations placed on grade such as footings or floor slabs. These forces may be computed using a coefficient of 0.4 between concrete and soil

Note that these values reflect free-draining, compact, granular backfill or native soils with no hydrostatic forces acting on the wall. Active and passive pressures are prismatic and act on the wall accordingly. Passive pressure was calculated using buoyant unit weights assuming the soils below the base of the excavation are saturated. Shoring requirements to

prevent sidewall failures and undercutting of nearby structures, roadways, and utilities should also consider wall side and surcharge loads may occur if they fall within a plane line extending out and up at a slope of 1H to 1V from the outer edge of the bottom of the excavation.

7.4 Construction Dewatering and Drainage

Based on our measurements in the piezometers installed in our borings, static water levels between Ophir Creek and the airport were typically about 3 to 8 feet bgs, but it was found to be deeper through portions of the alignment. Note that groundwater was generally deeper north of Ophir Creek as the ground elevation was higher at the townsite. These groundwater depths suggest that groundwater will be encountered during sending and receiving pit excavation work, water line trenching, and may be encountered during excavation work at the water tank and pump house site depending on the bottom depth of the proposed gravel pad. A moderate inflow of groundwater will likely be experienced in open excavations below the groundwater table. The amount of water encountered will depend on the contractor's excavation plan, seasonal fluctuations in the water table, depth and size of the excavations, and other factors. In our opinion, dewatering with sumps and pumps should generally be adequate to control groundwater during construction where excavations that do not penetrate more than 2 to 3 feet into water bearing zones; however, area-wide dewatering with well points or other dewatering methods may be required where excavations extend more than several feet below the water table (i.e., for Ophir Creek sending and receiving pits). These measures may also need to be used in tandem with temporary shoring.

Based on the gradation testing conducted, we estimate that average dewatering flows for a 10-foot-wide by 50-foot-long trench that extends 3 feet below the water table will be on the order of 150 to 300 gallons per minute (gpm) or less, but this could vary along the length of the alignment. Higher flows could be encountered if layers of more permeable materials are encountered in the trench. Based on groundwater depths in borings adjacent to Ophir Creek, up to roughly 6 feet of dewatering may be required for sending and receiving pit excavations. Based on our experience, and for preliminary planning purposes for entry and exit point excavations, we estimate initial pumping rates on the order of 200 to 400 gpm assuming 20-foot by 20-foot excavation and a drawdown on the order of 6 feet. The rates will decrease as the soils drain, such that pumping rates are roughly half of the initial pumping rates after seven days of pumping.

These rates and associated drawdowns are for planning purposes only and should be verified in the field. The relatively wide range of estimated discharge rates are a product of uncertainty associated with the empirical methods used to estimate hydraulic conductivity,

our assumption regarding excavation geometry, and the construction means and methods. Additional testing and analyses would be needed to refine our knowledge of the aquifer characteristics and the estimated pumping rates. It is also likely that permits from the Alaska Department of Natural Resources (DNR), ADEC, and other agencies will be required for construction dewatering.

Another critical water condition during construction will likely be surface water, runoff or direct precipitation entering excavations. Thus, the ground around open excavations should be contoured to direct surface water around the excavations. Excavation and backfilling work should be closely coordinated such that seepage and surface runoff is not allowed to collect and stand in open trenches for long time periods. Seepage from the excavation walls may cause local running or sloughing of the soil, which may require the use of a trench box or shoring depending on the excavation slope angles and depth of the excavations. Exposed fine-grained soils should be protected from additional moisture during construction as they are likely moisture sensitive and may lose significant strength if saturated.

7.5 Asphalt Pavement Repair and Reconstruction

We understand that DOT&PF recently (within the last four years) repaved Airport Road. We also understand that significant lengths of trenches for the new water line will be advanced through the side slope of the existing Airport Road embankment, as well as other road embankments, and that asphalt pavements may be damaged during construction. Where damaged, asphalt patching or possible reconstruction will be required. Based on our experience obtaining the ROW permit prior to the start of drilling, DOT&PF will want the relatively new asphalt on Airport Road undamaged to the extent practicable or repaired to their standards. While we do not have the Airport Road as-built drawings as of the time of this report, asphalt repairs to that roadway should generally match the existing pavement structural section, including 6 inches of D-1 base course directly beneath the asphalt surface.

We assume that the roadways within the project area will continue to be used for light residential traffic and occasional truck traffic for maintenance and other services, similar to its current traffic mix and volume. We also assume there will be no full-width reconstruction of asphalt pavements or any pavement construction more severe than patching the cuts made to install the proposed water main to its design depth. The existing soils can provide a suitable subgrade support for roadway pavements if the section is designed to accommodate the frost susceptibility of the soils. Based on laboratory testing and our observations, the subgrade soils encountered in the project area exhibited typically low to moderate frost susceptibility with frost classifications of NFS to F2.

We assume that excavation sizes will be minimized to reduce the amount of structural section that will need to be removed during construction. Therefore, repair work should focus on constructing replacement structural section materials that will behave similarly to the remaining roadway. To accomplish this, the thickness and materials used to repair the pavement structural section and asphalt should generally match the existing conditions. In areas where matching the existing structural section is not feasible (i.e., silty, organic, or debris material in the embankment), we recommend replacing the unsuitable material with Selected Material Type A. While only sporadically encountered in our borings advanced along existing roadways, we do not recommend reusing existing materials with more than about 10 percent fines in the pavement structural section. However, these soils may be reused between the Bedding material and structural section if the contractor is able to properly place and compact the material with moisture-density control, as described in Section 7.6.

If portions of the project outside of the existing paved roadways are planned to be paved, we assume that the traffic would primarily consist of relatively slow moving, lightly loaded passenger vehicles and occasional maintenance and other service vehicles that could be heavier. Therefore, we recommend that these areas, if paved, consist of the following:

Exhibit 7-1: New Pavement Structural Section

Thickness, inches	Layer Type	Material
2	Wearing Course	Hot Mix Asphalt (HMA)
2	Base Course	D-1
24	Subbase	Selected Material Type A

A large gravel pad is planned to be constructed at the site of the water tank and pump house to support those structures. We recommend that this pad is constructed as described in Section 7.2.1. Based on those recommendations, the gravel pad embankment should then be constructed using Selected Material Type A that is placed and compacted in 12-inch maximum lifts (see Section 7.6) up to the design embankment elevation. We understand that the surface of this pad will not be paved; therefore, we believe that a 4-inch surface course (i.e., E-1) may be placed at the ground surface to provide an optional driving surface that should perform better than using Selected Material Type A on the pad surface. Gradation requirements for E-1 Surface Course are presented in Figure 7.

7.6 Structural Fill and Compaction

Backfill will be required behind the foundation walls and under foundations and floor slabs for the pump house and 500,000-gallon water tank. Structural fill that is placed should be

clean, well-graded, granular soil to provide drainage and frost protection. Selected Material Type A as defined by the DOT&PF SSHC meets these requirements and may be placed in both wet and dry conditions. Gradation requirements for Selected Material Type A are provided in Figure 7.

Existing sub-organic soil materials encountered in our borings (including material removed for construction activities) typically contain low (3 to 9 percent) fines based on our laboratory testing. The soils are generally saturated and full of oversize and should not be used as structural fill under building foundations without further sorting. In nonstructural areas, any material placed and compacted should utilize proper moisture density control and the fines content should be less than 20 percent.

Structural fills below footings and floor slabs should be placed in lifts not to exceed 12 inches loose thickness and compacted to 95 percent of the maximum density as determined by the Modified Proctor compaction procedure (ASTM D1557). Non-structural fills should be placed in similar lifts and compacted to at least 90 percent of the Modified Proctor maximum density. We recommend the removal of any cobbles more than an 8-inch diameter from either Selected Material Type A or native material (used where appropriate) as backfill. We recommend that our services be retained to inspect the quality of fill compaction during construction. Backfills at the groundwater interface will resist compaction and structural fills should be placed in a dewatered environment as per Section 7.4.

When backfilling in and around the foundation thickened edge stem walls, material should be placed in layers not to exceed 6 inches loose thickness and densely compacted with hand operated equipment. Heavy equipment shall not be used as it could cause increased lateral pressures and/or disruptive vibrations and damage walls.

7.7 Geogrid Material

We have included recommendations for incorporating a geogrid into the building pad beneath the tank and pump house to increase the tensile strength and stability of the structural fill pad. By increasing the tensile strength of the soils, potential differential settlements due to liquefaction should be decreased both longitudinally and laterally from the center of the pad. We recommend installing the geogrid layer below the proposed gravel pad, at the bottom of the site preparation excavation, and constructing the pad with compacted lifts (see Section 7.6) of Selected Material Type A backfill.

This geogrid layer should be placed per the manufacturer's recommendations. In the absence of manufacturer recommendations, we recommend that there should be at least 3 feet of overlap between grid sheets, with seams sewn as recommended in the standard

specifications from the product manufacturer. End seams should also be staggered by a distance equal to the roll width, and traffic on top of the initial lift over the geogrid should travel in straight lines to prevent damage. We recommend that geogrid used for this project should consist of Mirafi® BXG120, or equivalent.

Exhibit 7-2: Geogrid Properties (Mirafi® BXG120)

Mechanical Properties	Minimum Average Roll Value
2% Strain Tensile Strength by ASTM D6637	410 lb/ft (MD) 620 lb/ft (CD)
5% Strain Tensile Strength by ASTM D6637	810 lb/ft (MD) 1340 lb/ft (CD)
Ult. Strain Tensile Strength by ASTM D6637	1310 lb/ft (MD) 1970 lb/ft (CD)
Grid Aperture Size	1.0 (MD) 1.3 (CMD)
Resistance to Installation Damage	95% (SC), 93% (SW), 90% (GP)
Resistance to Long Term Degradation	100%
Resistance to UV Degradation	100%

8 CLOSURE AND LIMITATIONS

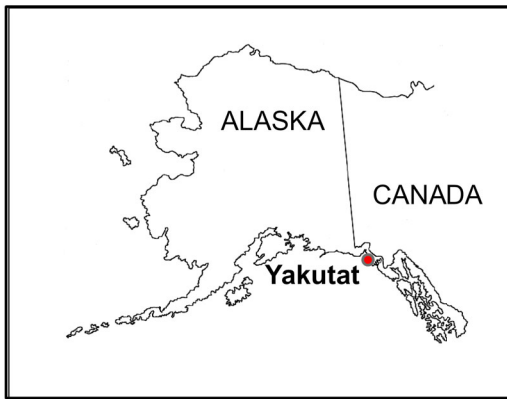
This report was prepared for the exclusive use of our client and their representatives for evaluating the site as it relates to the geotechnical aspects discussed herein. The analyses, conclusions, and recommendations contained in this report are based on site conditions as they presently exist, and further assume that the explorations are representative of the subsurface conditions throughout the site; that is, the subsurface conditions everywhere are not significantly different from those disclosed by the explorations. If subsurface conditions different from those encountered in the explorations are encountered or appear to be present during construction, we should be advised at once so that we can review these conditions and reconsider our recommendations, where necessary. If there is a substantial lapse of time between the submission of this report and the start of construction at the site, or if conditions have changed because of natural forces or construction operations at or adjacent to the site, we recommend that we review our report to determine the applicability of the conclusions and recommendations.

Within the limitations of scope, schedule, and budget, the analyses, conclusions, and recommendations presented in this report were prepared in accordance with generally accepted professional geotechnical engineering principles and practice in this area at the time this report was prepared. We make no other warranty, either express or implied. These conclusions and recommendations were based on our understanding of the project as described in this report and the site conditions as observed at the time of our explorations.

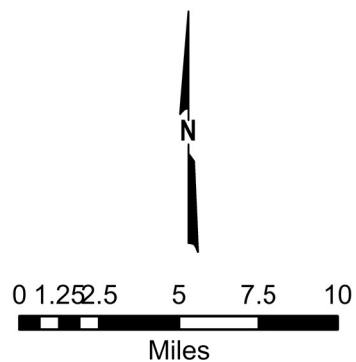
Unanticipated soil conditions are commonly encountered and cannot fully be determined by merely taking soil samples or advancing borings. Such unexpected conditions frequently require that additional expenditures be made to attain a properly constructed project. Therefore, some contingency fund is recommended to accommodate such potential extra costs. Shannon & Wilson has prepared the attachments in *Important Information About Your Geotechnical/Environmental Report* to assist you and others in understanding the use and limitations of the reports.

Copies of documents that may be relied upon by our client are limited to the printed copies (also known as hard copies) that are signed or sealed by Shannon & Wilson with a wet, blue ink signature. Files provided in electronic media format are furnished solely for the convenience of the client. Any conclusion or information obtained or derived from such electronic files shall be at the user's sole risk. If there is a discrepancy between the electronic files and the hard copies, or you question the authenticity of the report please contact us.

We appreciate this opportunity to be of service. Please contact us with questions or comments concerning the contents of this report.



State of Alaska, Esri Canada, Esri, TomTom, Garmin, SafeGraph, FAO, METI/NASA, USGS, EPA, NPS, USFWS, NRCAN, Parks Canada, Esri, CGIAR, USGS



Yakutat Airport Water Line Extension
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VICINITY MAP

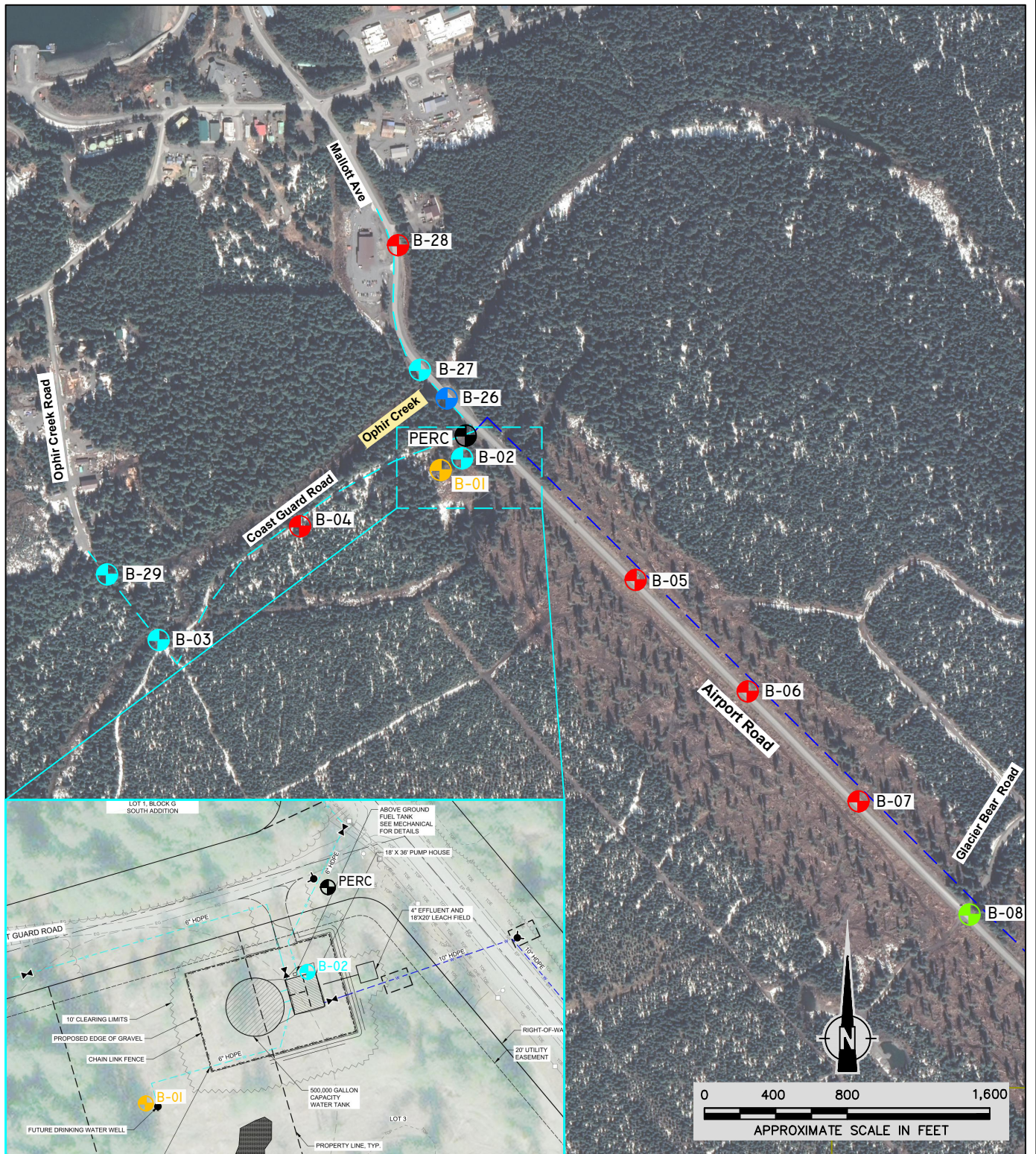
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FIG. 1



LEGEND

- | | | | |
|--|-------------------------|--|---------------------------|
| ● B-04 | 15-foot Boring | ● B-08 | 15-foot Boring with piezo |
| ● B-26 | 30-foot Boring | ● B-03 | 30-foot Boring with piezo |
| --- | Proposed 6" Water Mains | ● B-01 | Well Boring with screen |
| --- | Proposed 8" Water Mains | ● PERC | Percolation Test location |

NOTES

- Map adapted from aerial imagery provided from Google Earth, May 2021.

Yakutat Airport Water Line Extension Yakutat, Alaska

SITE PLAN

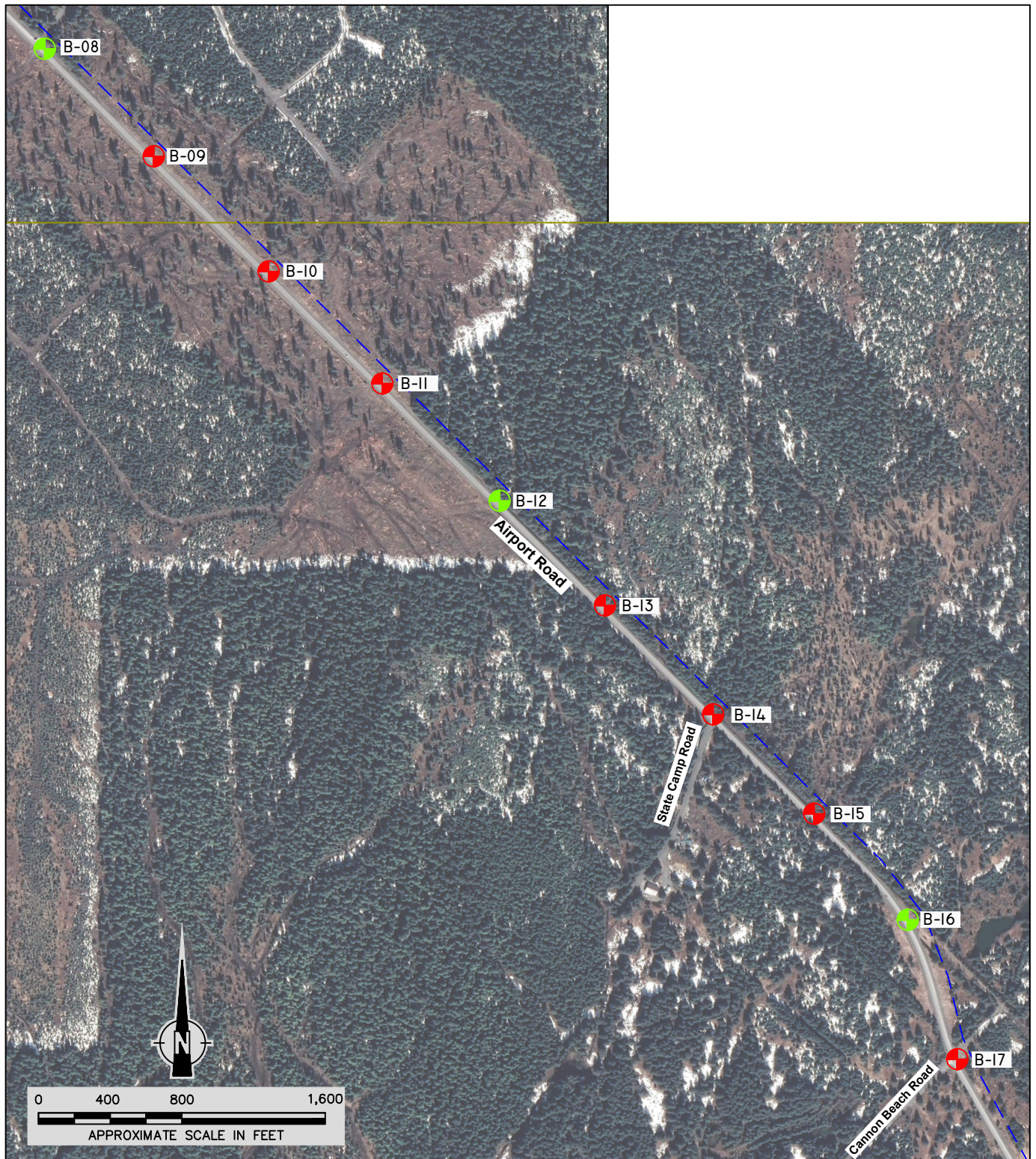
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FIG. 2
Sheet 1 or 3



LEGEND

- B-09 15-foot Boring
- B-08 15-foot Boring with piezo
- Proposed 6" Water Mains
- Proposed 8" Water Mains

NOTES

1. Map adapted from aerial imagery provided from Google Earth, May 2021.

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SITE PLAN

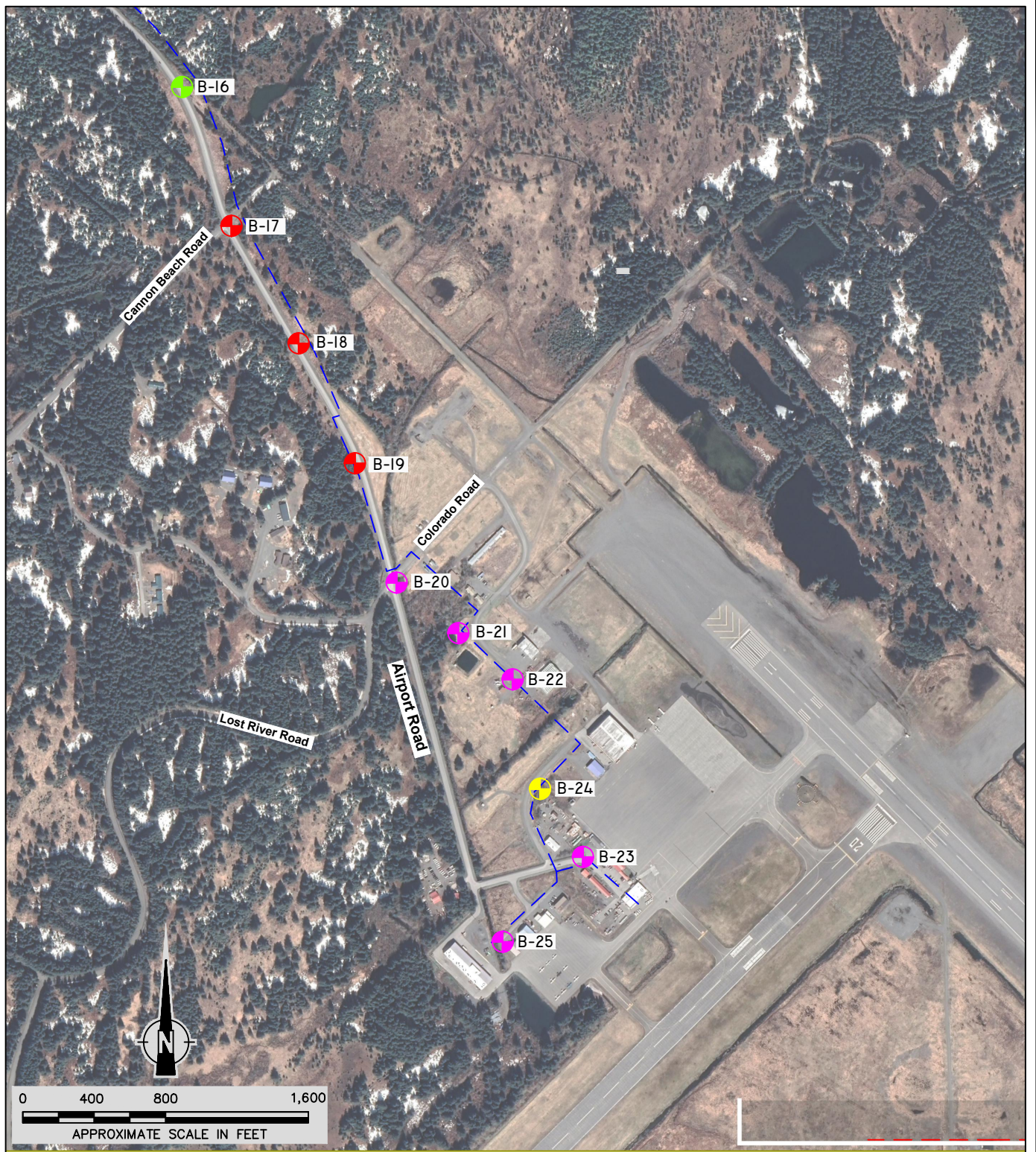
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FIG. 2
Sheet 2 of 3



LEGEND

- | | | | |
|--|---------------------------|--|---|
|  B-17 | 15-foot Boring |  B-20 | 15-foot Boring with PFAS sample |
|  B-16 | 15-foot Boring with piezo |  B-24 | 15-foot Boring with PFAS sample and piezo |
|  | Proposed 6" Water Mains | | |
|  | Proposed 8" Water Mains | | |

NOTES

1. Map adapted from aerial imagery provided from Google Earth, May 2021.

Yakutat Airport Water Line Extension
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SITE PLAN

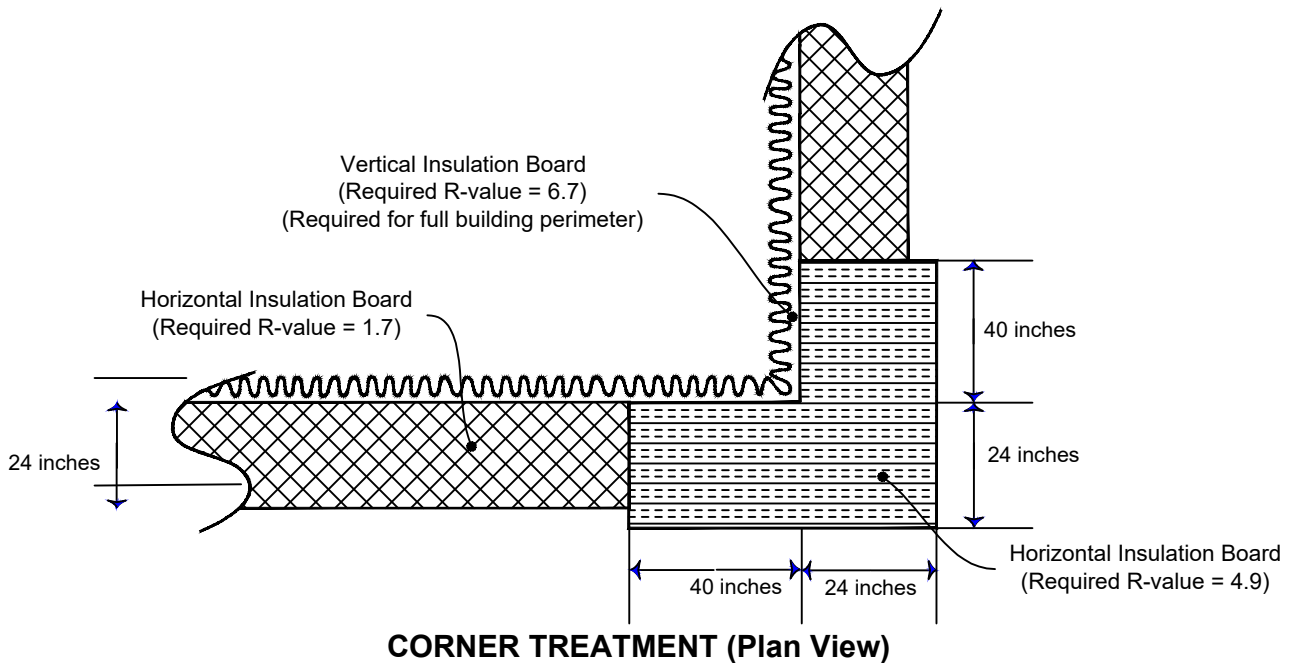
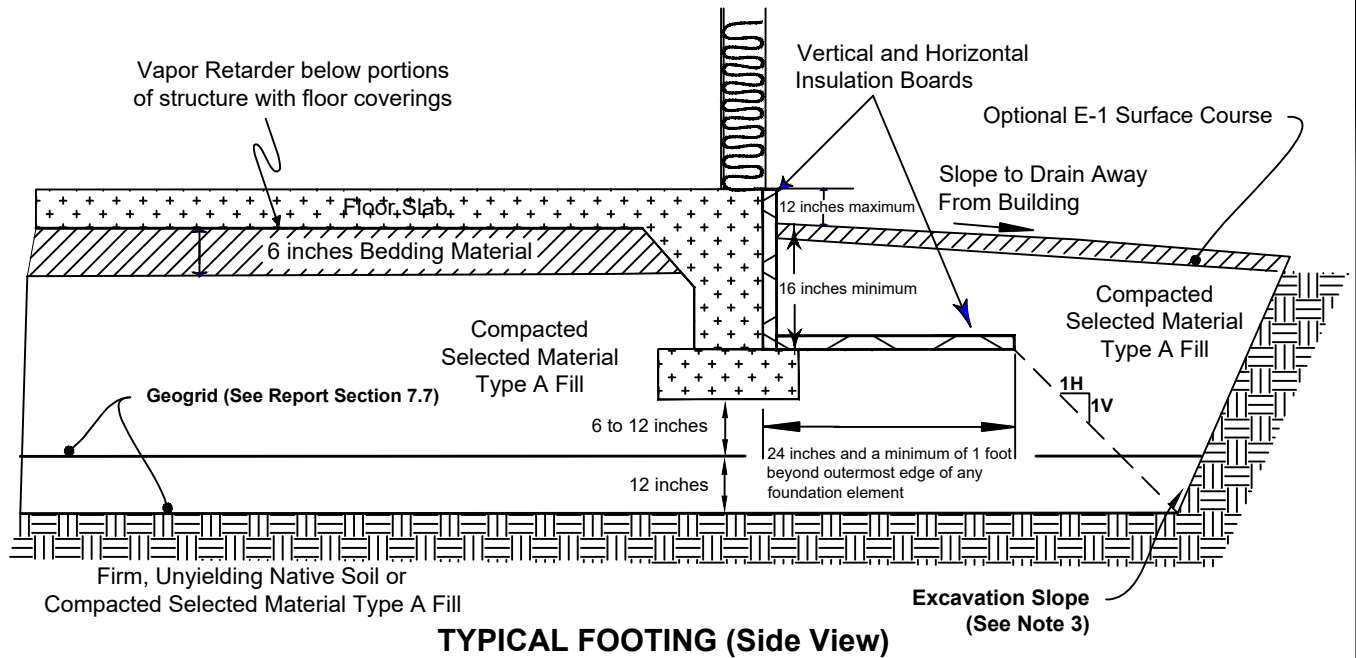
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FIG. 2
Sheet 3 of 3



NOTES

- Design based on over-excavating existing unsuitable soils (i.e., loose, soft, silty, or soils containing organics) and replacing with compacted Selected Material Type A. Backfill below footings and floor slab with free-draining granular soil with not more than 6 percent (by weight, based on minus 3-inch portion) passing the No. 200 sieve (by wet sieving) with no plastic fines. Insulation may not be necessary if NFS material thickness extends 6 feet or more below the ground surface.
- Fill should be placed in layers not exceeding 10 to 12 inches loose thickness and densely compacted. Beneath footings, floor slabs, or sidewalk areas, compact to at least 95 percent Modified Proctor maximum density (ASTM D1557). Otherwise compact to 90 percent minimum.
- Backfill within 18 inches of wall should be placed in layers not exceeding 6 inches and densely compacted with hand-operated equipment. Heavy equipment should not be used for backfill, as such equipment operated near the foundation could increase lateral earth pressures and possibly damage the structure.
- The required R-values shown above should be divided by the R factor (R per inch) of the insulation board selected for construction to determine the thickness of insulation required in the vertical, central horizontal, and corner horizontal areas of the building to protect the foundation from frost action.

* Required R-values are in terms of hr-sq ft-deg F/Btu Drawing Not to Scale

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FROST-PROTECTED SHALLOW FOUNDATION DETAIL (HEATED STRUCTURE)

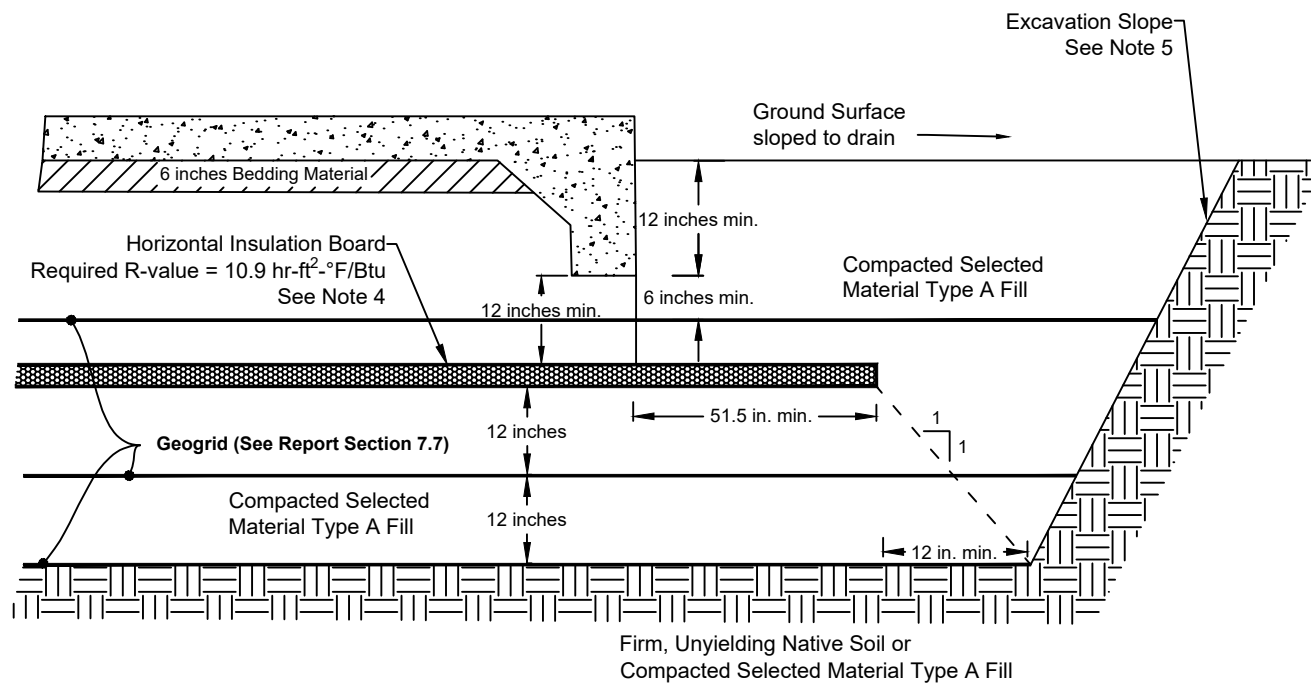
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FIG. 4



TYPICAL FOOTING (Side View)

Notes:

1. Fill material below ground insulation should consist of compacted structural fill that meets the gradation requirements for Selected Material Type A with a maximum particle size of 3 inches.
2. Fill should be placed in layers not exceeding 12 inches loose thickness and densely compacted. Beneath footings and driving areas compact to at least 95 percent Modified Proctor minimum density (ASTM D1557). Otherwise compact to 90 percent minimum.
3. Backfill within 18 inches of wall should be compacted with hand-operated equipment. Heavy equipment should not be used for backfill as such equipment operated near the foundation could increase lateral earth pressures and possibly damage the structure.
4. Required R-value and insulation board is based on burial depth and configuration shown above. Insulation board type should be strong enough to support the loads upon it including the soil and concrete as well as the tank and its contents. If board is buried deeper than shown, different R-values or insulation strength/type may be needed.
5. OSHA requires slope protection and support for all trenches greater than 4 feet deep. Side slope requirements are variable depending upon soil type and the duration of time in which the trench remains open. The contractor should be made responsible for compliance to these regulations as they are at the project on a day to day basis and are aware of the changing conditions.

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FROST-PROTECTED SHALLOW FOUNDATION FOOTING DETAIL (UNHEATED STRUCTURE)

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Drawing Not to Scale

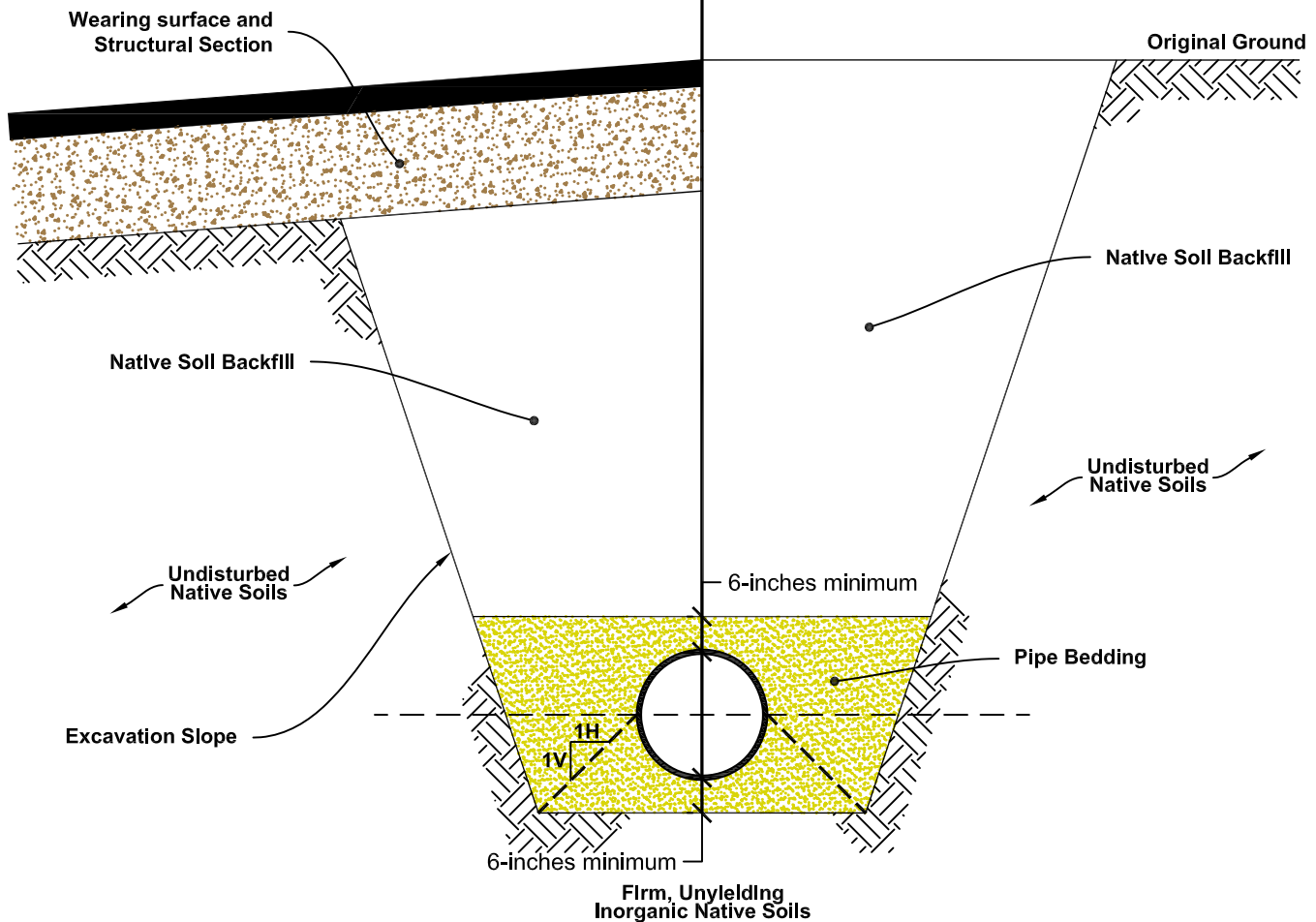


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FIG. 5

Trench Under Paved Areas

Trench Under Non-Structural Areas



NOTES

1. Trench backfill under paved areas should be placed in loose lifts not to exceed 12 inches and compacted to at least 95 percent of its maximum dry density as determined by ASTM D1557.
2. Trench backfill under non-structural areas should be placed in loose lifts not to exceed 18 inches and compacted to at least 90 percent of its maximum dry density as determined by ASTM D1557.
3. Pipe bedding should conform to Selected Material Type A (modified as shown on Figure 7) or as recommended by pipe manufacturer.
4. Pipe bedding and cover thickness shown above should be used absent pipe manufacturer requirements.
5. OSHA requires slope protection and support for all trenches greater than 4 feet deep. Side slope requirements are variable, including near vertical in the bottom foot of the trench, depending upon soil type and the duration of time in which the trench remains open. The contractor should be made responsible for compliance to these regulations as they are at the project on a day to day basis, is aware of the changing conditions and has authority to direct work.
6. If used, insulation should be placed above the pipe in the trench shall have a minimum R-value of 8.0 hr-ft-deg F/Btu. The insulation shall be placed a minimum of 18 inches below the ground surface, 2 feet above the pipe, and above the anticipated high groundwater depth.

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UTILITY TRENCH DETAIL

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FIG. 6

GRADATION AND DURABILITY REQUIREMENTS

After: Alaska Department of Transportation & Public Facilities
Standard Specifications for Highway Construction (2020 Edition)

E-1 Surface Course

U.S. STANDARD SIEVE SIZE		PERCENT PASSING BY
English	Metric	WEIGHT
1 in.	25 mm	100
3/4 in.	19 mm	70 - 100
3/8 in.	9.5 mm	50 - 85
No. 4	4.75 mm	35 - 65
No. 8	2.36 mm	20 - 50
No. 50	0.300 mm	15 - 30
No. 200	0.075 mm	8 - 15

D-1 Base Course

U.S. STANDARD SIEVE SIZE		PERCENT PASSING BY
English	Metric	WEIGHT
1 in.	25 mm	100
3/4 in.	19 mm	70 - 100
3/8 in.	9.5 mm	50 - 80
No. 4	4.75 mm	35 - 65
No. 8	2.36 mm	20 - 50
No. 50	0.300 mm	6 - 30
No. 200	0.075 mm	0 - 6

Selected Material Type A

U.S. STANDARD SIEVE SIZE		PERCENT PASSING BY
English	Metric	WEIGHT
No. 4	4.75 mm	20 - 60
No. 200	0.075 mm	6 Max. on minus 3-in. portion

Aggregate containing no muck, frozen material, roots, sod or other deleterious matter and with a plastic index not greater than 6 as tested by WAQTC FOP for AASHTO T 89/T 90. Meet the gradation as tested by WAQTC FOP for AASHTO T 27/T11.

Bedding

U.S. STANDARD SIEVE SIZE		PERCENT PASSING BY
English	Metric	WEIGHT
3-inch	75 mm	100
No. 4	4.75 mm	20 - 60
No. 200	0.075 mm	6 Max. on minus 3-in. portion

Aggregate containing no muck, frozen material, roots, sod or other deleterious matter and with a plastic index not greater than 6 as tested by WAQTC FOP for AASHTO T 89/T 90. Meet the gradation as tested by WAQTC FOP for AASHTO T 27/T11.

Porous Backfill Material

U.S. STANDARD SIEVE SIZE		PERCENT PASSING BY
English	Metric	WEIGHT
3 in.	75 mm	100
1 in.	25 mm	0 - 10
No. 200	0.075 mm	0 - 5

Gravel consisting of crushed or naturally occurring granular material containing not more than 1% clay lumps or other readily decomposed material (AASHTO T 112).

Coarse Aggregate Durability

Retained on #4 Sieve

Test Type	Percent Loss
L.A. Abrasion	45 - 50 max. *
Degradation	45 min.
Sulfate Soundness	9 max.

* Asphalt and Surface Course = 45% max
Base Course = 50% max

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GRADATION AND DURABILITY REQUIREMENTS

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FIG. 7

Appendix A

Boring Logs and Laboratory Test Results

CONTENTS

- Soil Description and Log Key
- Frost Classification
- Boring Logs B-01 through B-29
- Grain Size Analysis (20 sheets)

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 Clay³	Sand or Gravel⁴
Modifying (Secondary) Precedes major constituent	30% or more coarse-grained: Sandy or Gravelly⁴	More than 12% fine-grained: Silty or Clayey³
Minor Follows major constituent	15% to 30% coarse-grained: with Sand or with Gravel⁴ 30% or more total coarse-grained and lesser coarse-grained constituent is 15% or more: with Sand or with Gravel⁵	5% to 12% fine-grained: with Silt or with Clay³ 15% or more of a second coarse-grained constituent: with Sand or with Gravel⁵

¹All percentages are by weight of total specimen passing a 3-inch sieve.

²The order of terms is: *Modifying Major with Minor*.

³Determined based on behavior.

⁴Determined based on which constituent comprises a larger percentage.

⁵Whichever 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%

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

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SOIL DESCRIPTION AND LOG KEY

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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.

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**SOIL DESCRIPTION
AND LOG KEY**

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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 TERMS¹

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

PLASTICITY²

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 TERMS¹

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.

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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 TERMS¹

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.

Yakutat Airport Water Line Extension
Yakutat, Alaska

**SOIL DESCRIPTION
AND LOG KEY**

April 2025

112494-003



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FIG. A-1
Sheet 3 of 3

FROST CLASSIFICATION
(after Municipality of Anchorage, 2007)

GROUP		0.02 Mil.	P-200*	USC SYSTEM (based on P-200 results)
NFS	Sandy Soils	0 to 3	0 to 6	SW, SP, SW-SM, SP-SM
	Gravelly Soils	0 to 3	0 to 6	GW, GP, GW-GM, GP-GM
F1	Gravelly Soils	3 to 10	6 to 13	GM, GW-GM, GP-GM
F2	Sandy Soils	3 to 15	6 to 19	SP-SM, SW-SM, SM
	Gravelly Soils	10 to 20	13 to 25	GM
F3	Sands, except very fine silty sands**	Over 15	Over 19	SM, SC
	Gravelly Soils	Over 20	Over 25	GM, GC
	Clays, PI>12			CL, CH
F4	All Silts			ML, MH
	Very fine silty sands**	Over 15	Over 19	SM, SC
	Clays, PI<12			CL, CL-ML
	Varved clays and other finned grained, banded sediments			CL and ML CL, ML, and SM; SL, SH, and ML; CL, CH, ML, and SM

PI = Plasticity Index

P-200 = Percent passing the number 200 sieve

0.02 Mil. = Percent material below 0.02 millimeter grain size

*Approximate P-200 value equivalent for frost classification.
Value range based on typical, well-graded soil curves.

** Very fine sand : greater than 50% of sand
fraction passing the number 100 sieve

Yakutat Airport Water Line Extension
Yakutat, Alaska

FROST CLASSIFICATION LEGEND

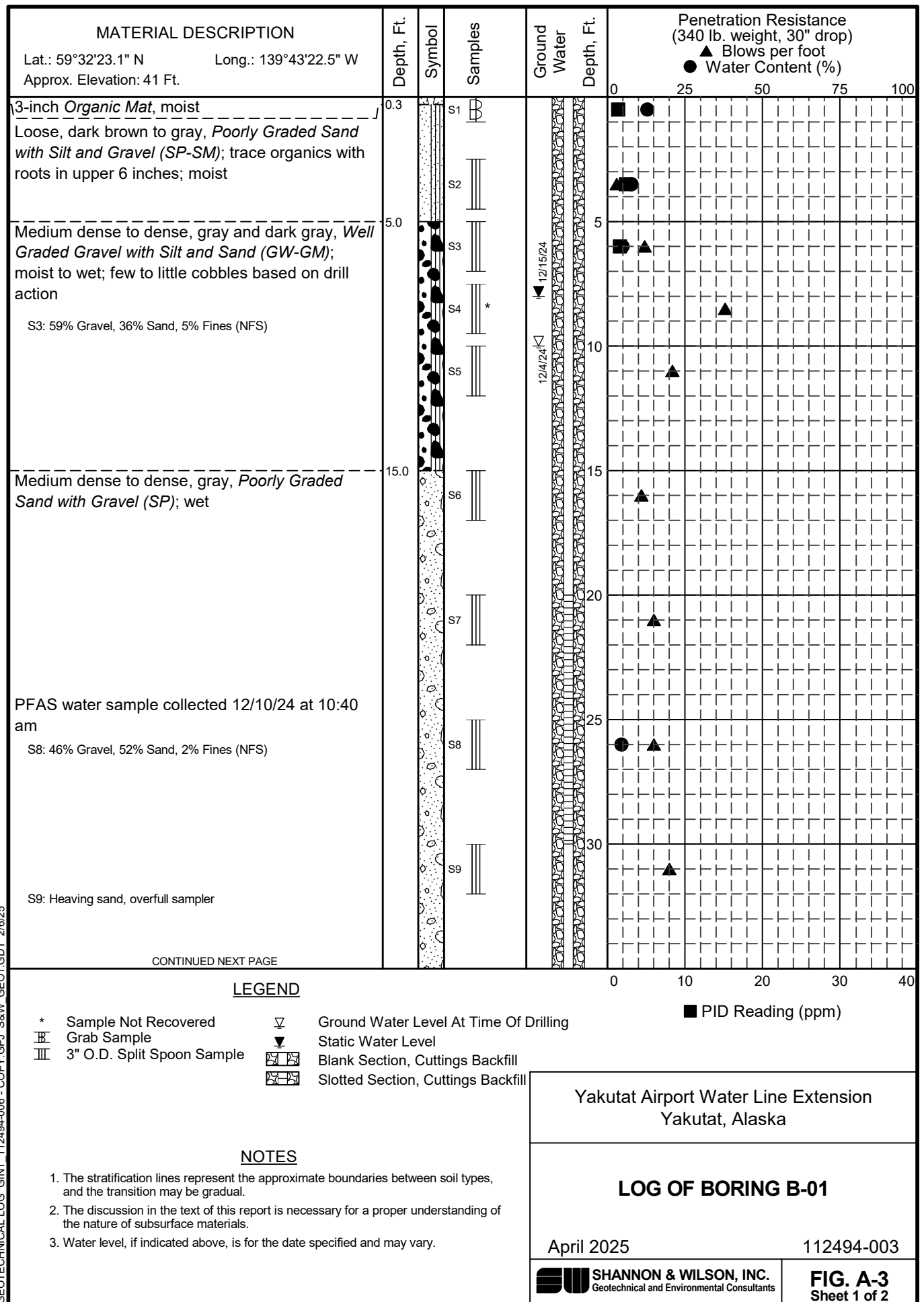
April 2025

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FIG. A-2



GEOTECHNICAL LOG GINT 112494-006 - COPY.GPJ S&W GEO1.GDT 2/6/25

MATERIAL DESCRIPTION	Depth, Ft.	Symbol	Samples	Ground Water	Depth, Ft.
Lat.: 59°32'23.1" N Long.: 139°43'22.5" W Approx. Elevation: 41 Ft.					
Medium dense to dense, gray, Poorly Graded Sand with Gravel (SP); wet S10: 35% Gravel, 61% Sand, 4% Fines (NFS)					
Dense, gray, Poorly Graded Sand with Silt and Gravel (SP-SM); wet S13: Heaving sand, overfull sampler S: 29% Gravel, 61% Sand, 10% Fines (F2)	49.0				
Bottom of Boring Boring Completed 12/4/24	60.0				
Boring backfilled with bentonite from approx. 10 feet bgs to ground surface					

Penetration Resistance (340 lb. weight, 30" drop)
▲ Blows per foot
● Water Content (%)

Depth (ft)	Blows per foot	Water Content (%)
0	~15	~85
10	~10	-
20	~18	-
30	~12	~75
40	~22	-
50	~15	-
60	-	-

LEGEND

* Sample Not Recovered	▽ Ground Water Level At Time Of Drilling
▬ Grab Sample	▼ Static Water Level
▮ 3" O.D. Split Spoon Sample	Blank Section, Cuttings Backfill
	Slotted Section, Cuttings Backfill

NOTES

- The stratification lines represent the approximate boundaries between soil types, and the transition may be gradual.
- The discussion in the text of this report is necessary for a proper understanding of the nature of subsurface materials.
- Water level, if indicated above, is for the date specified and may vary.

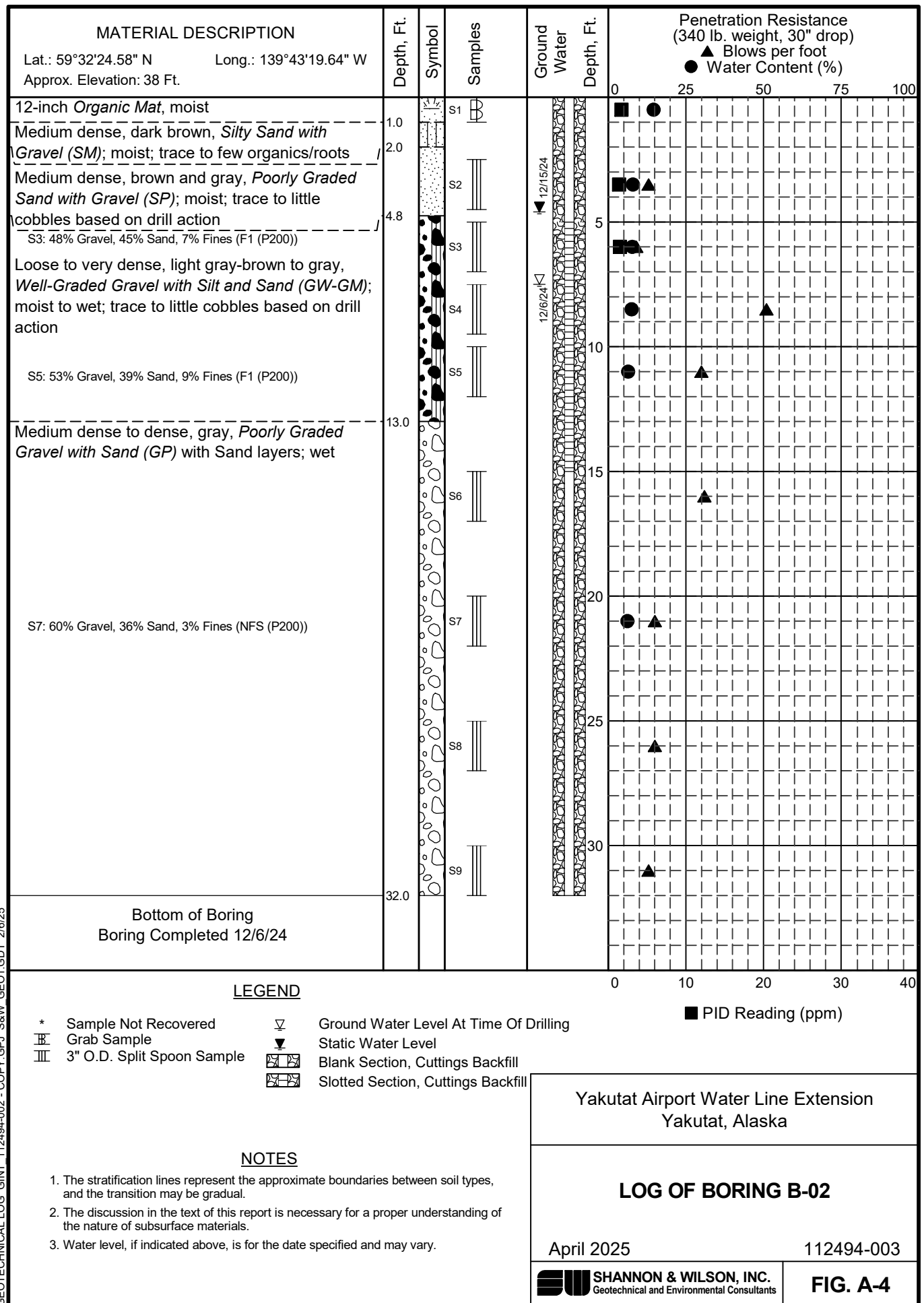
Yakutat Airport Water Line Extension
Yakutat, Alaska

LOG OF BORING B-01

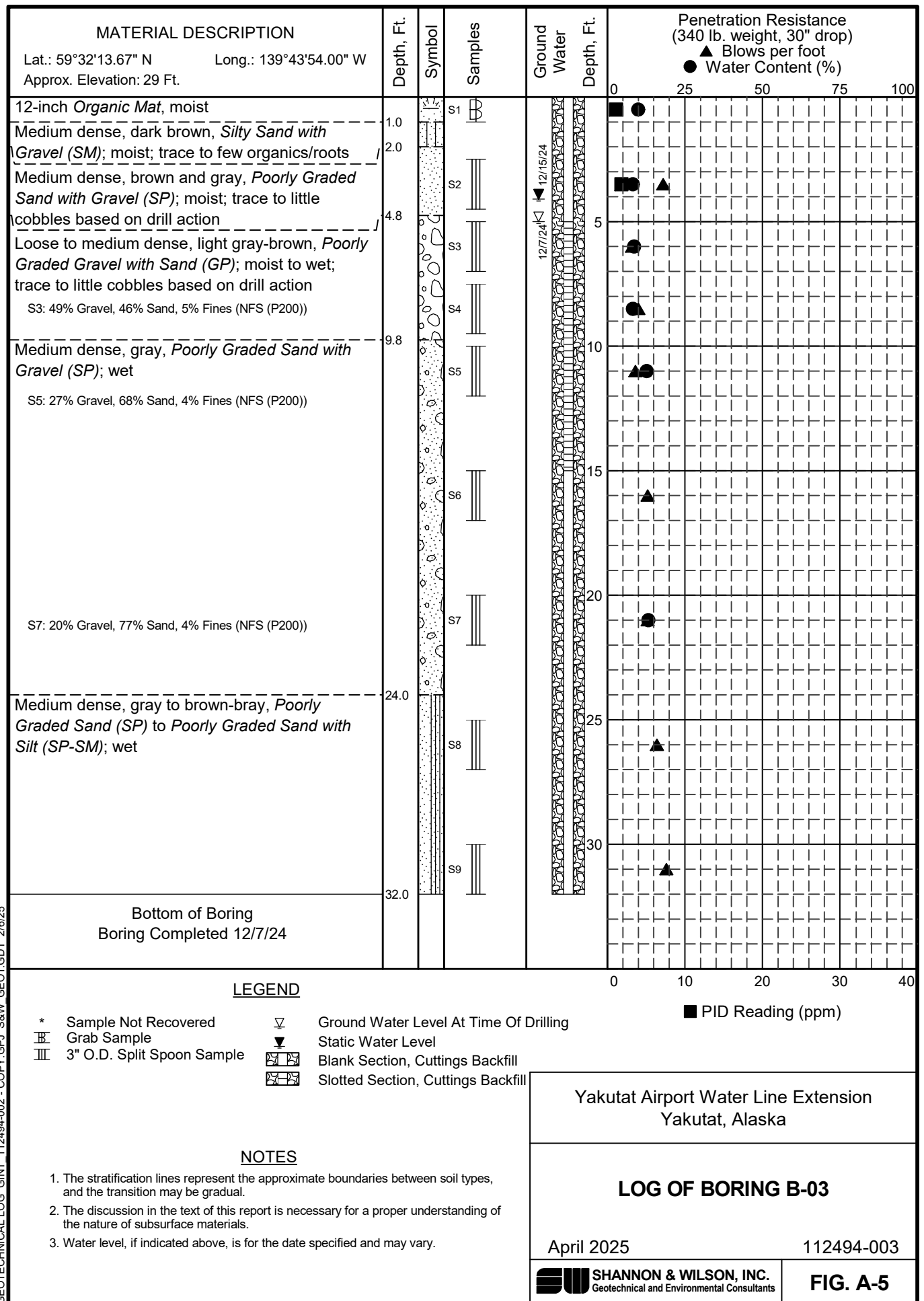
April 2025112494-003

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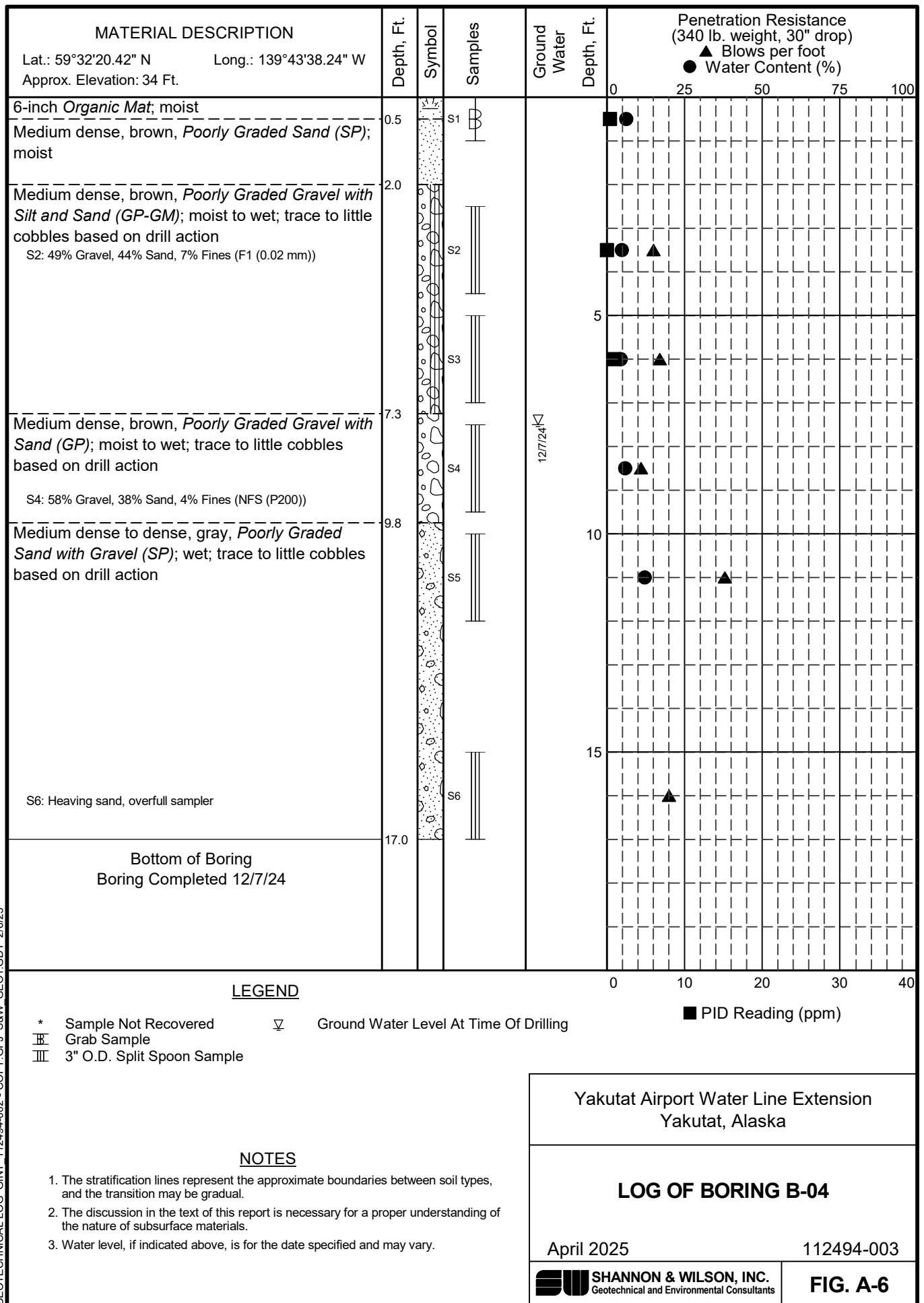
FIG. A-3
Sheet 2 of 2



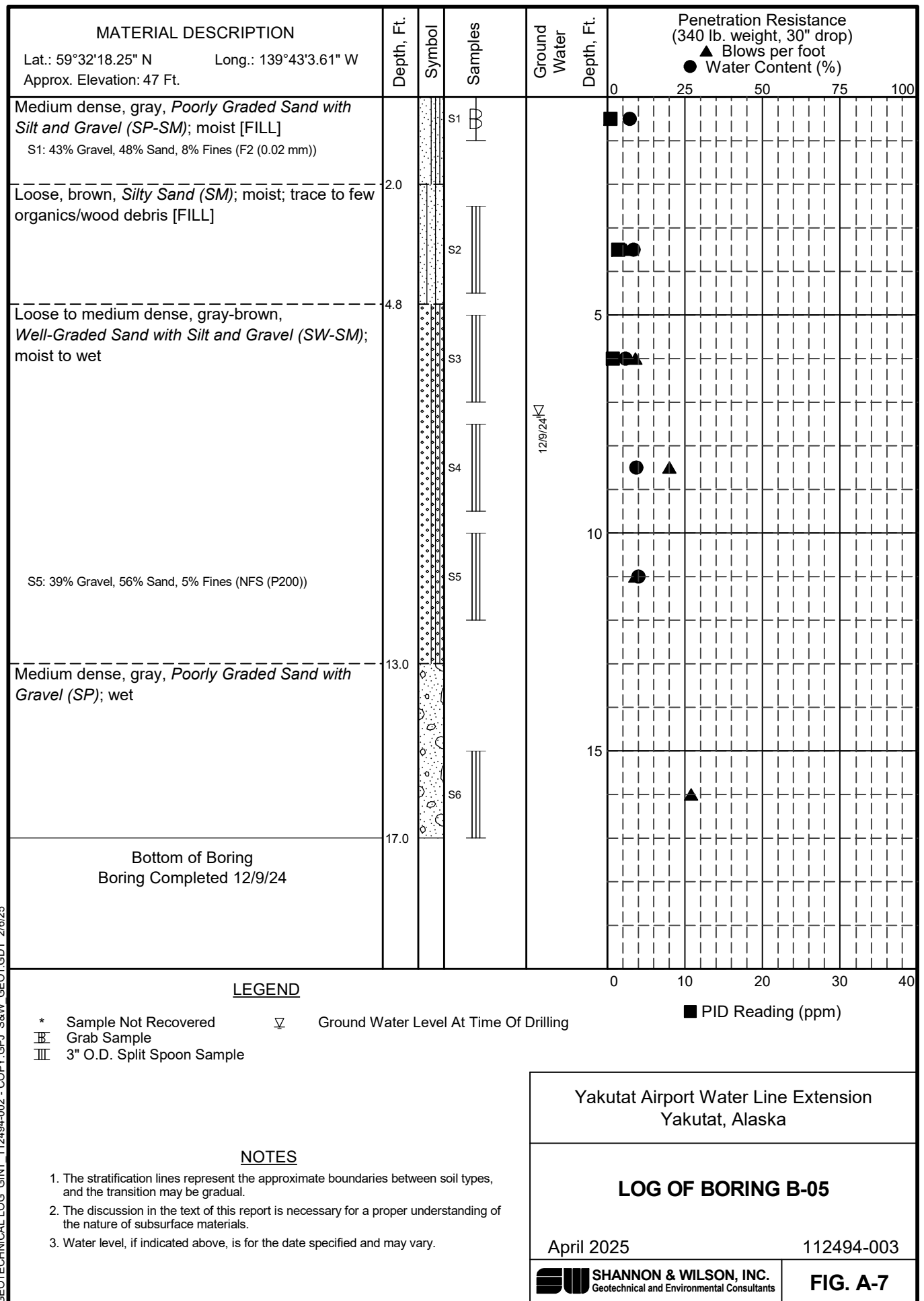
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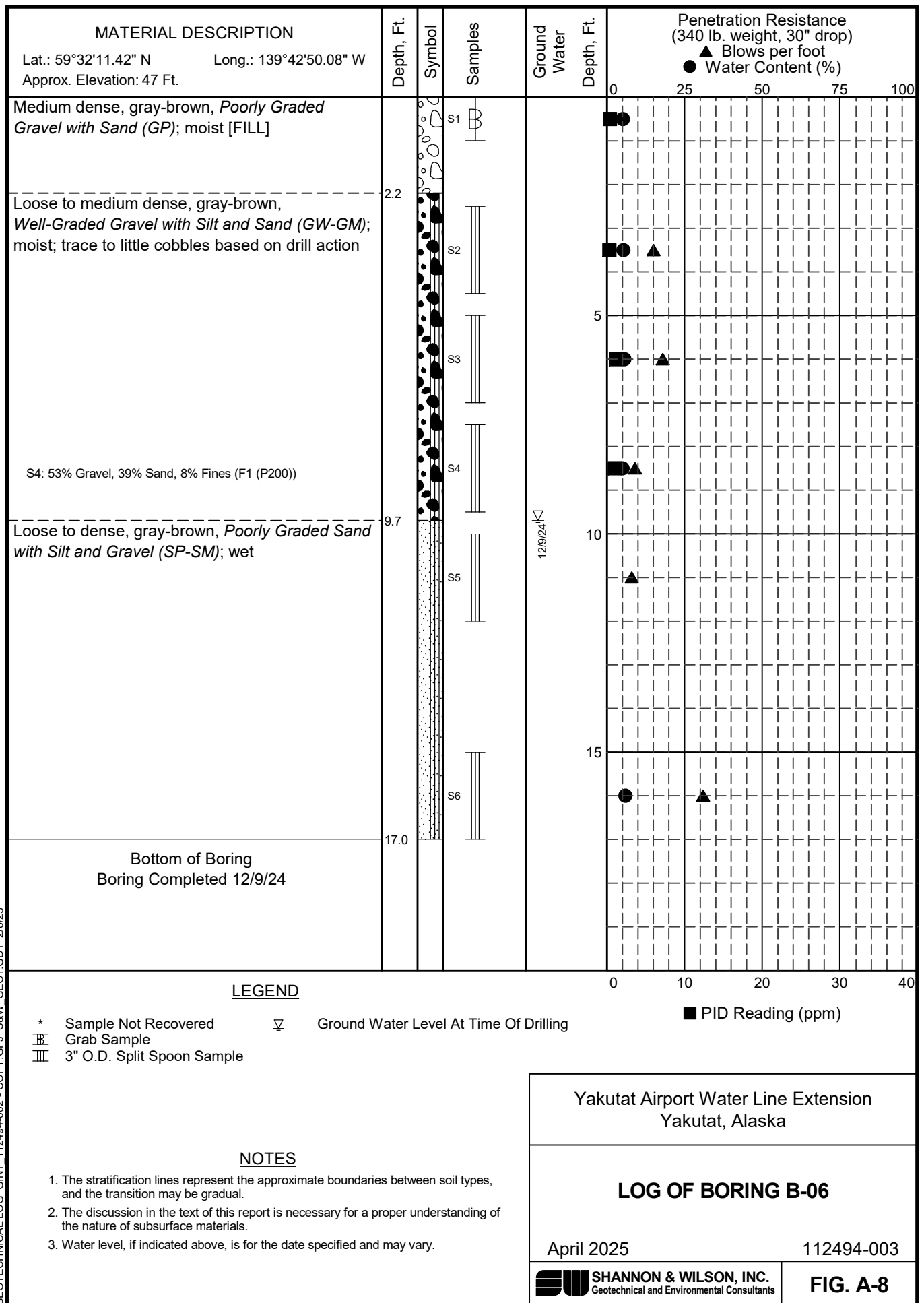
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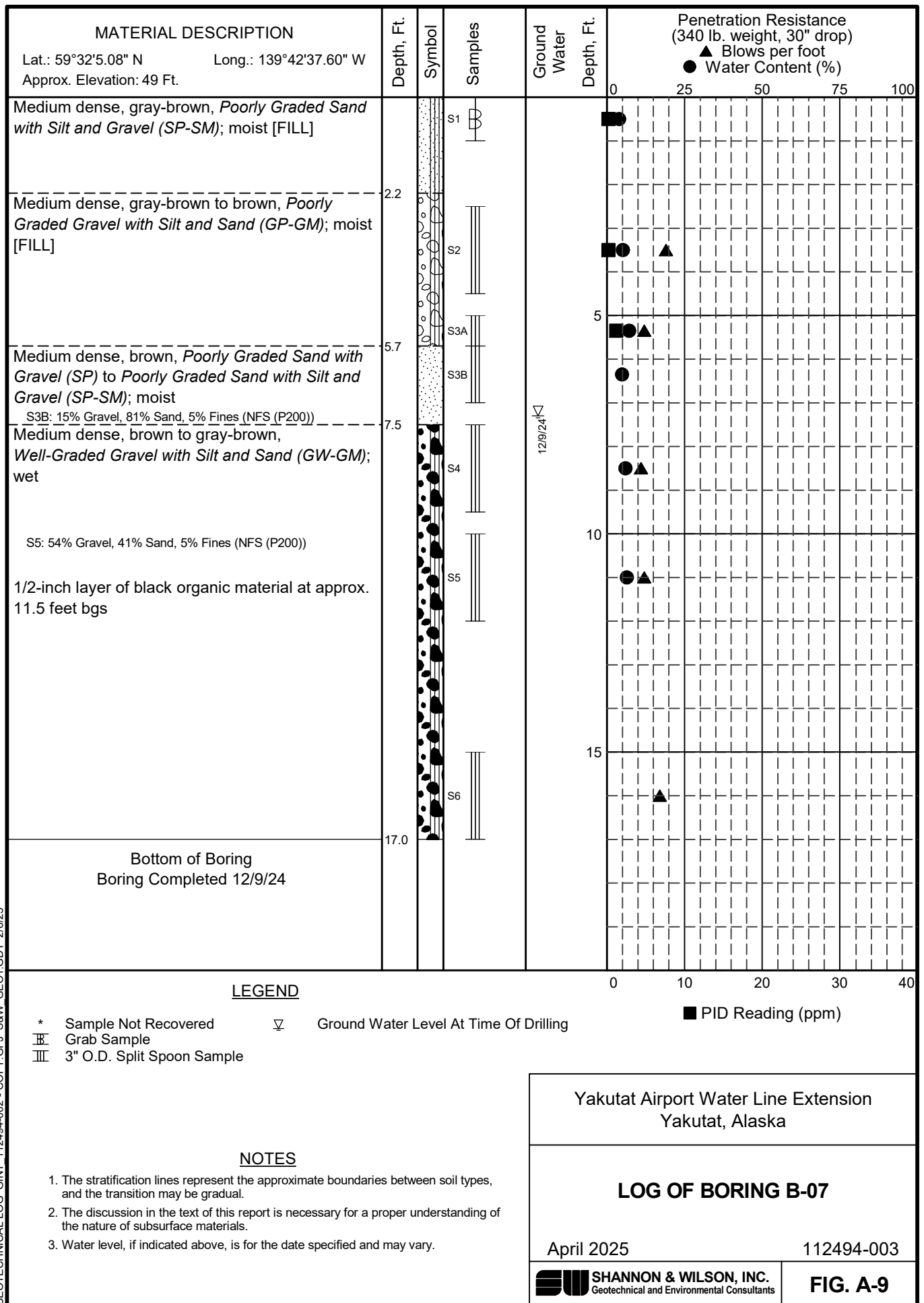
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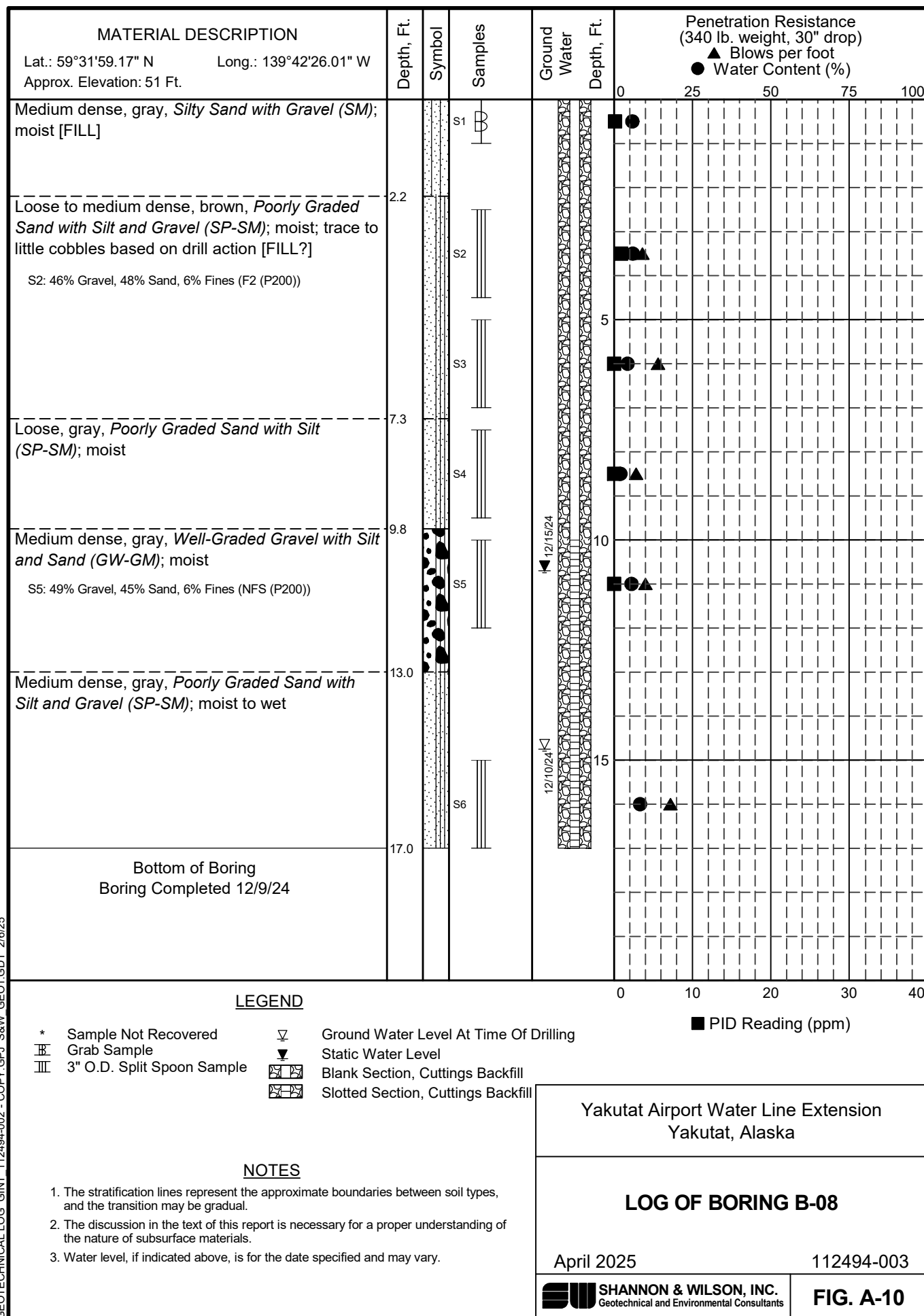
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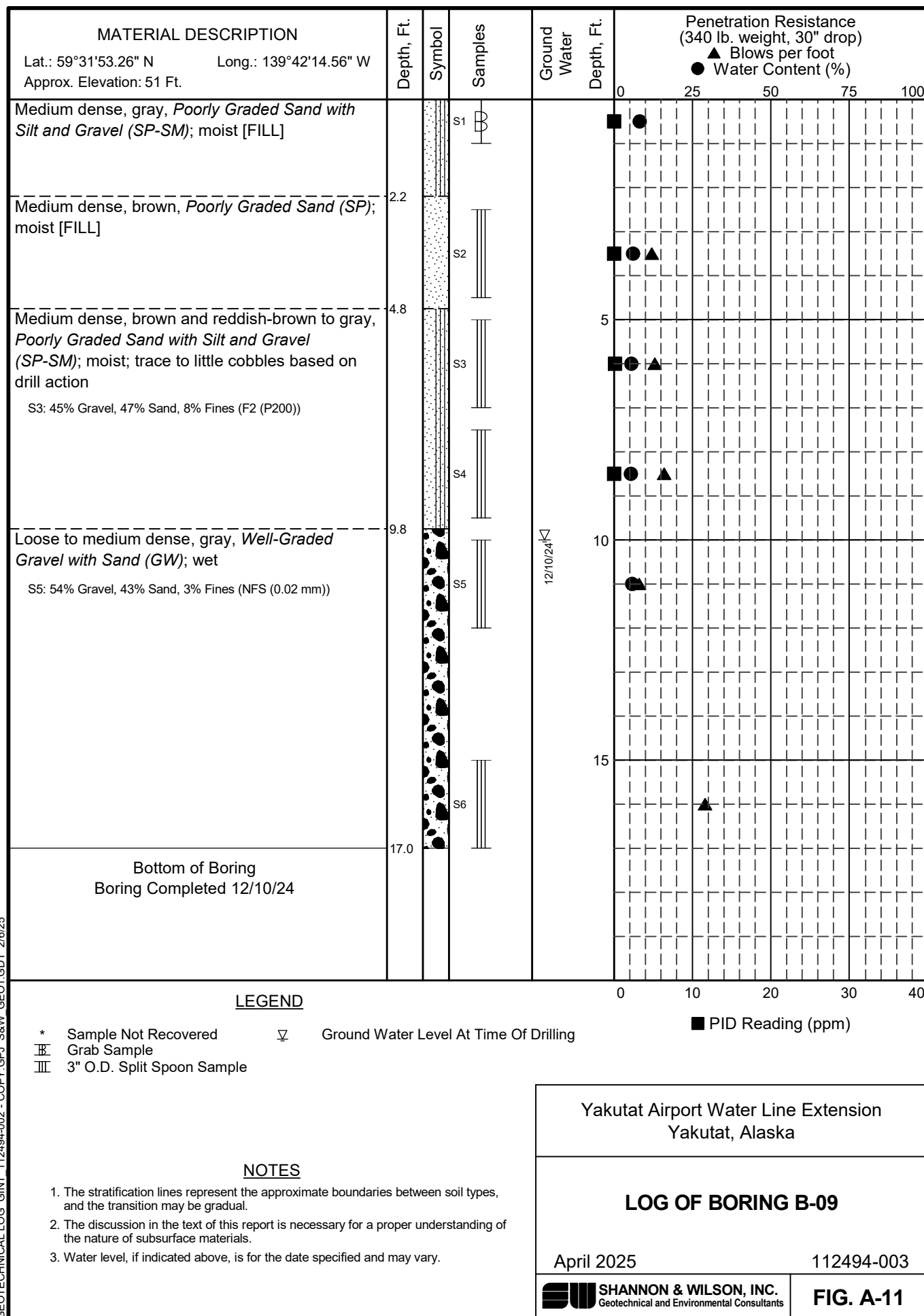
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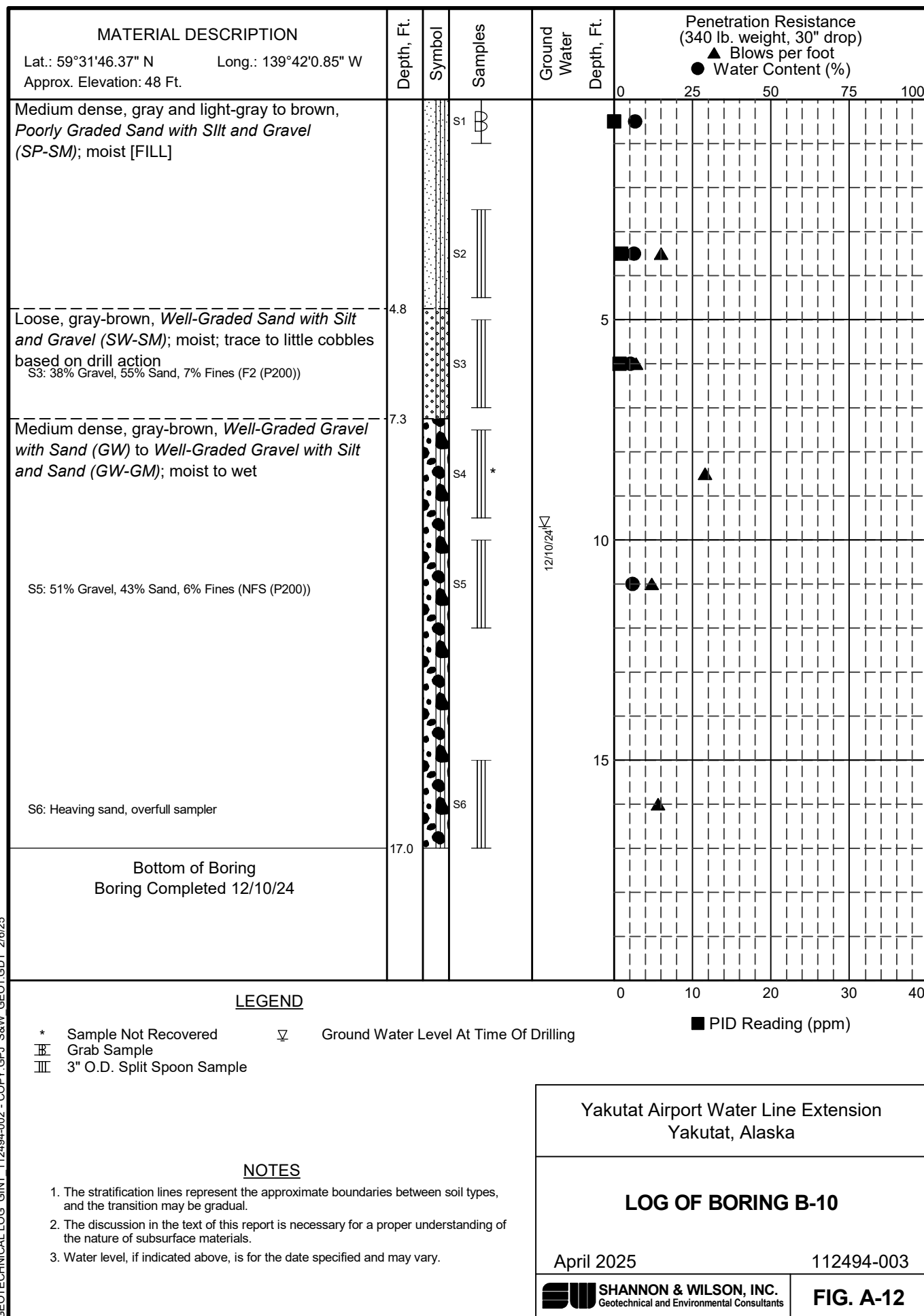
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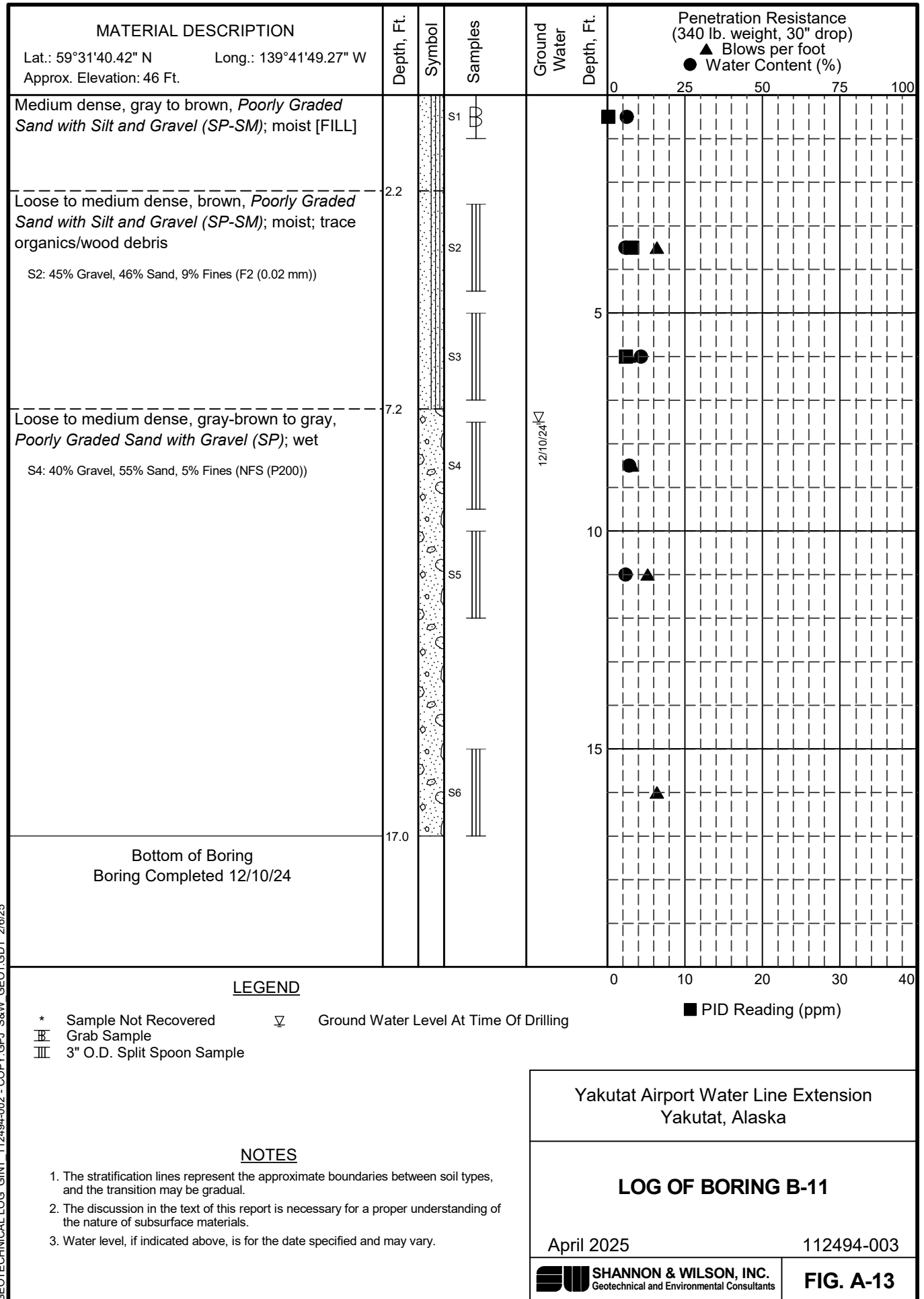


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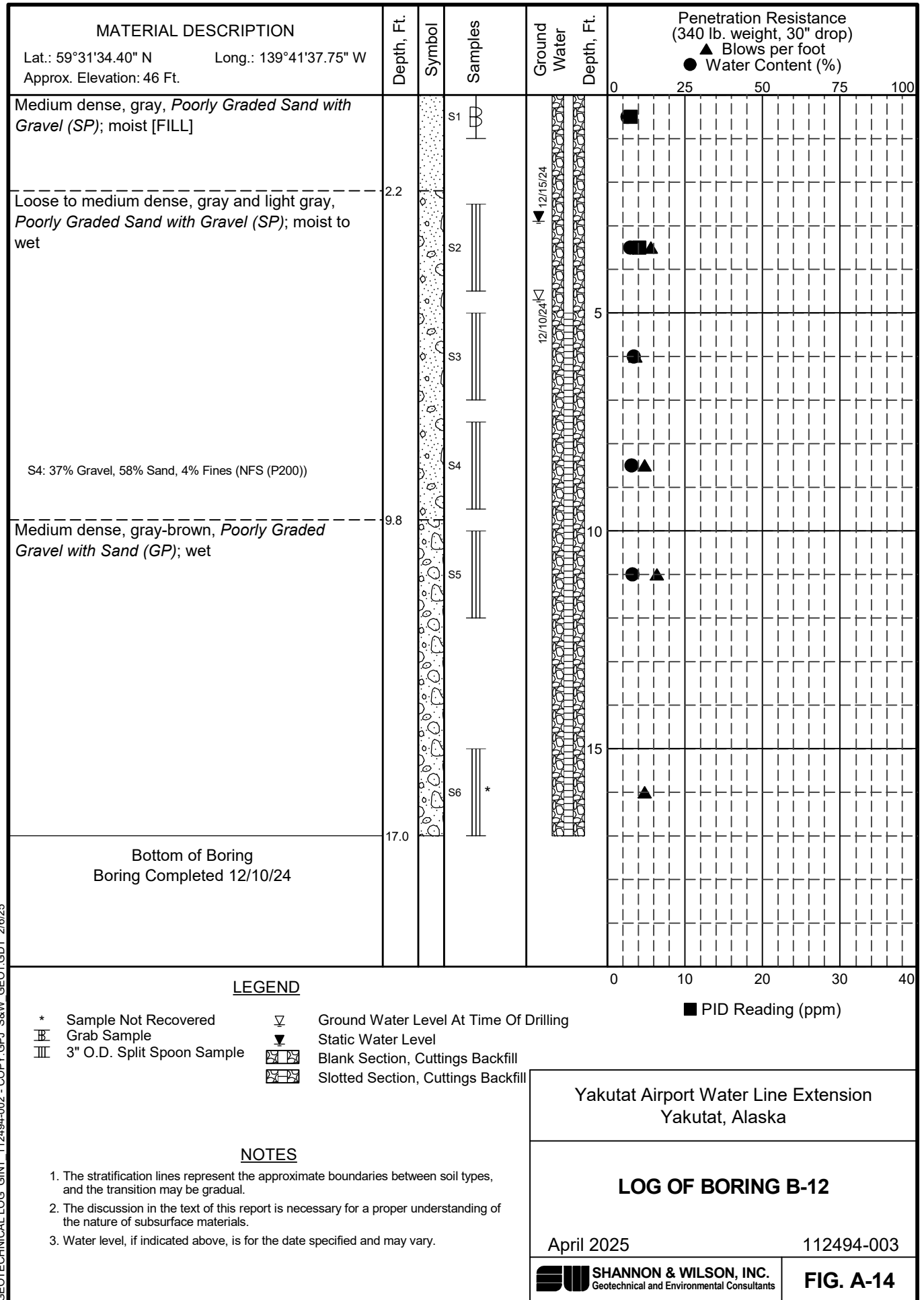


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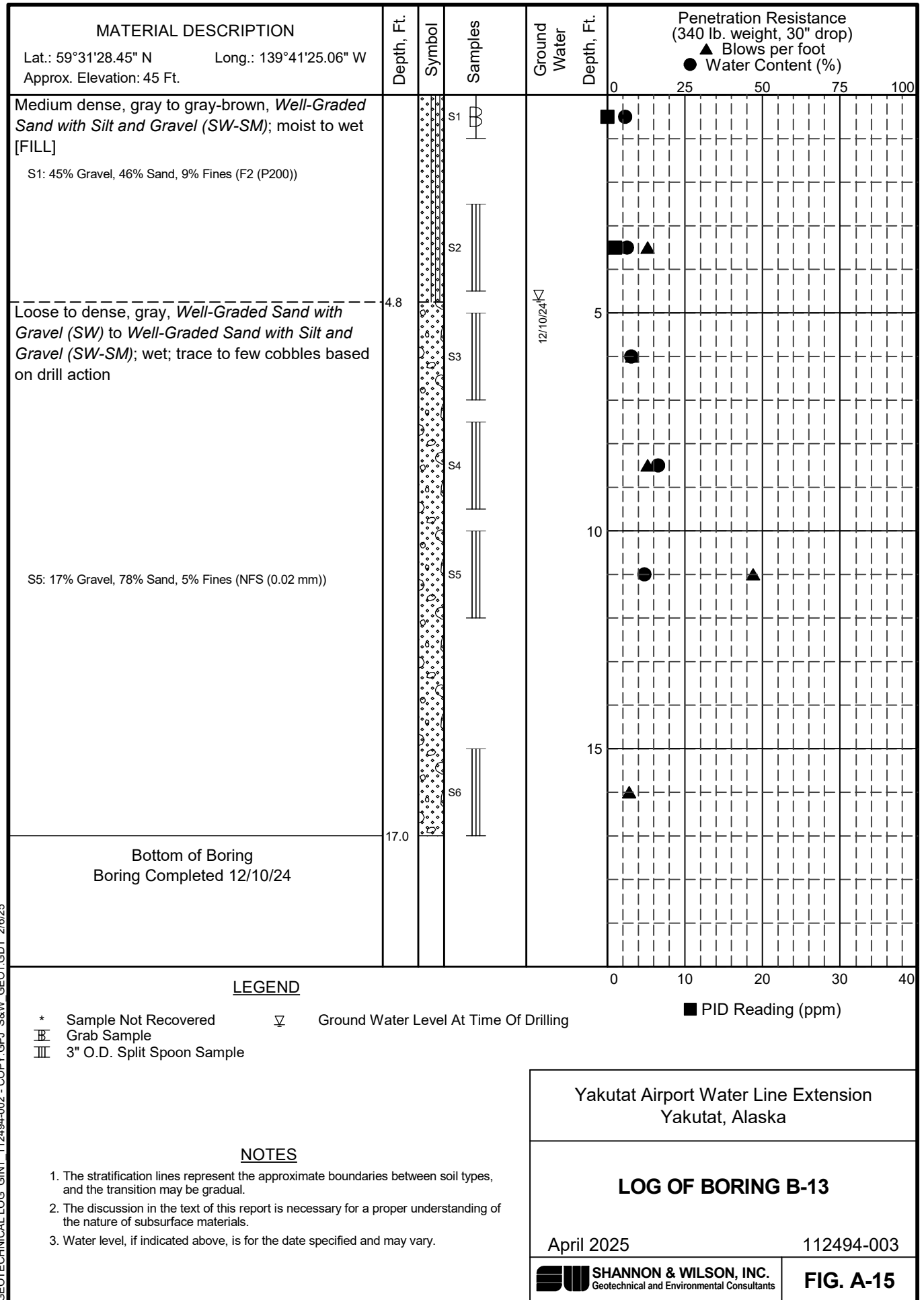




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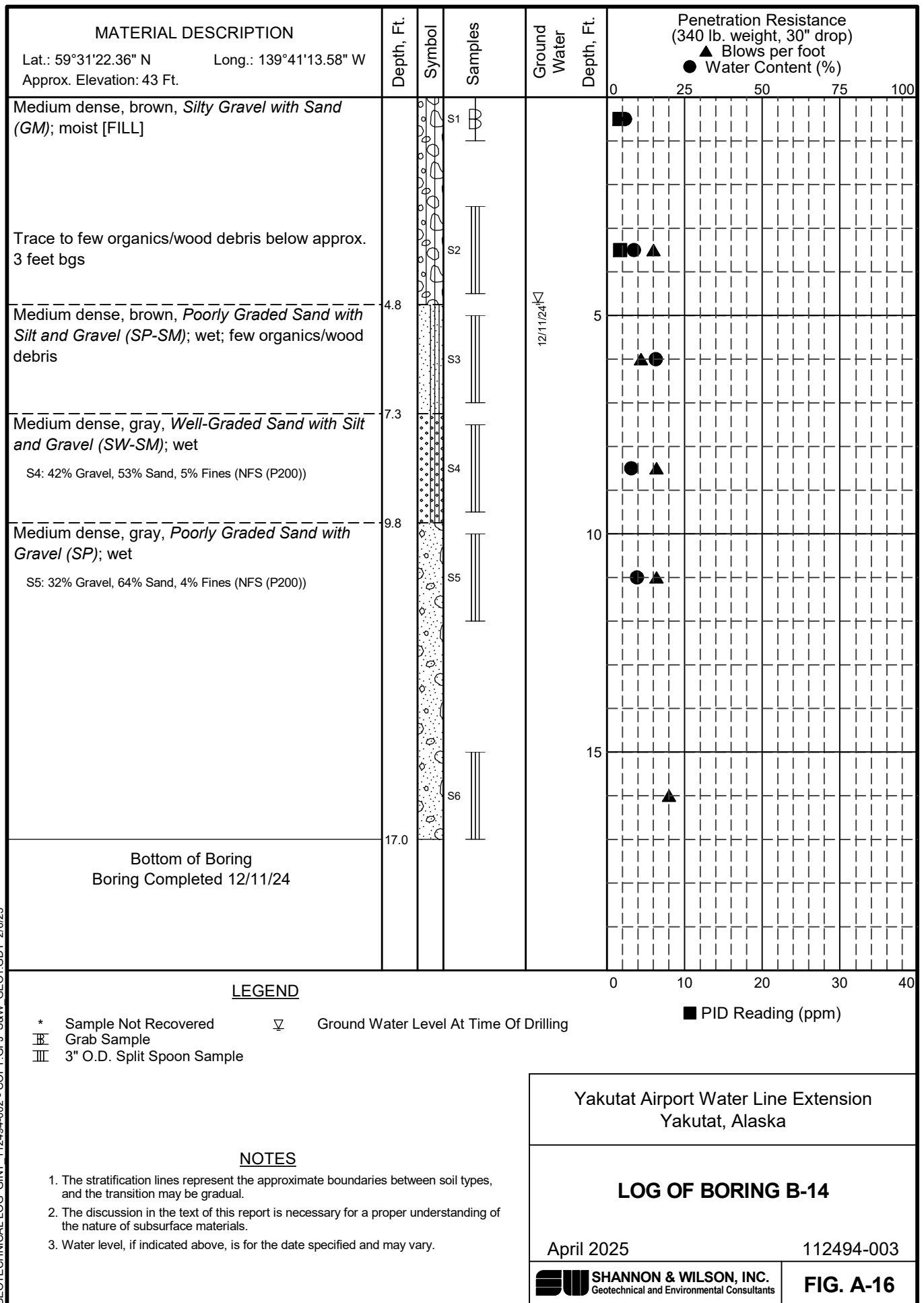


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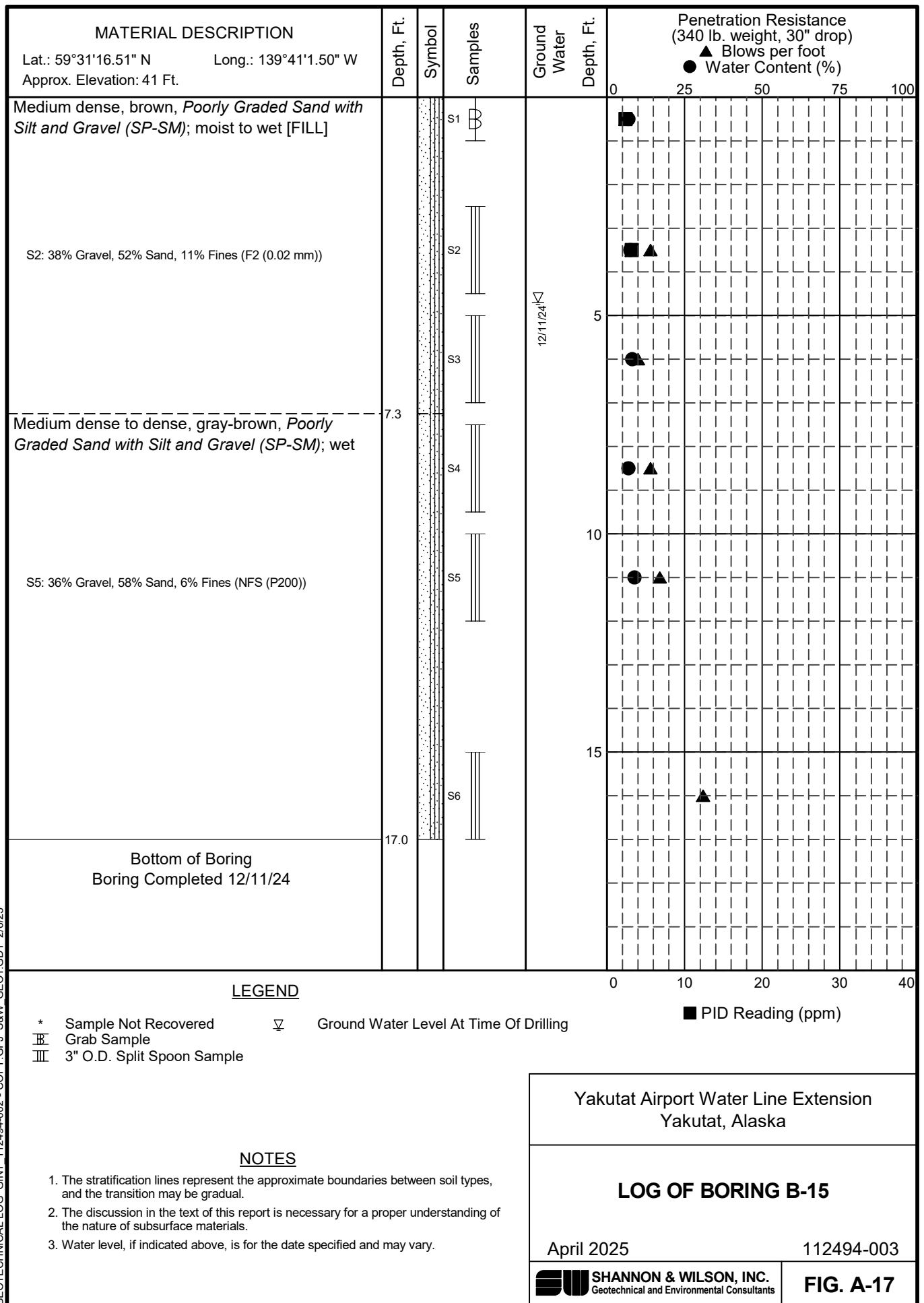


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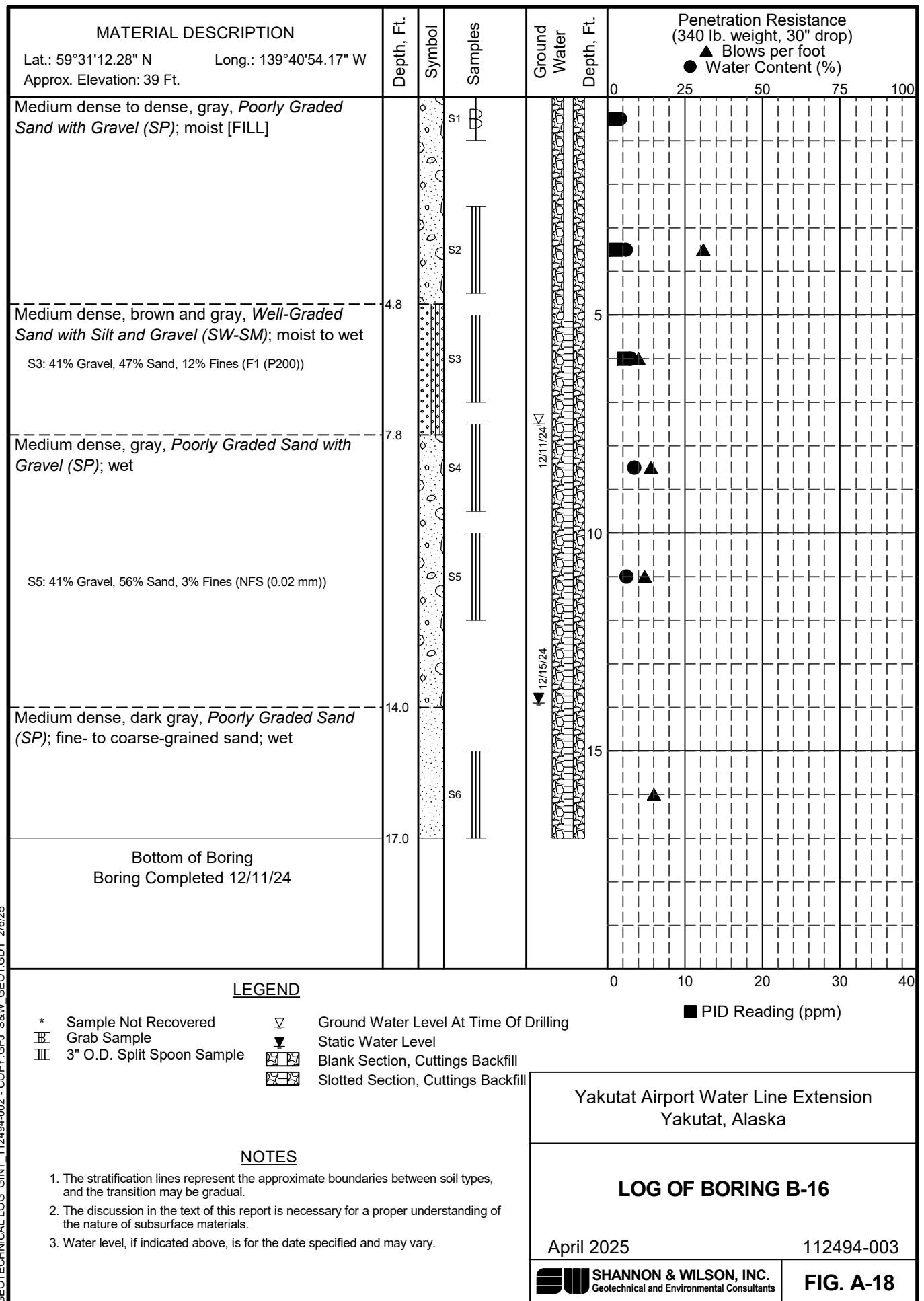
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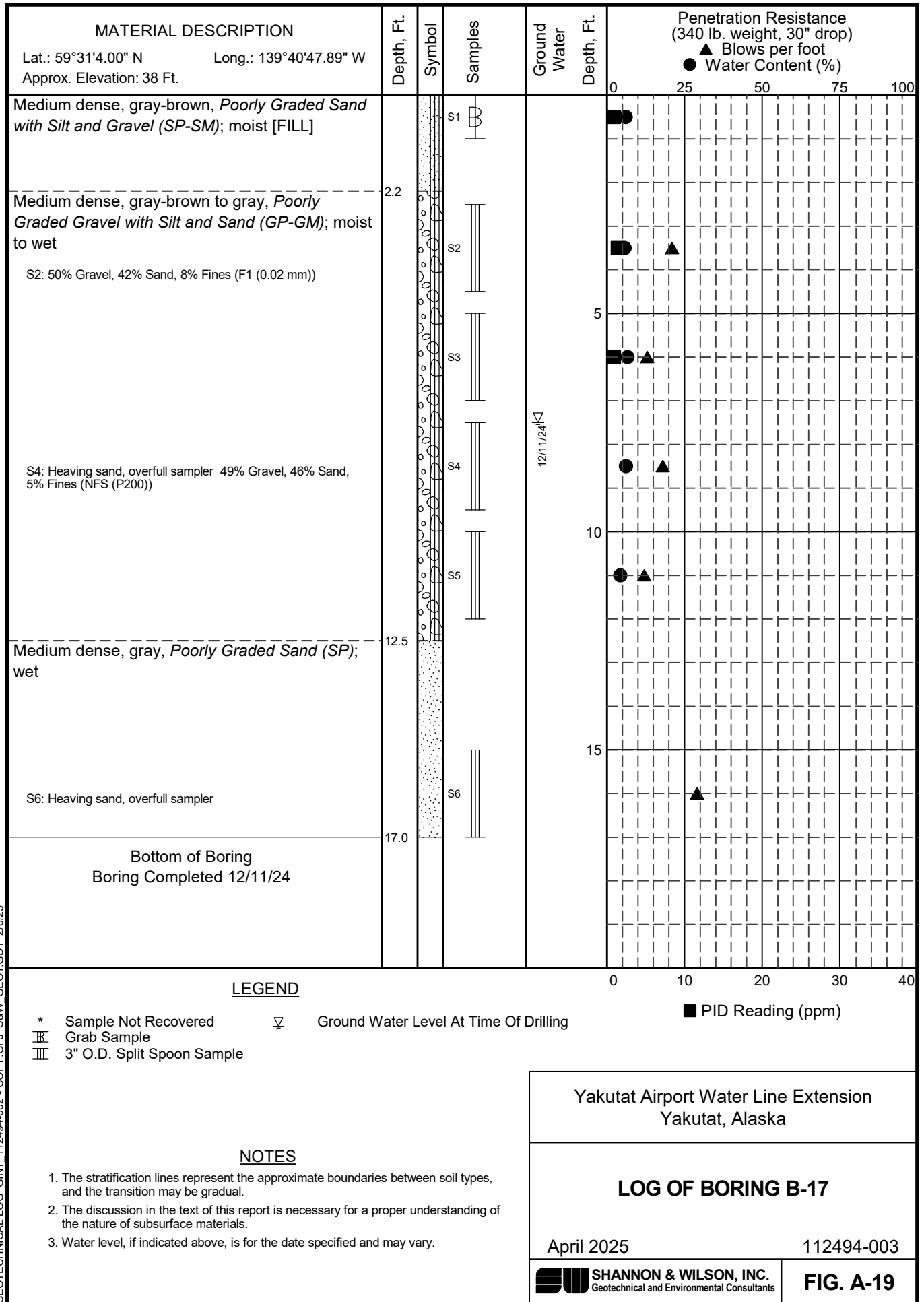
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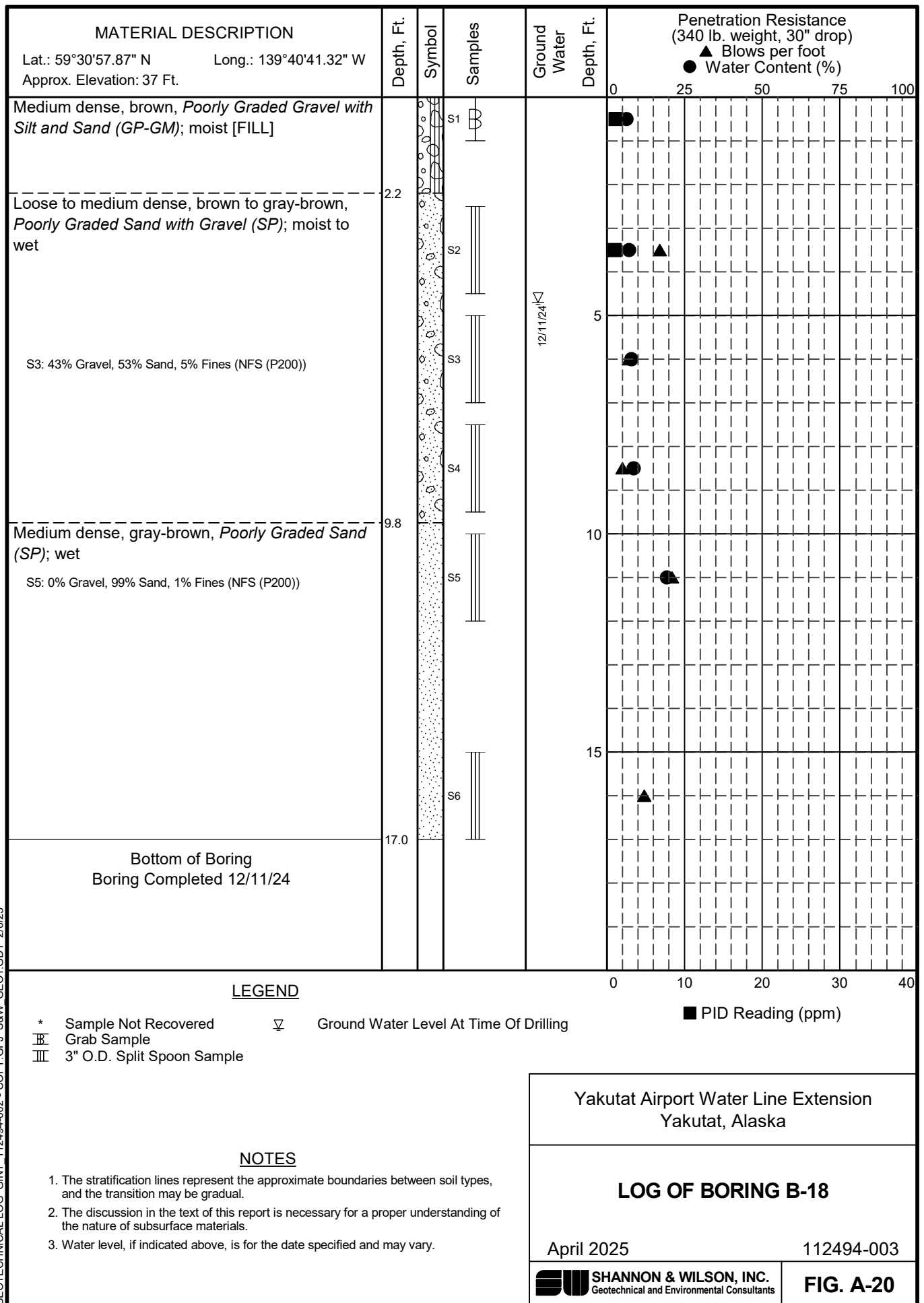
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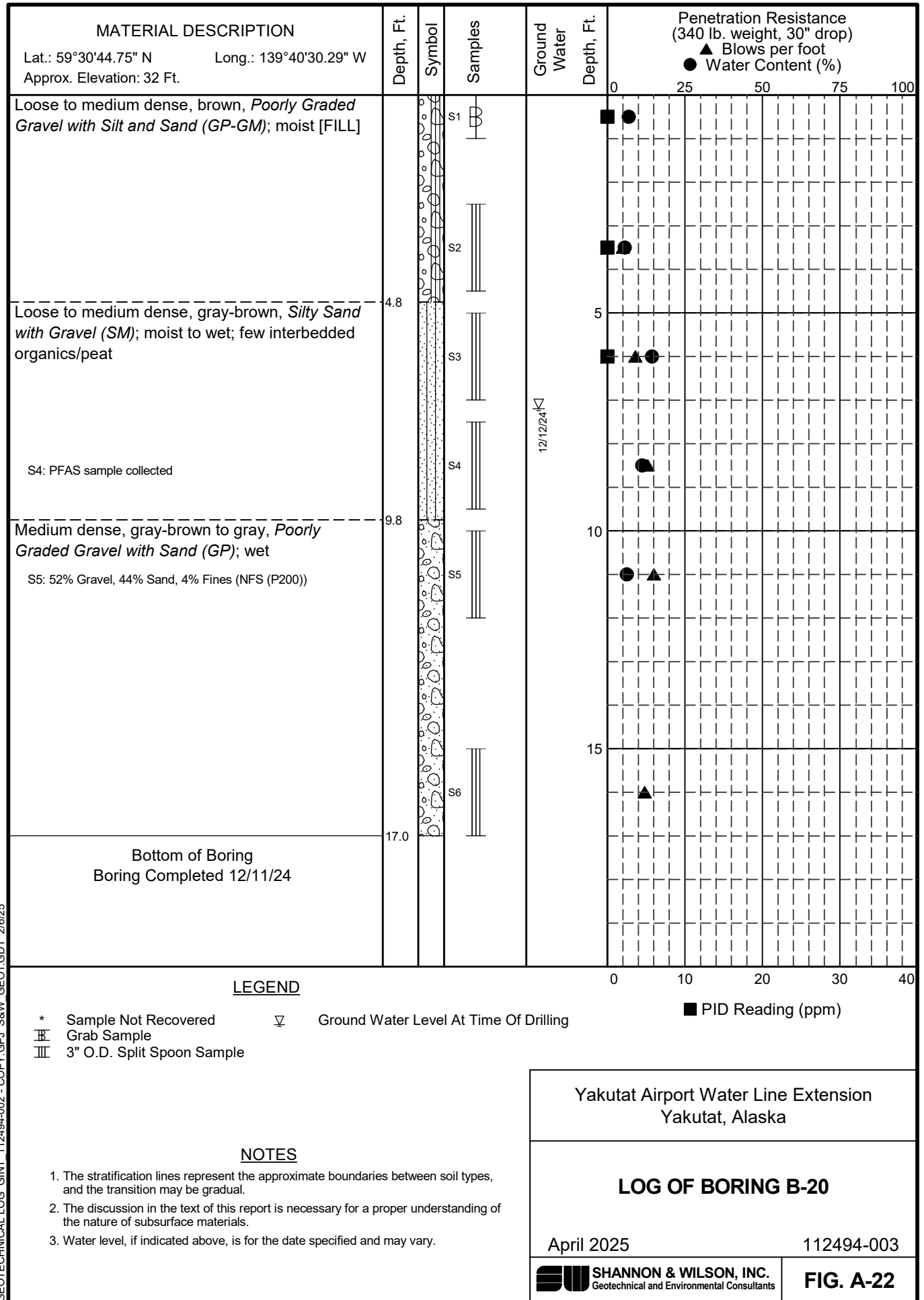
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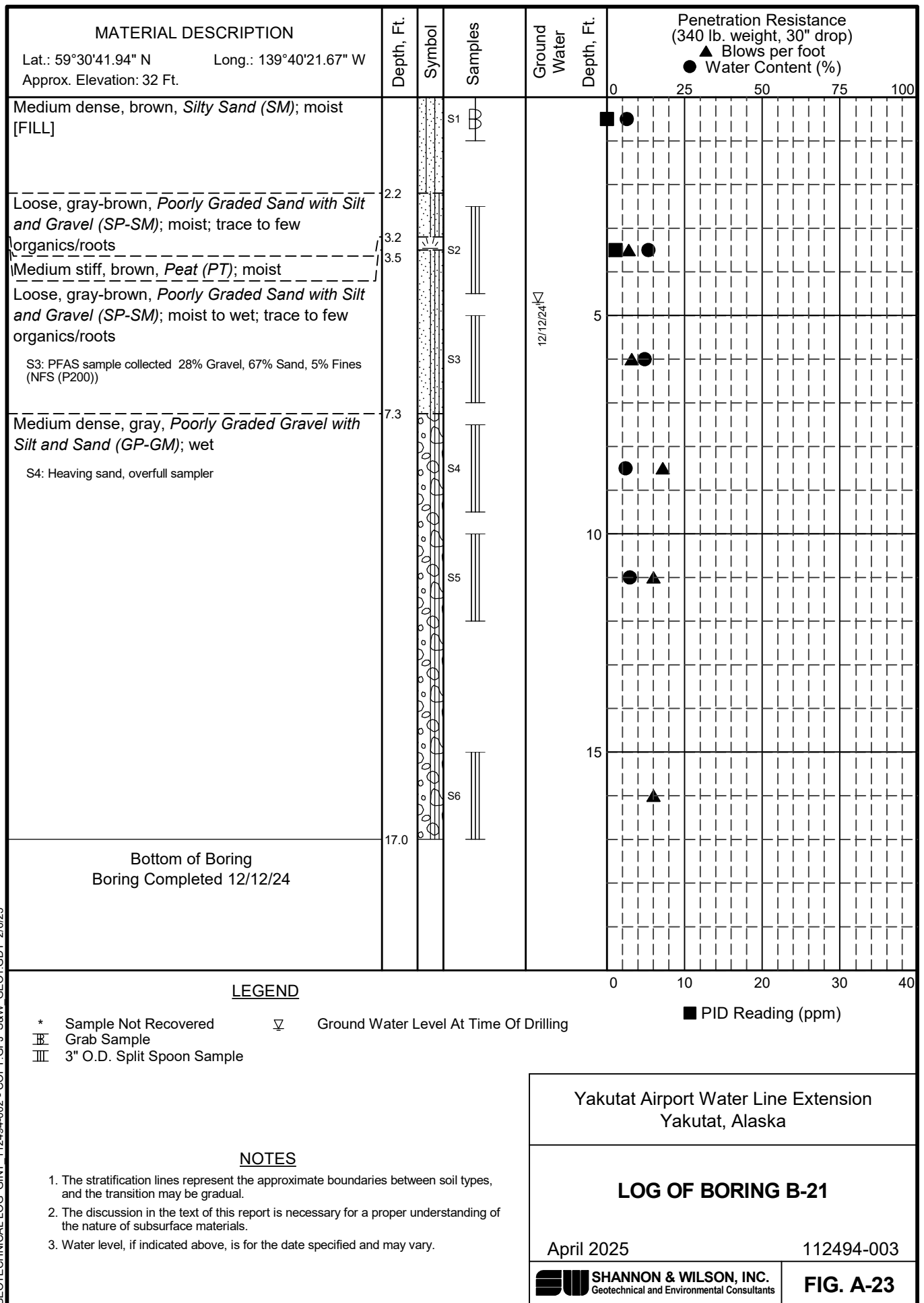


MATERIAL DESCRIPTION	Depth, Ft.	Symbol	Samples	Ground Water Depth, Ft.	Penetration Resistance (340 lb. weight, 30" drop) ▲ Blows per foot ● Water Content (%)
Lat.: 59°30'50.47" N Long.: 139°40'33.46" W Approx. Elevation: 36 Ft.					0 25 50 75 100
Medium dense, brown, Poorly Graded Sand with Silt and Gravel (SP-SM); moist [FILL] S1: 40% Gravel, 52% Sand, 8% Fines (F2 (P200))			S1		
			S2		
	4.8			12/11/24	
Medium dense, brown, Poorly Graded Sand with Gravel (SP); wet; trace to few organics/wood debris			S3		
	7.4				
Medium dense, gray-brown, Poorly Graded Sand with Silt and Gravel (SP-SM); wet S4: 28% Gravel, 66% Sand, 6% Fines (F2 (0.02 mm))			S4		
			S5		
			S6		
	17.0				
Bottom of Boring Boring Completed 12/11/24					
LEGEND					
* Sample Not Recovered	▽	Ground Water Level At Time Of Drilling			
▤ Grab Sample		■ PID Reading (ppm)			
▨ 3" O.D. Split Spoon Sample					
NOTES					
1. The stratification lines represent the approximate boundaries between soil types, and the transition may be gradual.					
2. The discussion in the text of this report is necessary for a proper understanding of the nature of subsurface materials.					
3. Water level, if indicated above, is for the date specified and may vary.					
Yakutat Airport Water Line Extension Yakutat, Alaska					
LOG OF BORING B-19					
April 2025 112494-003					
SHANNON & WILSON, INC. Geotechnical and Environmental Consultants					
FIG. A-21					

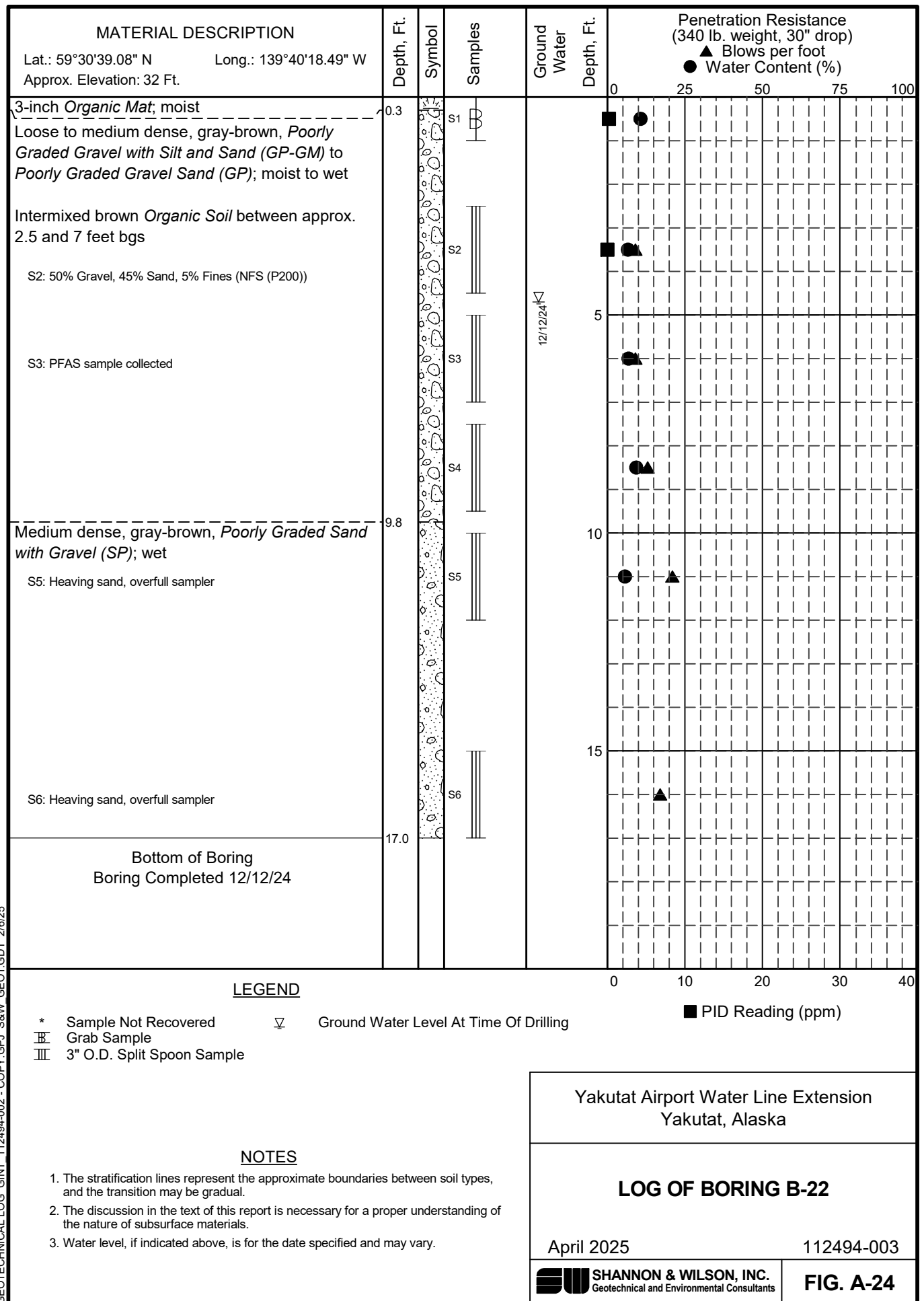


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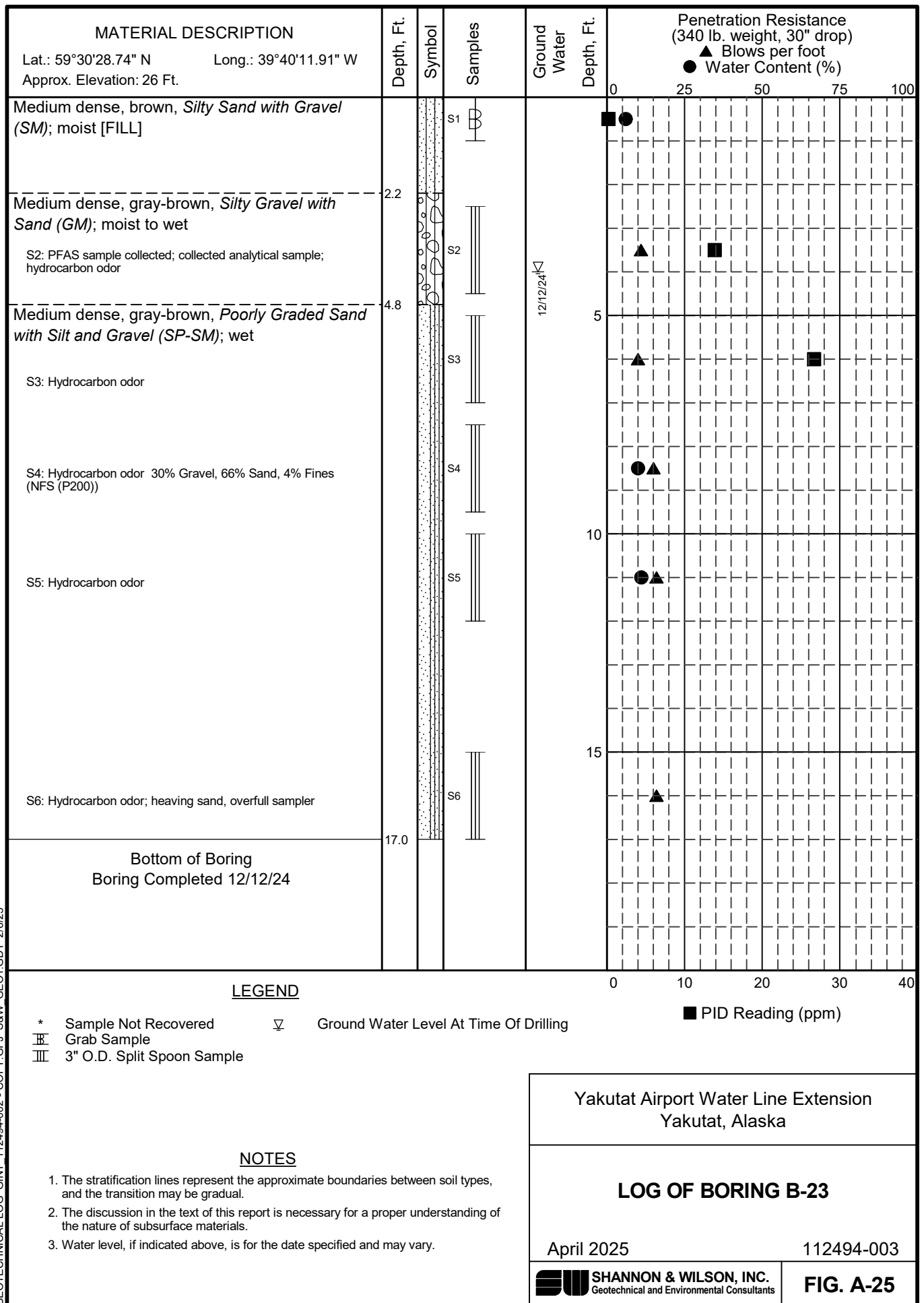
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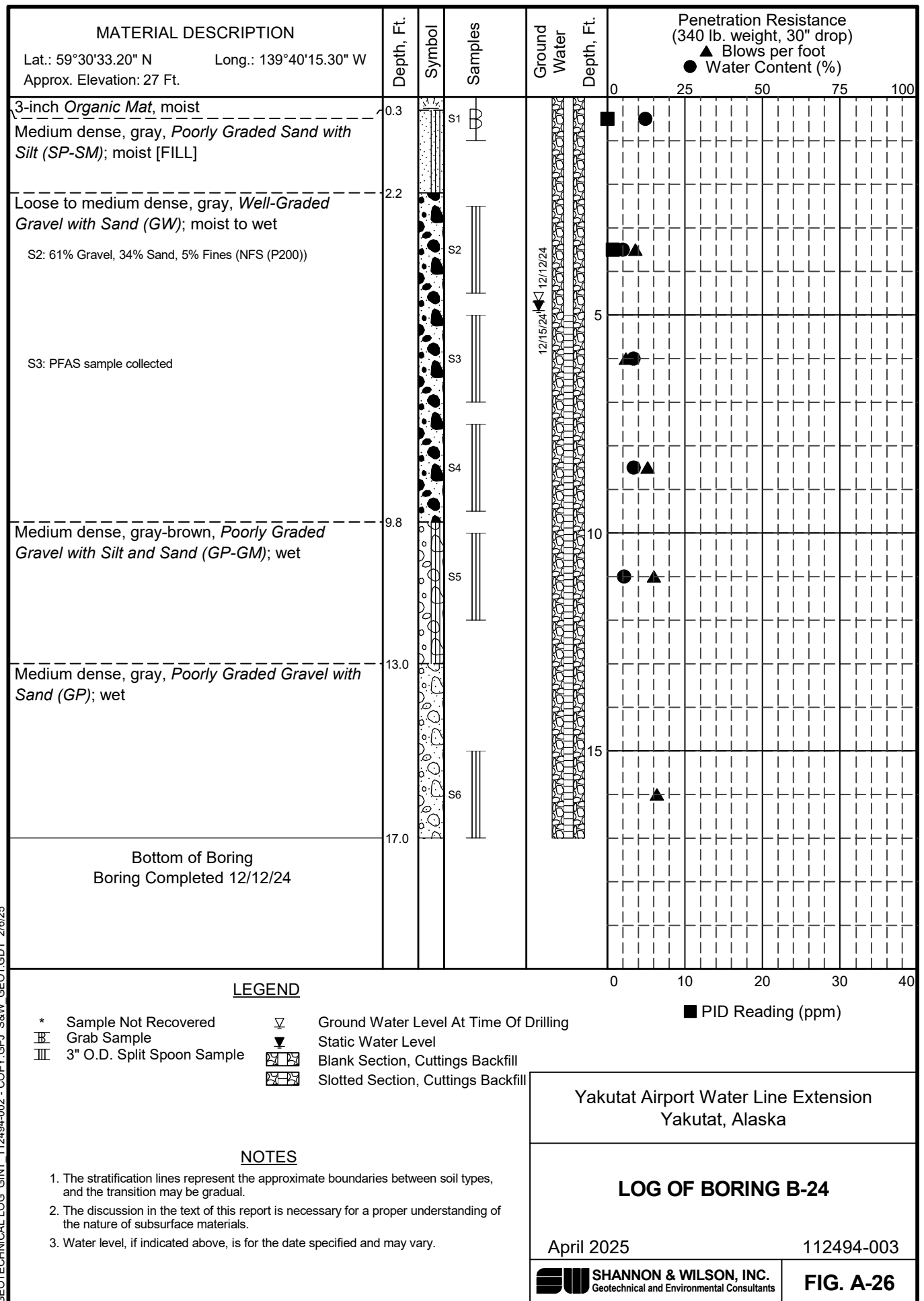
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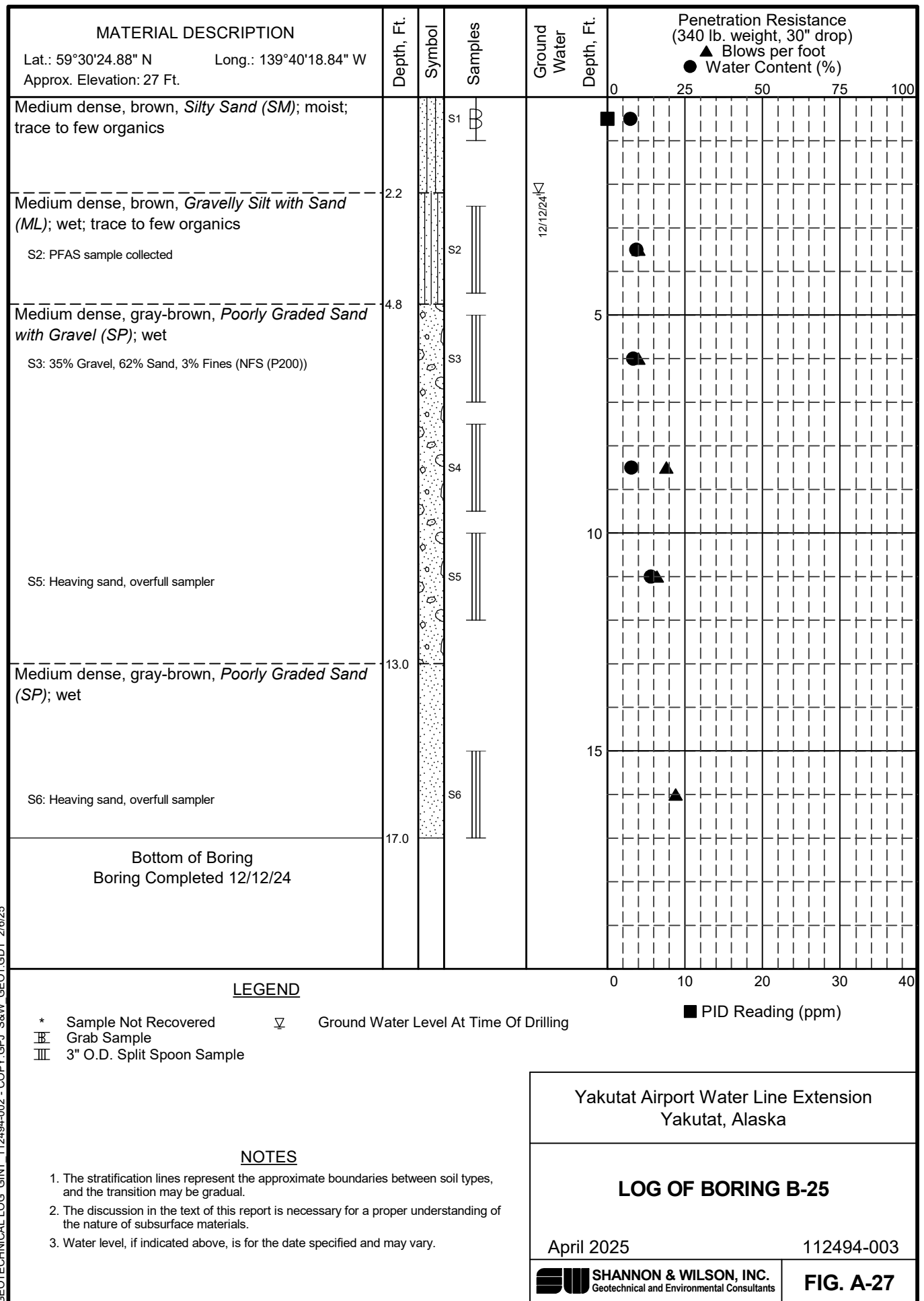
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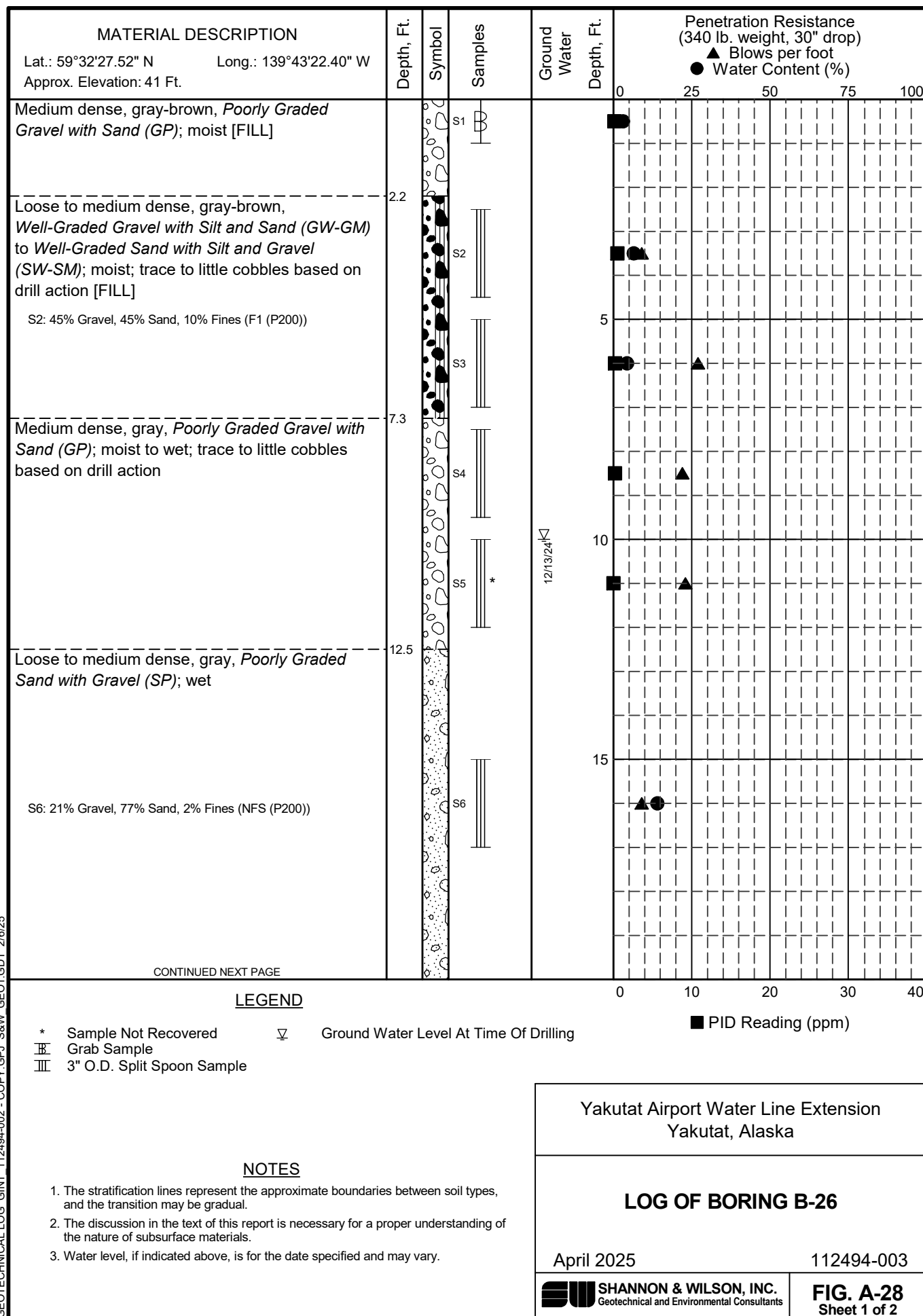
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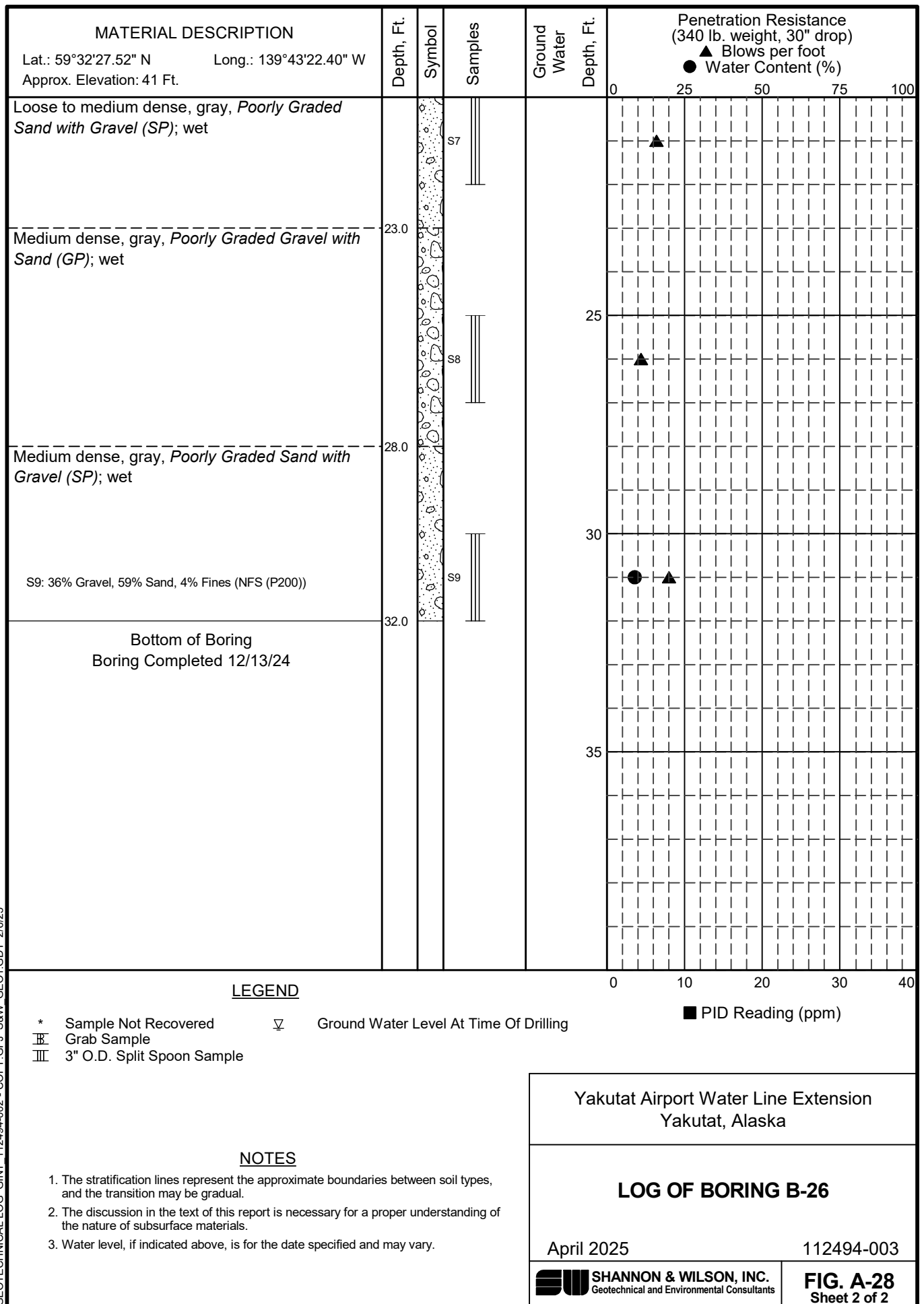
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GEOTECHNICAL LOG GINT 112494-002 - COPY.GPJ S&W GEO1.GDT 2/6/25



GEOTECHNICAL LOG GINT 112494-002 - COPY.GPJ S&W GEO1.GDT 2/6/25



[illegible]

MATERIAL DESCRIPTION	Depth, Ft.	Symbol	Samples	Ground Water Depth, Ft.
Lat.: 59°32'35.49" N Long.: 139°43'27.09" W Approx. Elevation: 72 Ft. Loose, brown, Poorly Graded Sand with Silt and Gravel (SP-SM); moist [FILL] S1: 41% Gravel, 50% Sand, 9% Fines (F2 (P200)) Medium dense to dense, brown, Poorly Graded Sand with Silt and Gravel (SP-SM); moist; trace to little cobbles based on drill action S2: Overfull sampler, slough/drill cuttings in borehole S3: 37% Gravel, 56% Sand, 7% Fines (F2 (0.02 mm)) Dense to very dense, brown and gray, Poorly Graded Gravel with Sand (GP); moist; few to little cobbles based on drill action Bottom of Boring Boring Completed 12/13/24	0 2.2 9.8 17.0	[Symbols for soil types and samples]	S1 S2 S3 S4 S5 S6	0 25 50 75 100
				Penetration Resistance (340 lb. weight, 30" drop) ▲ Blows per foot ● Water Content (%)
				PID Reading (ppm)

LEGEND

- * Sample Not Recovered
- H Grab Sample
- III 3" O.D. Split Spoon Sample

NOTES

- The stratification lines represent the approximate boundaries between soil types, and the transition may be gradual.
- The discussion in the text of this report is necessary for a proper understanding of the nature of subsurface materials.
- Water level, if indicated above, is for the date specified and may vary.

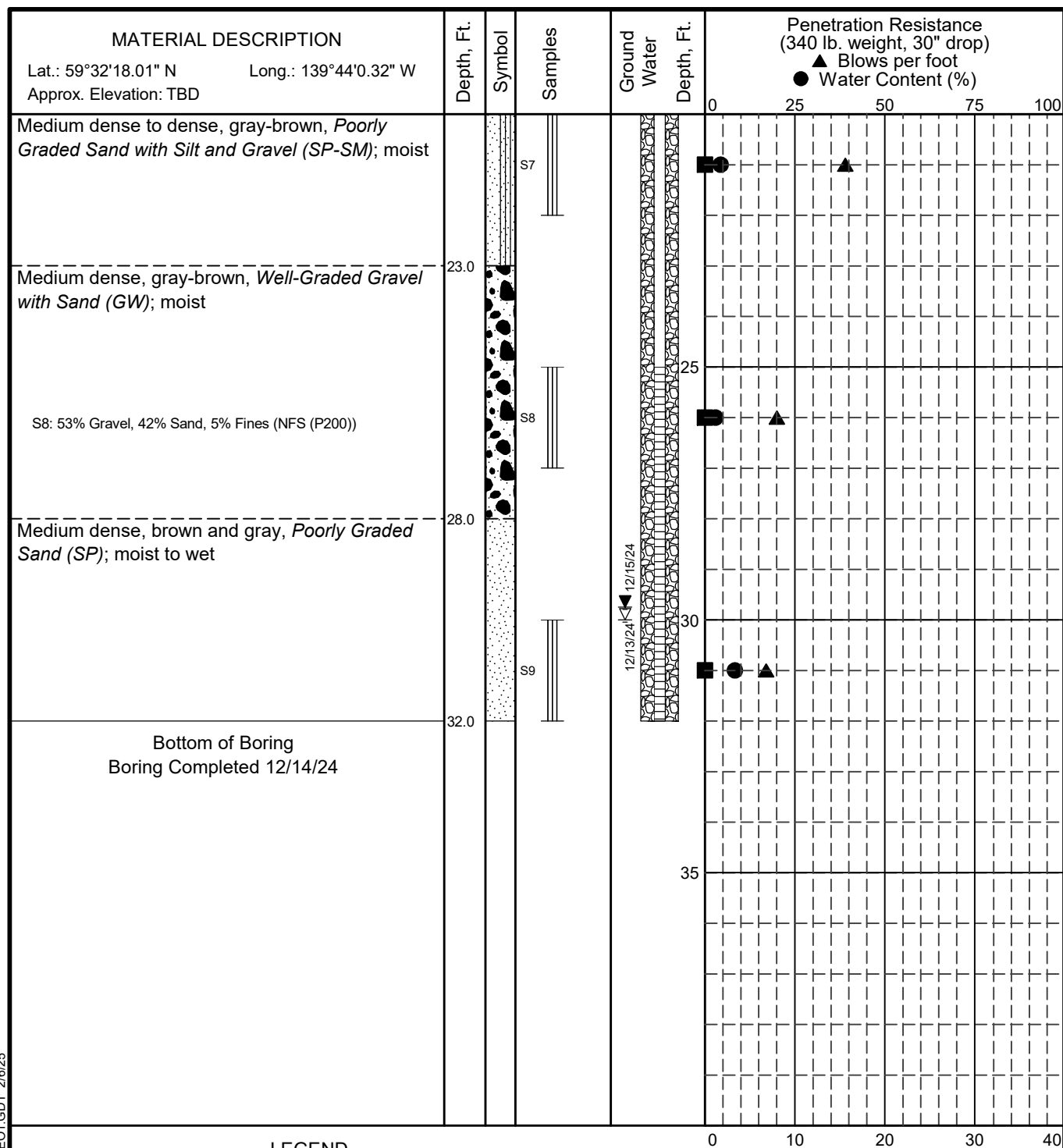
Yakutat Airport Water Line Extension
Yakutat, Alaska

LOG OF BORING B-28

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FIG. A-30



LEGEND

- | | | |
|--|---|--|
| * Sample Not Recovered |  | Ground Water Level At Time Of Drilling |
|  Grab Sample |  | Static Water Level |
|  3" O.D. Split Spoon Sample |  | Blank Section, Cuttings Backfill |
| |  | Slotted Section, Cuttings Backfill |

NOTES

1. The stratification lines represent the approximate boundaries between soil types, and the transition may be gradual.
2. The discussion in the text of this report is necessary for a proper understanding of the nature of subsurface materials.
3. Water level, if indicated above, is for the date specified and may vary.

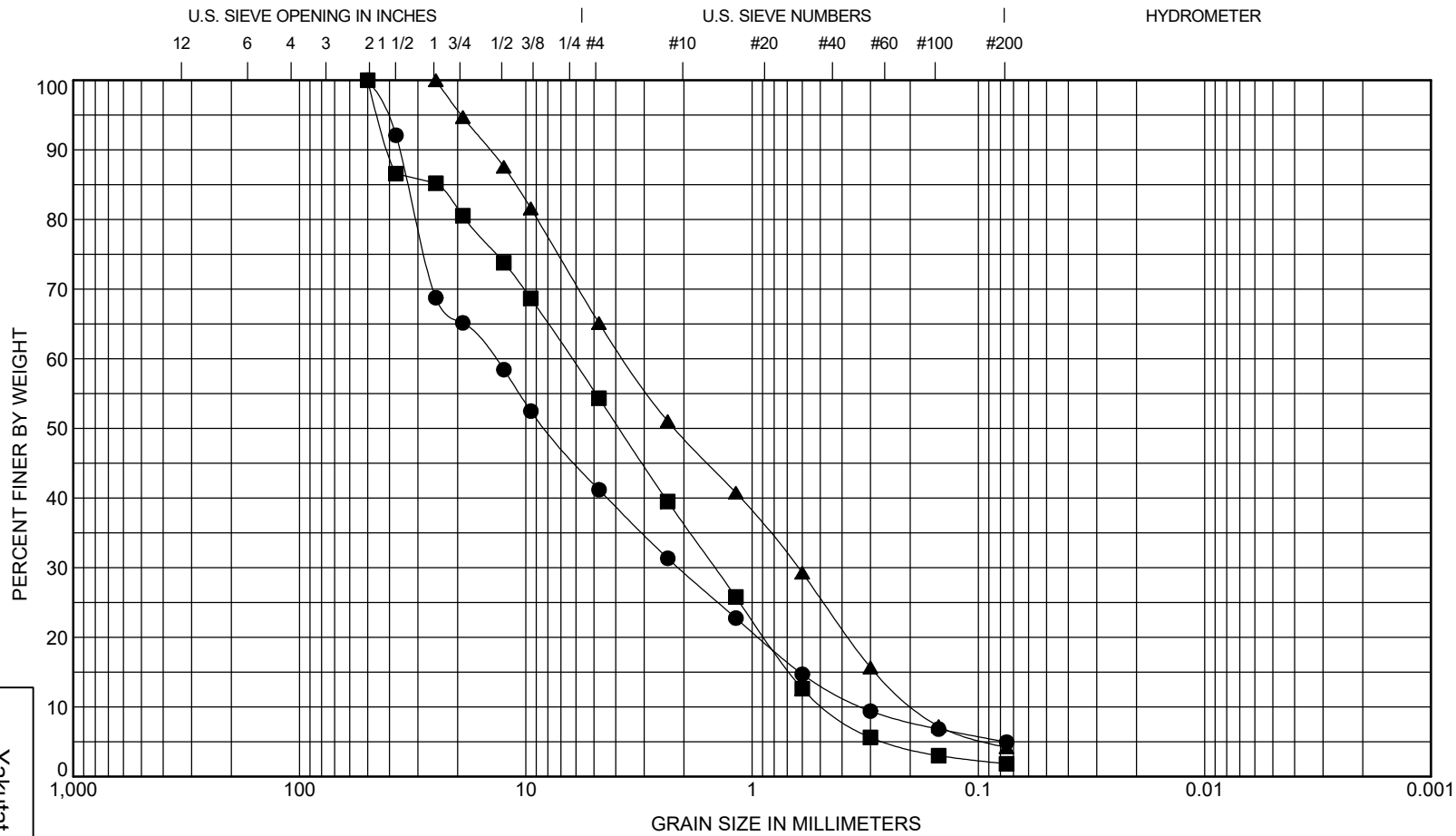
Yakutat Airport Water Line Extension
Yakutat, Alaska

LOG OF BORING B-29

April 2025

112494-003

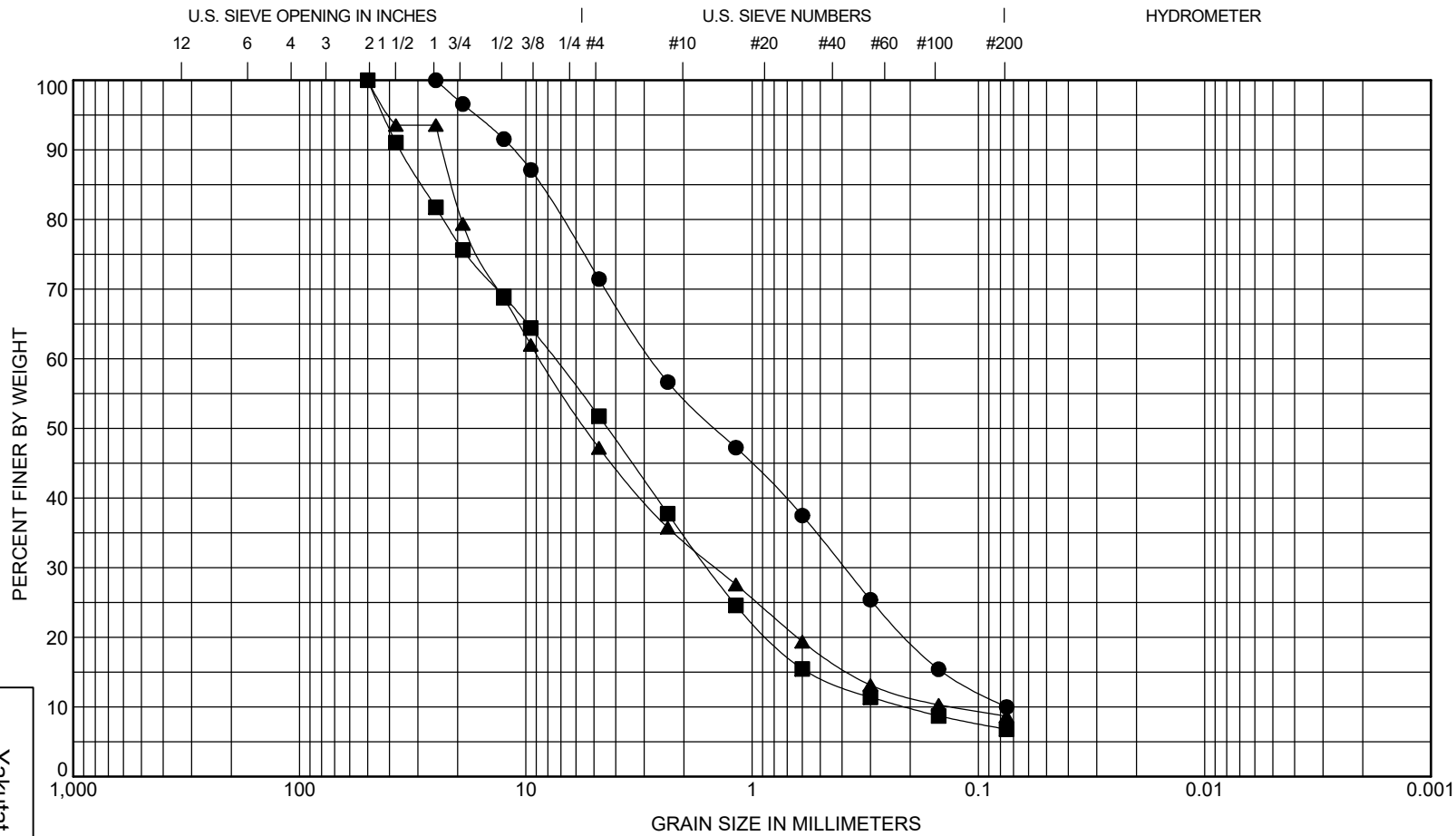
GEOTECHNICAL LOG GINT 112494-002 - COPY.GPJ S&W GEO1.GDT 2/6/25



COBBLES		GRAVEL		SAND			SILT OR CLAY				
		coarse	fine	coarse	medium	fine					
Sample	Depth, Ft	USCS Classification					LL	PL	PI	Cc	Cu
● B-01 S3	5.0 - 6.5	Well-Graded Gravel with Silt and Sand (GW-GM)								1.0	42.5
■ B-01 S8	25.0 - 26.5	Poorly Graded Sand with Gravel (SP)								0.7	13.5
▲ B-01 S10	35.0 - 36.5	Poorly Graded Sand with Gravel (SP)								0.6	19.6
Sample	Depth, Ft	D100	D60	D30	D10	%Gravel	%Sand	%Silt		%Clay	
● B-01 S3	5.0 - 6.5	50	13.78	2.12	0.32	59	36	5			
■ B-01 S8	25.0 - 26.5	50	6.25	1.46	0.46	46	52	2			
▲ B-01 S10	35.0 - 36.5	25	3.68	0.63	0.19	35	61	4			

Yakutat Airport Water Line Extension
Yakutat, Alaska

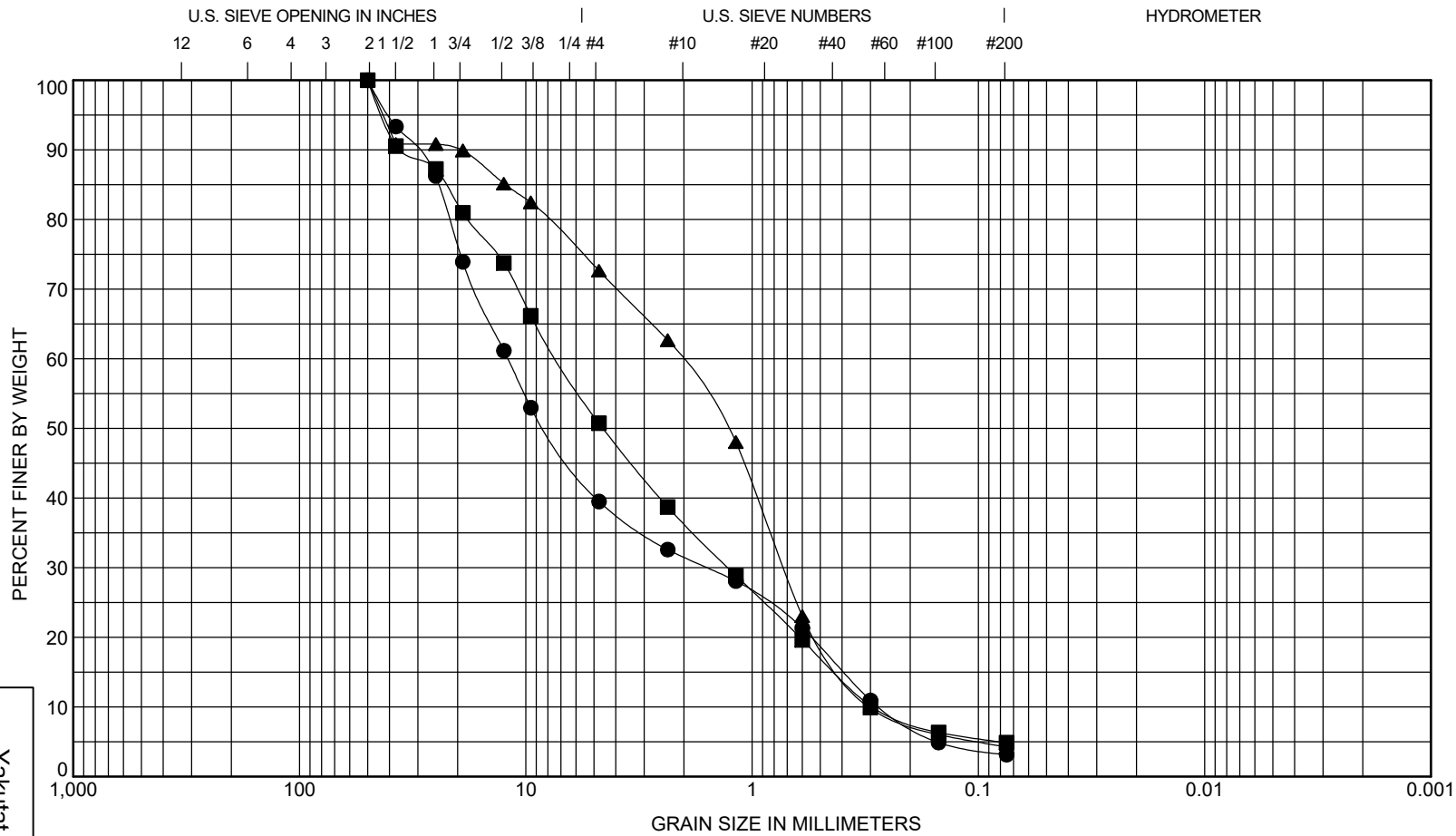
GRAIN SIZE CLASSIFICATION



COBBLES		GRAVEL		SAND			SILT OR CLAY				
		coarse	fine	coarse	medium	fine					
Sample	Depth, Ft	USCS Classification					LL	PL	PI	Cc	Cu
● B-01 S13	50.0 - 51.5	Poorly Graded Sand with Silt and Gravel (SP-SM)								0.7	36.8
■ B-02 S3	5.0 - 7.0	Well-Graded Gravel with Silt and Sand (GW-GM)								1.6	35.7
▲ B-02 S5	10.0 - 12.0	Well-Graded Gravel with Silt and Sand (GW-GM)								1.8	65.9
Sample	Depth, Ft	D100	D60	D30	D10	%Gravel	%Sand	%Silt		%Clay	
● B-01 S13	50.0 - 51.5	25	2.76	0.39	0.08	29	61	10			
■ B-02 S3	5.0 - 7.0	50	7.46	1.57	0.21	48	45	7			
▲ B-02 S5	10.0 - 12.0	50	8.66	1.45	0.13	53	39	9			

Yakutat Airport Water Line Extension
Yakutat, Alaska

GRAIN SIZE CLASSIFICATION

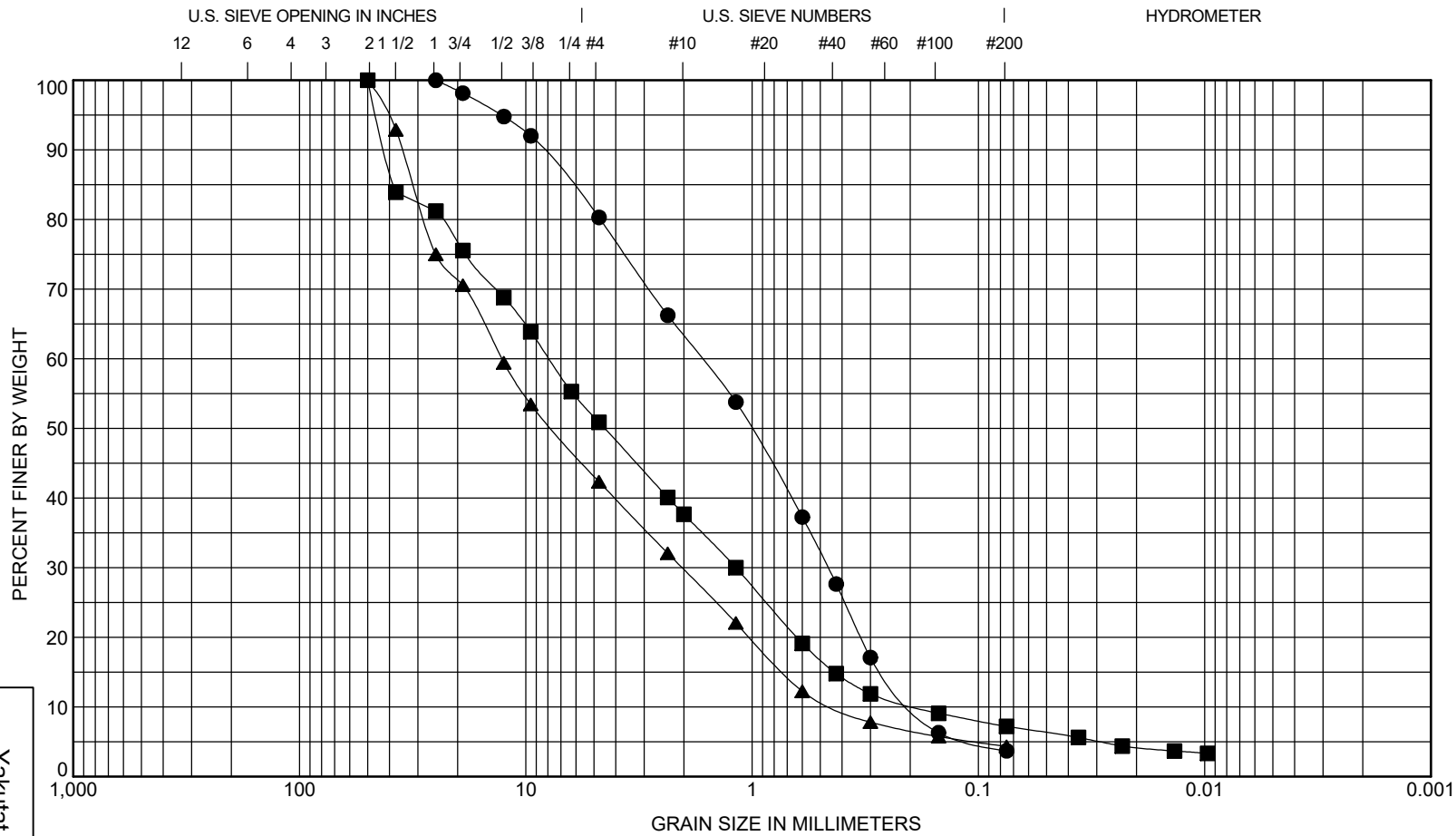


COBBLES		GRAVEL		SAND			SILT OR CLAY				
		coarse	fine	coarse	medium	fine					
Sample	Depth, Ft	USCS Classification					LL	PL	PI	Cc	Cu
● B-02 S7	20.0 - 22.0	Poorly Graded Gravel with Sand (GP)								0.8	44.7
■ B-03 S3	5.0 - 7.0	Poorly Graded Gravel with Sand (GP)								0.8	24.7
▲ B-03 S5	10.0 - 12.0	Poorly Graded Sand with Gravel (SP)								0.8	6.9
Sample	Depth, Ft	D100	D60	D30	D10	%Gravel	%Sand	%Silt		%Clay	
● B-02 S7	20.0 - 22.0	50	12.03	1.59	0.27	60	36	3			
■ B-03 S3	5.0 - 7.0	50	7.2	1.28	0.29	49	46	5			
▲ B-03 S5	10.0 - 12.0	50	2.08	0.72	0.3	27	68	4			

Yakutat Airport Water Line Extension
Yakutat, Alaska

GRAIN SIZE CLASSIFICATION

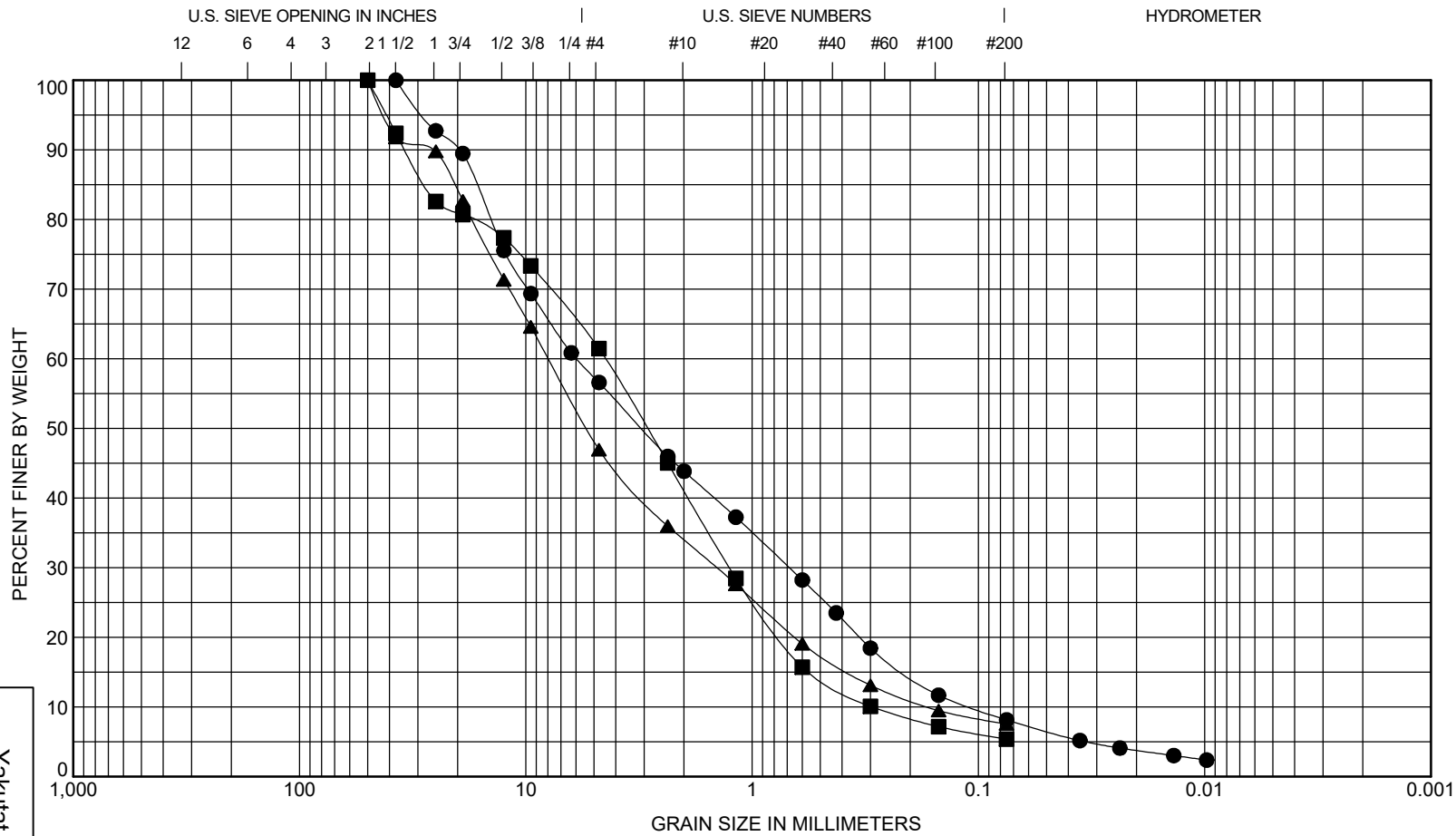
April 2025 112494-003

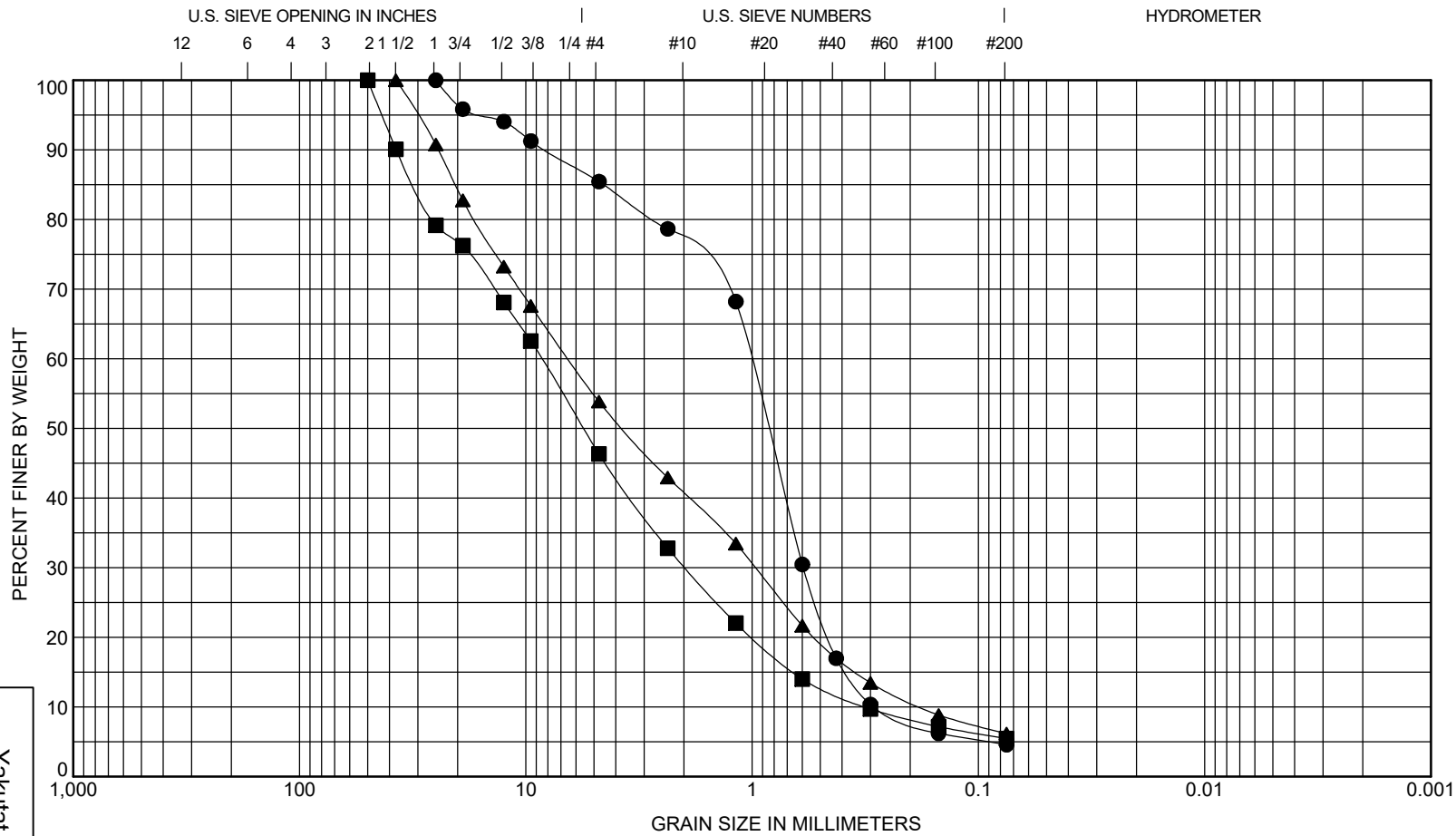


		GRAVEL		SAND			SILT OR CLAY				
		coarse	fine	coarse	medium	fine					
Sample	Depth, Ft	USCS Classification					LL	PL	PI	Cc	Cu
● B-03 S7	20.0 - 22.0	Poorly Graded Sand with Gravel (SP)								0.7	8.8
■ B-04 S2	2.5 - 4.5	Poorly Graded Gravel with Silt and Sand (GP-GM)								0.9	42.0
▲ B-04 S4	7.5 - 9.5	Poorly Graded Gravel with Sand (GP)								0.8	30.2
Sample	Depth, Ft	D100	D60	D30	D10	%Gravel	%Sand	%Silt		%Clay	
● B-03 S7	20.0 - 22.0	25	1.67	0.46	0.19	20	77	4			
■ B-04 S2	2.5 - 4.5	50	7.89	1.18	0.19	49	44	7			
▲ B-04 S4	7.5 - 9.5	50	12.77	2.05	0.42	58	38	4			

Yakutat Airport Water Line Extension
Yakutat, Alaska

GRAIN SIZE CLASSIFICATION

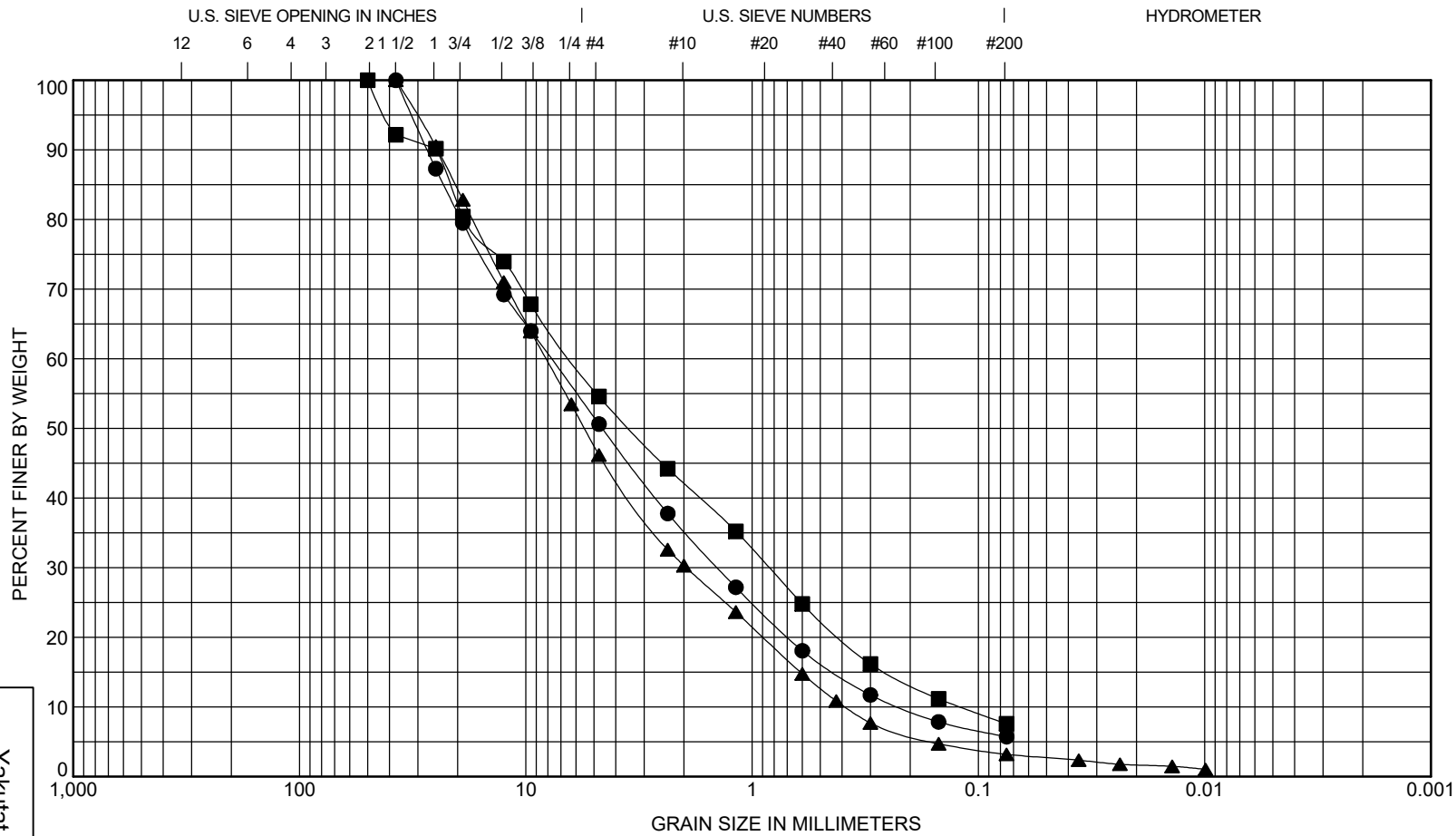




COBBLES		GRAVEL		SAND			SILT OR CLAY				
		coarse	fine	coarse	medium	fine					
Sample	Depth, Ft	USCS Classification					LL	PL	PI	Cc	Cu
● B-07 S3B	5.7 - 7.0	Poorly Graded Sand with Gravel (SP)								1.2	3.6
■ B-07 S5	10.0 - 12.0	Well-Graded Gravel with Silt and Sand (GW-GM)								1.4	27.0
▲ B-08 S2	2.5 - 4.5	Poorly Graded Sand with Silt and Gravel (SP-SM)								0.8	36.4
Sample	Depth, Ft	D100	D60	D30	D10	%Gravel	%Sand	%Silt		%Clay	
● B-07 S3B	5.7 - 7.0	25	1.02	0.59	0.28	15	81	5			
■ B-07 S5	10.0 - 12.0	50	8.52	1.97	0.32	54	41	5			
▲ B-08 S2	2.5 - 4.5	37.5	6.48	0.97	0.18	46	48	6			

Yakutat Airport Water Line Extension
Yakutat, Alaska

GRAIN SIZE CLASSIFICATION

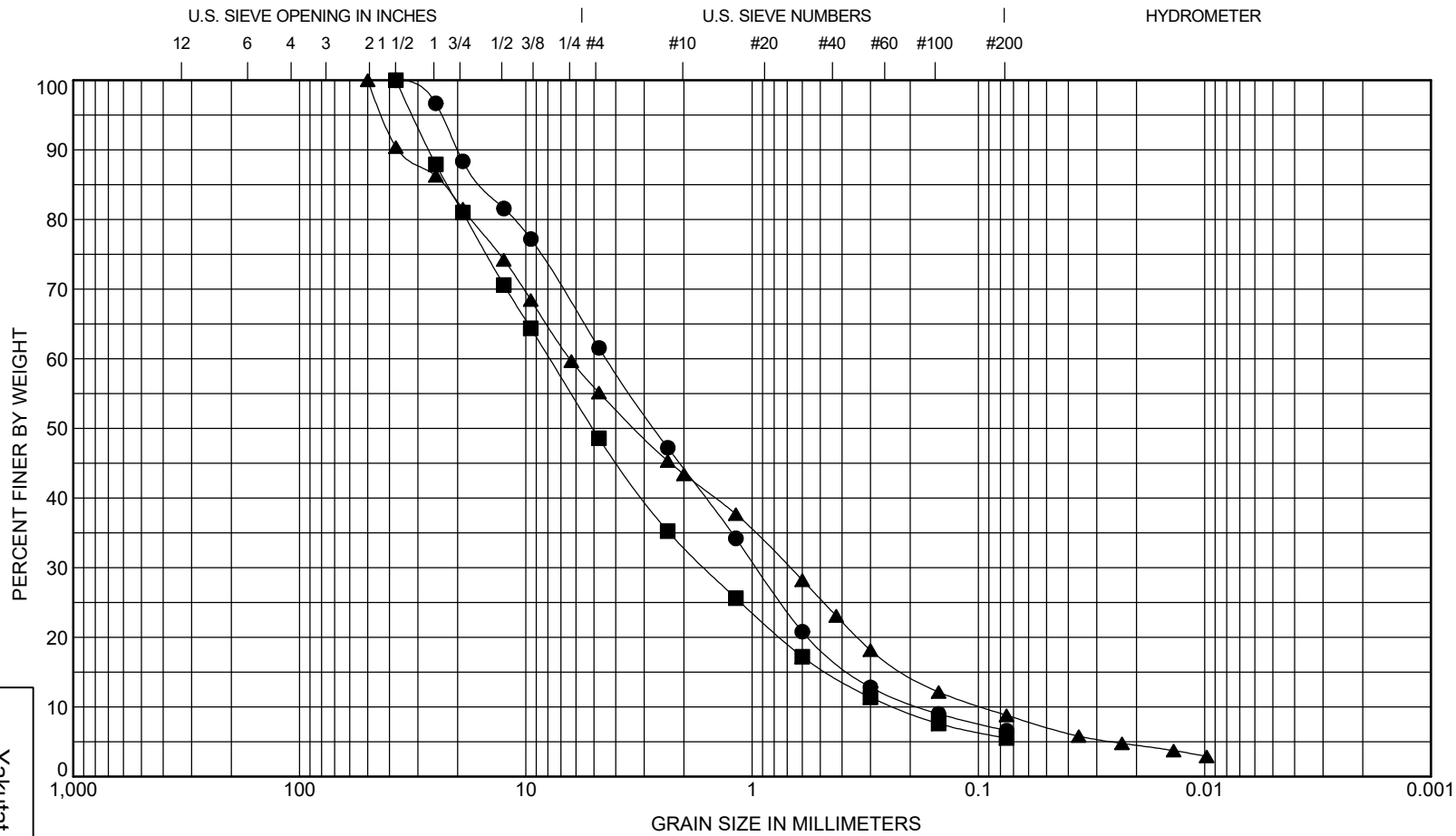


		GRAVEL		SAND			SILT OR CLAY				
		coarse	fine	coarse	medium	fine					
Sample	Depth, Ft	USCS Classification					LL	PL	PI	Cc	Cu
● B-08 S5	10.0 - 12.0	Well-Graded Gravel with Silt and Sand (GW-GM)								1.2	35.0
■ B-09 S3	5.0 - 7.0	Poorly Graded Sand with Silt and Gravel (SP-SM)								0.9	52.7
▲ B-09 S5	10.0 - 12.0	Well-Graded Gravel with Sand (GW)								1.2	21.1
Sample	Depth, Ft	D100	D60	D30	D10	%Gravel	%Sand	%Silt		%Clay	
● B-08 S5	10.0 - 12.0	37.5	7.72	1.42	0.22	49	45	6			
■ B-09 S3	5.0 - 7.0	50	6.31	0.84	0.12	45	47	8			
▲ B-09 S5	10.0 - 12.0	37.5	8.15	1.95	0.39	54	43	3			

Yakutat Airport Water Line Extension
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GRAIN SIZE CLASSIFICATION

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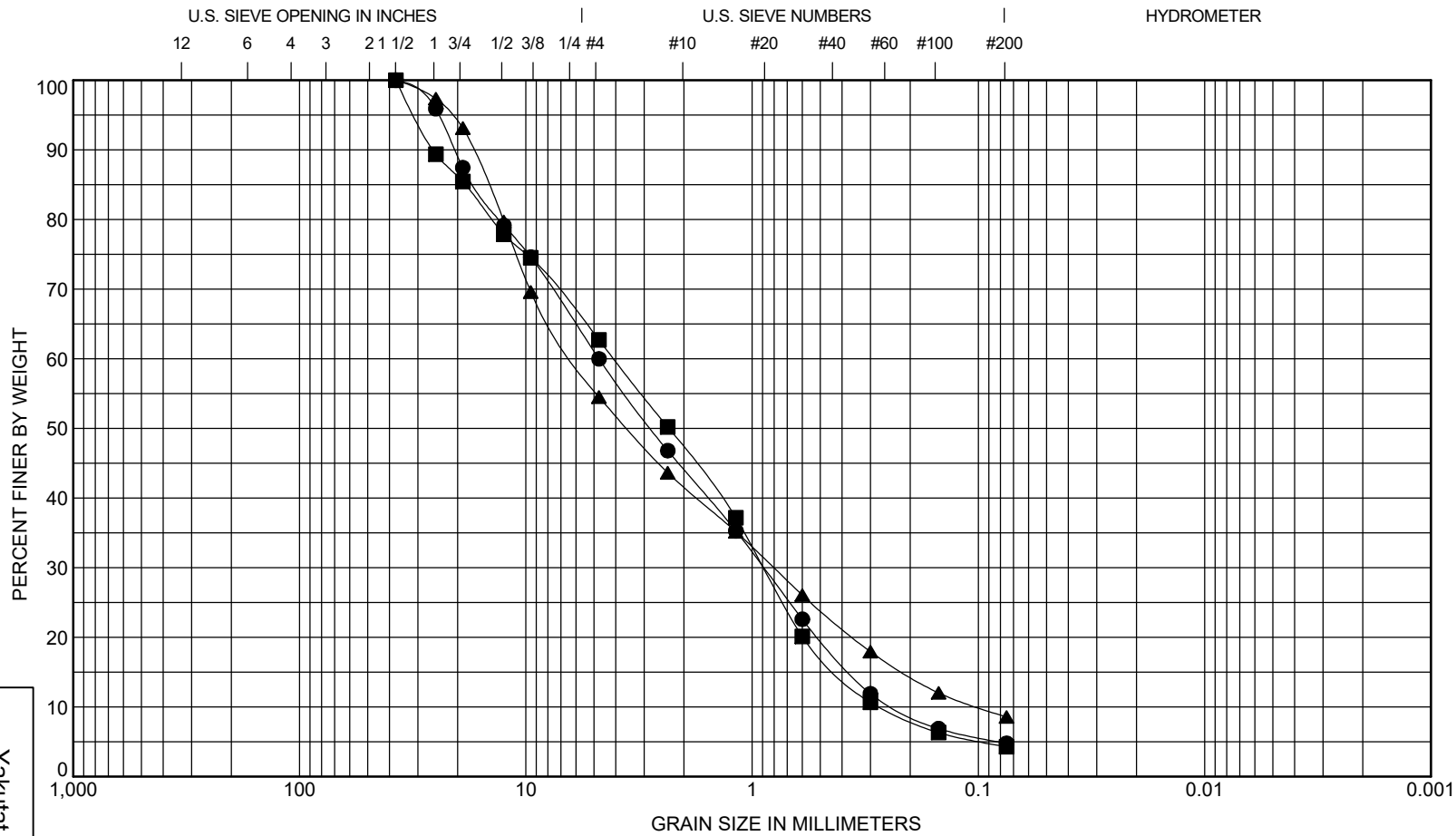
COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Sample			Depth, Ft	USCS Classification					LL	PL	PI	Cc	Cu
●	B-10 S3	5.0 - 7.0	Well-Graded Sand with Silt and Gravel (SW-SM)								1.2	24.5	
■	B-10 S5	10.0 - 12.0	Well-Graded Gravel with Silt and Sand (GW-GM)								1.4	33.7	
▲	B-11 S2	2.5 - 4.5	Poorly Graded Sand with Silt and Gravel (SP-SM)								0.8	66.4	
Sample			Depth, Ft	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay		
●	B-10 S3	5.0 - 7.0	37.5	4.4	0.95	0.18	38	55	7				
■	B-10 S5	10.0 - 12.0	37.5	7.84	1.62	0.23	51	43	6				
▲	B-11 S2	2.5 - 4.5	50	6.4	0.68	0.1	45	46	9				

Yakutat Airport Water Line Extension
Yakutat, Alaska

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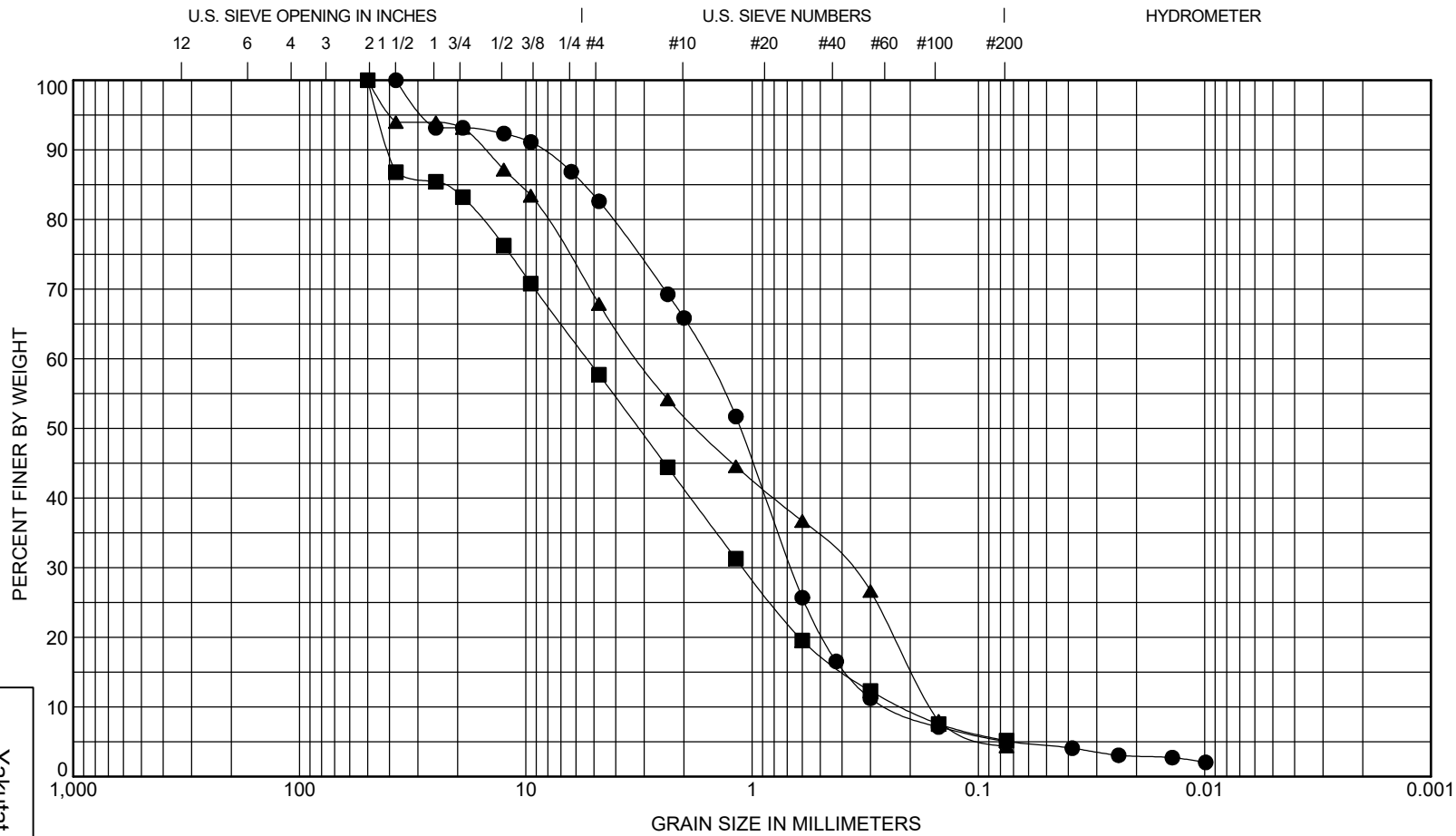


COBBLES			GRAVEL		SAND			SILT OR CLAY					
			coarse	fine	coarse	medium	fine						
Sample		Depth, Ft	USCS Classification						LL	PL	PI	Cc	Cu
●	B-11 S4	7.5 - 9.5	Poorly Graded Sand with Gravel (SP)									0.7	20.6
■	B-12 S4	7.5 - 9.5	Poorly Graded Sand with Gravel (SP)									0.7	15.1
▲	B-13 S1	0.0 - 2.0	Well-Graded Sand with Silt and Gravel (SW-SM)									1.1	61.1
Sample		Depth, Ft	D100	D60	D30	D10	%Gravel	%Sand	%Silt		%Clay		
●	B-11 S4	7.5 - 9.5	37.5	4.75	0.89	0.23	40	55	5				
■	B-12 S4	7.5 - 9.5	37.5	4.08	0.89	0.27	37	58	4				
▲	B-13 S1	0.0 - 2.0	37.5	6.12	0.8	0.1	45	46	9				

Yakutat Airport Water Line Extension
Yakutat, Alaska

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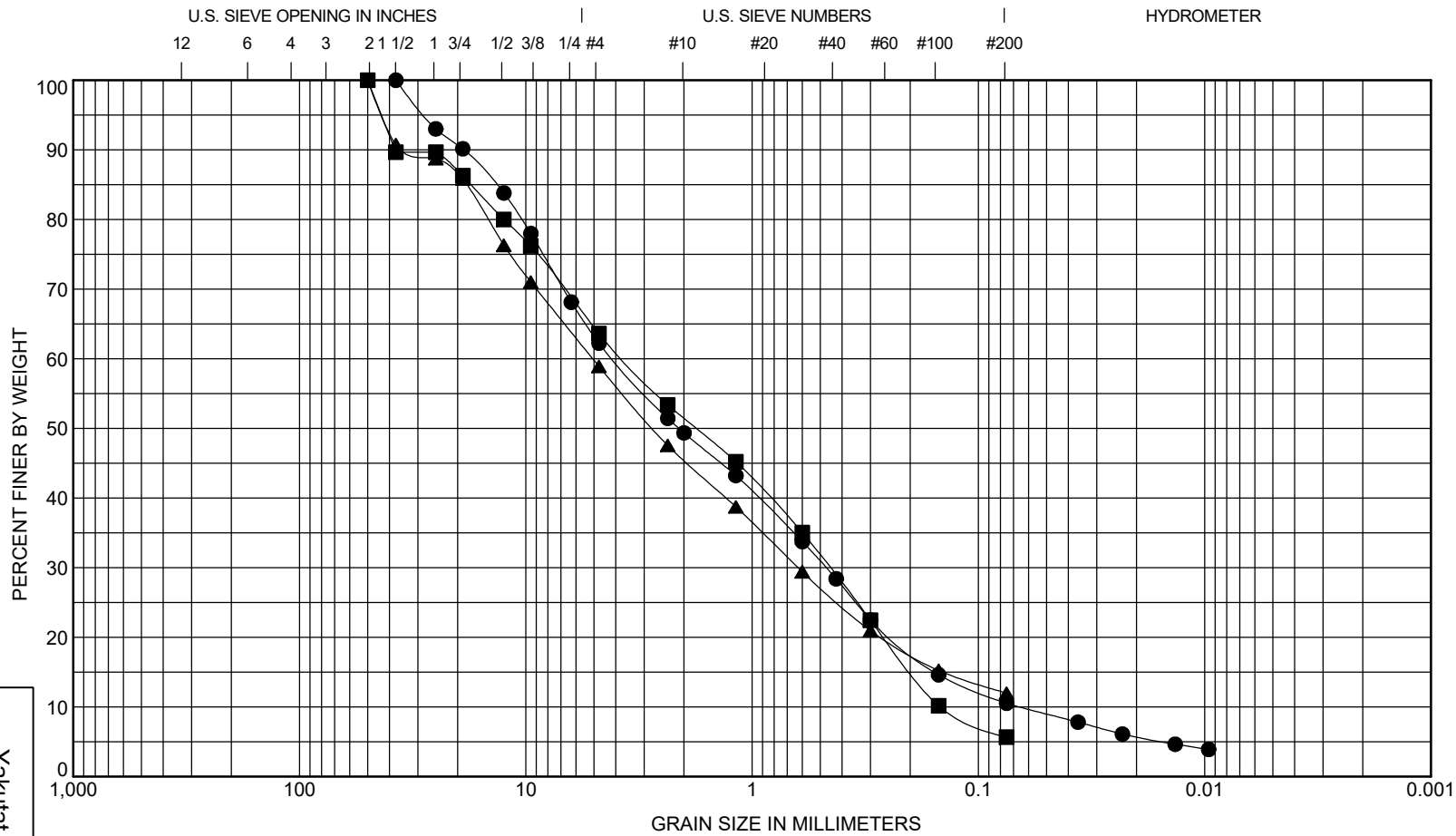
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COBBLES		GRAVEL		SAND			SILT OR CLAY				
		coarse	fine	coarse	medium	fine					
Sample	Depth, Ft	USCS Classification					LL	PL	PI	Cc	Cu
● B-13 S5	10.0 - 12.0	Well-Graded Sand with Silt and Gravel (SW-SM)								1.1	6.6
■ B-14 S4	7.5 - 9.5	Well-Graded Sand with Silt and Gravel (SW-SM)								1.0	25.0
▲ B-14 S5	10.0 - 12.0	Poorly Graded Sand with Gravel (SP)								0.3	19.7
Sample	Depth, Ft	D100	D60	D30	D10	%Gravel	%Sand	%Silt		%Clay	
● B-13 S5	10.0 - 12.0	37.5	1.61	0.67	0.24	17	78	5			
■ B-14 S4	7.5 - 9.5	50	5.36	1.1	0.21	42	53	5			
▲ B-14 S5	10.0 - 12.0	50	3.18	0.38	0.16	32	64	4			

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COBBLES		GRAVEL		SAND			SILT OR CLAY				
		coarse	fine	coarse	medium	fine					
Sample	Depth, Ft	USCS Classification					LL	PL	PI	Cc	Cu
● B-15 S2	2.5 - 4.5	Poorly Graded Sand with Silt and Gravel (SP-SM)								0.8	63.2
■ B-15 S5	10.0 - 12.0	Poorly Graded Sand with Silt and Gravel (SP-SM)								0.4	25.4
▲ B-16 S3	5.0 - 7.0	Well-Graded Sand with Silt and Gravel (SW-SM)								1.6	102.9
Sample	Depth, Ft	D100	D60	D30	D10	%Gravel	%Sand	%Silt		%Clay	
● B-15 S2	2.5 - 4.5	37.5	4.11	0.47	0.06	38	52	11			
■ B-15 S5	10.0 - 12.0	50	3.71	0.45	0.15	36	58	6			
▲ B-16 S3	5.0 - 7.0	50	5.06	0.63		41	47	12			

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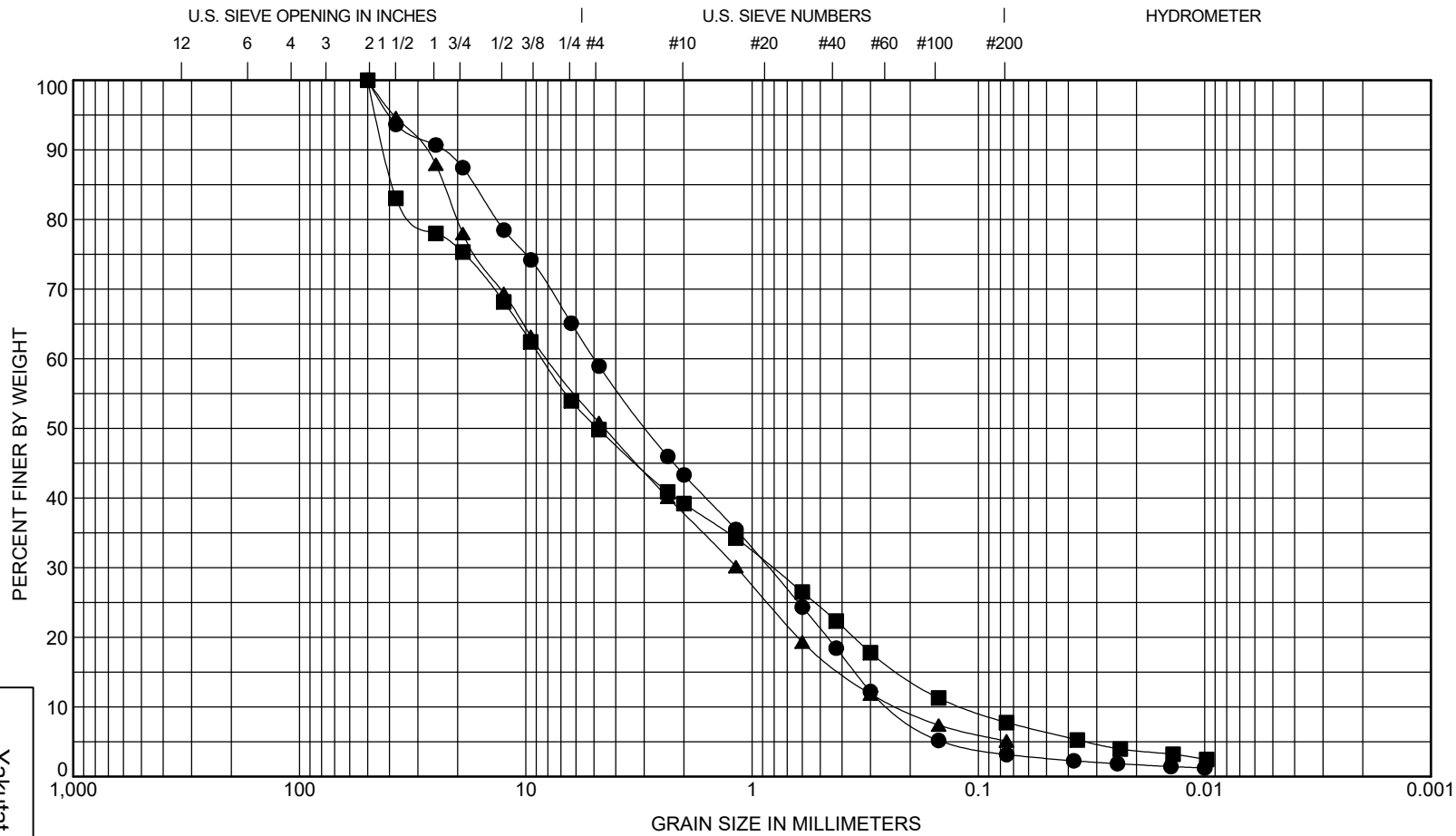
GRAIN SIZE CLASSIFICATION

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

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FIG. A-32
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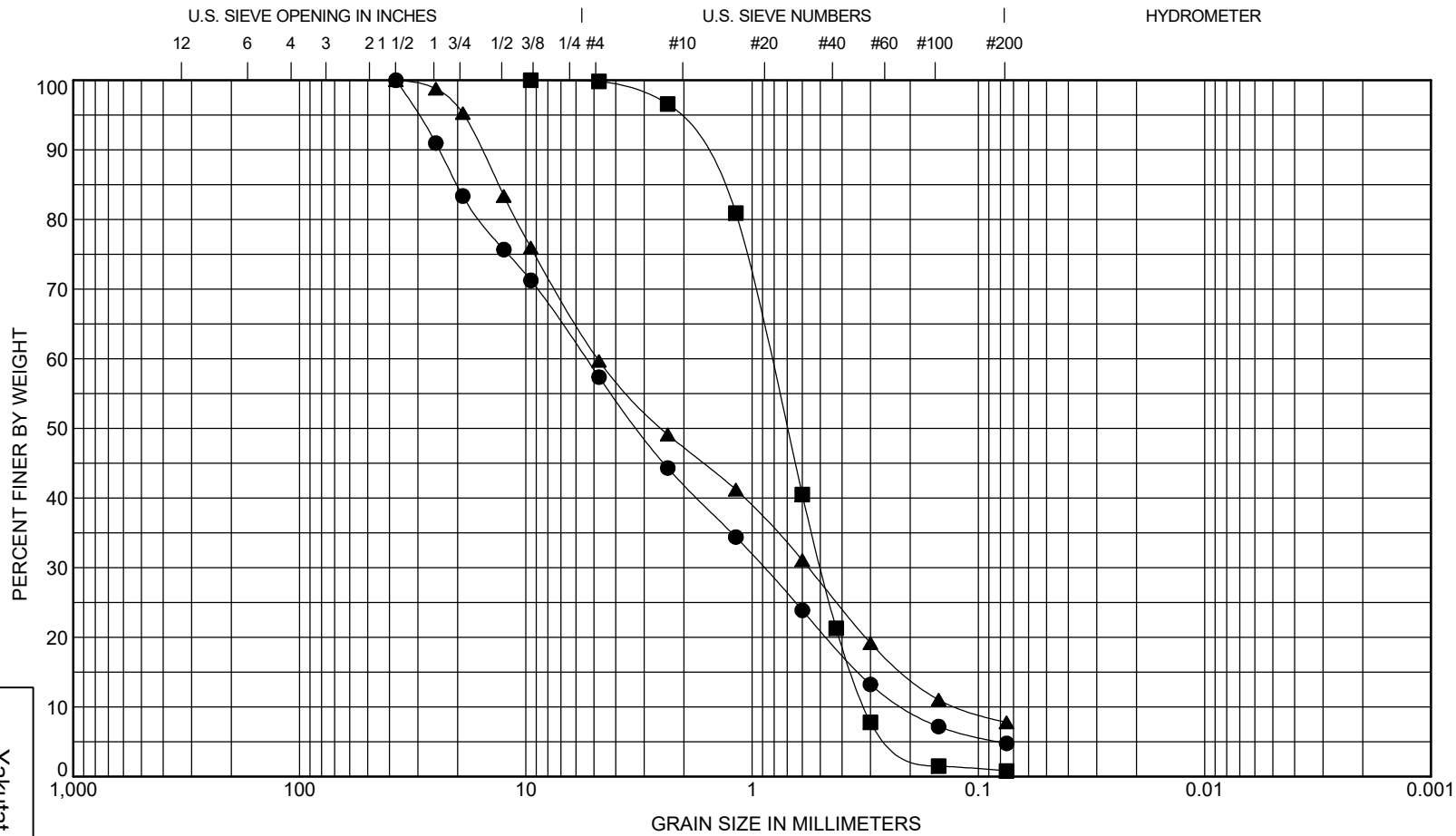
112494-003

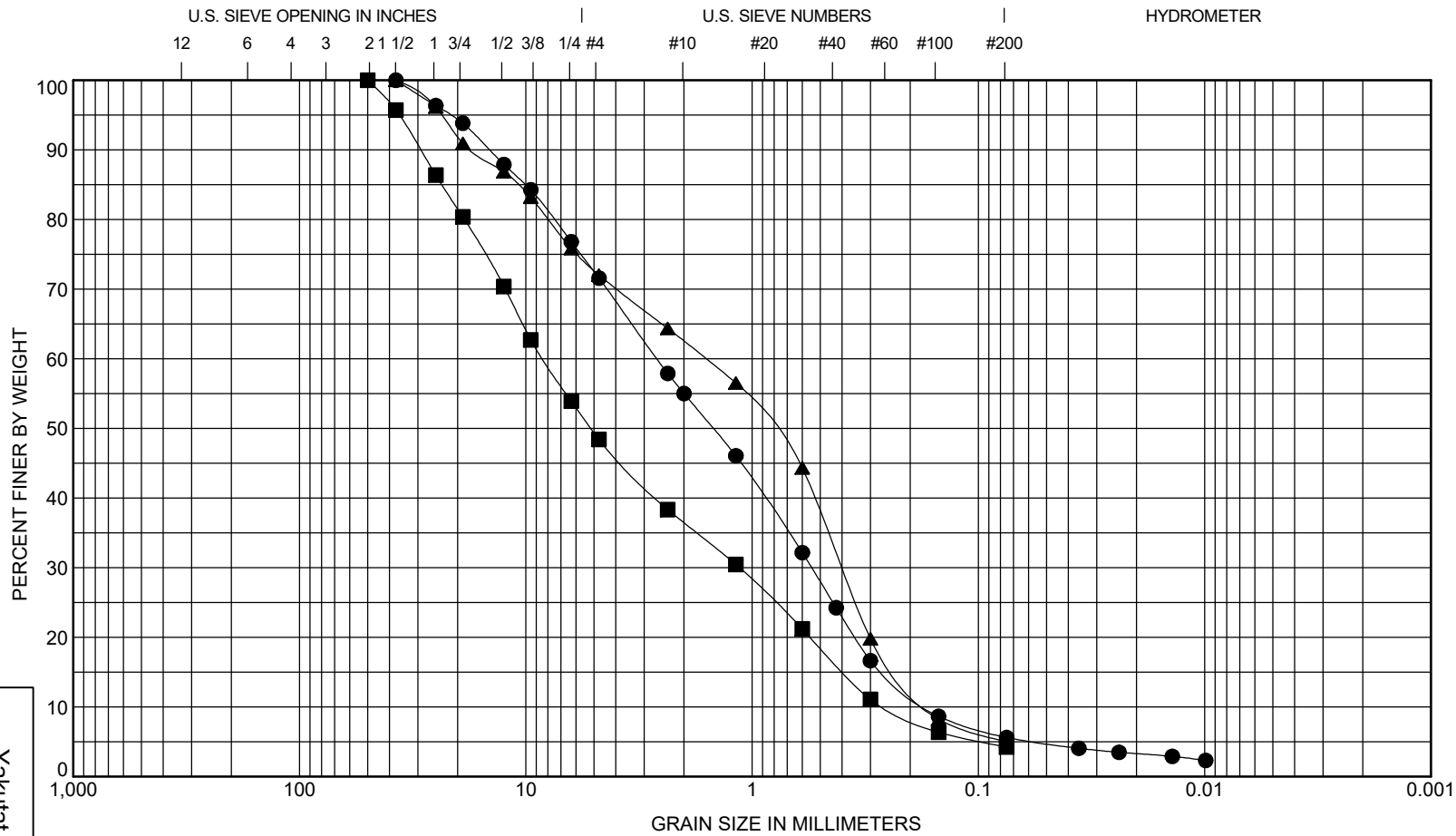


COBBLES			GRAVEL		SAND			SILT OR CLAY					
			coarse	fine	coarse	medium	fine						
Sample		Depth, Ft	USCS Classification						LL	PL	PI	Cc	Cu
●	B-16 S5	10.0 - 12.0	Poorly Graded Sand with Gravel (SP)									0.6	20.7
■	B-17 S2	2.5 - 4.5	Poorly Graded Gravel with Silt and Sand (GP-GM)									0.7	72.5
▲	B-17 S4	7.5 - 9.5	Poorly Graded Gravel with Silt and Sand (GP-GM)									0.8	35.3
Sample		Depth, Ft	D100	D60	D30	D10	%Gravel	%Sand	%Silt		%Clay		
●	B-16 S5	10.0 - 12.0	50	4.99	0.85	0.24	41	56	3				
■	B-17 S2	2.5 - 4.5	50	8.45	0.82	0.12	50	42	8				
▲	B-17 S4	7.5 - 9.5	50	7.93	1.17	0.22	49	46	5				

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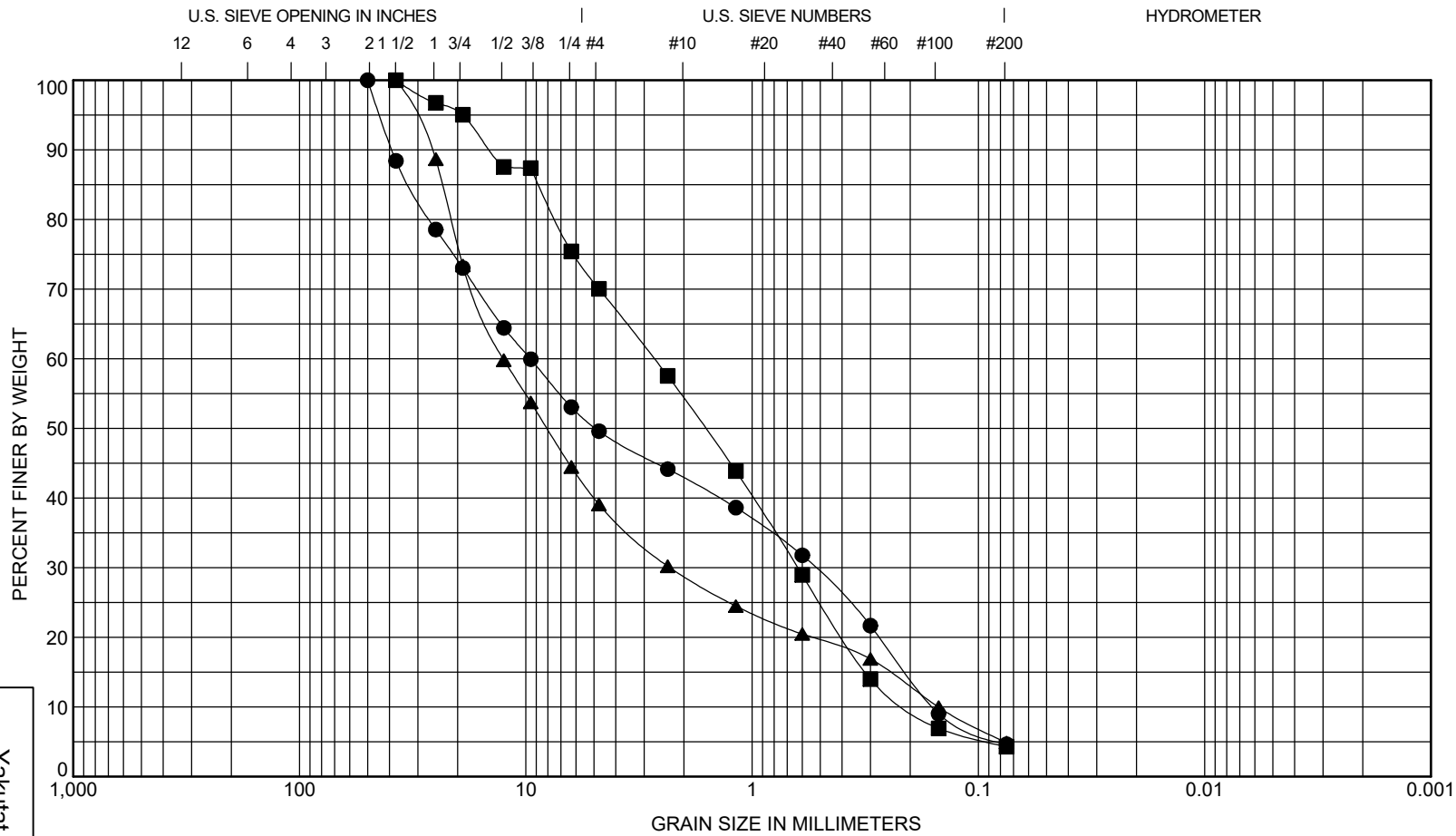


		GRAVEL		SAND			SILT OR CLAY				
		coarse	fine	coarse	medium	fine					
Sample	Depth, Ft	USCS Classification					LL	PL	PI	Cc	Cu
● B-19 S4	7.5 - 9.5	Poorly Graded Sand with Silt and Gravel (SP-SM)								0.7	15.6
■ B-20 S5	10.0 - 12.0	Poorly Graded Gravel with Sand (GP)								0.6	32.7
▲ B-21 S3	5.0 - 7.0	Poorly Graded Sand with Silt and Gravel (SP-SM)								0.6	9.6
Sample	Depth, Ft	D100	D60	D30	D10	%Gravel	%Sand	%Silt		%Clay	
● B-19 S4	7.5 - 9.5	37.5	2.63	0.55	0.17	28	66	6			
■ B-20 S5	10.0 - 12.0	50	8.37	1.14	0.26	52	44	4			
▲ B-21 S3	5.0 - 7.0	37.5	1.6	0.4	0.17	28	67	5			

Yakutat Airport Water Line Extension
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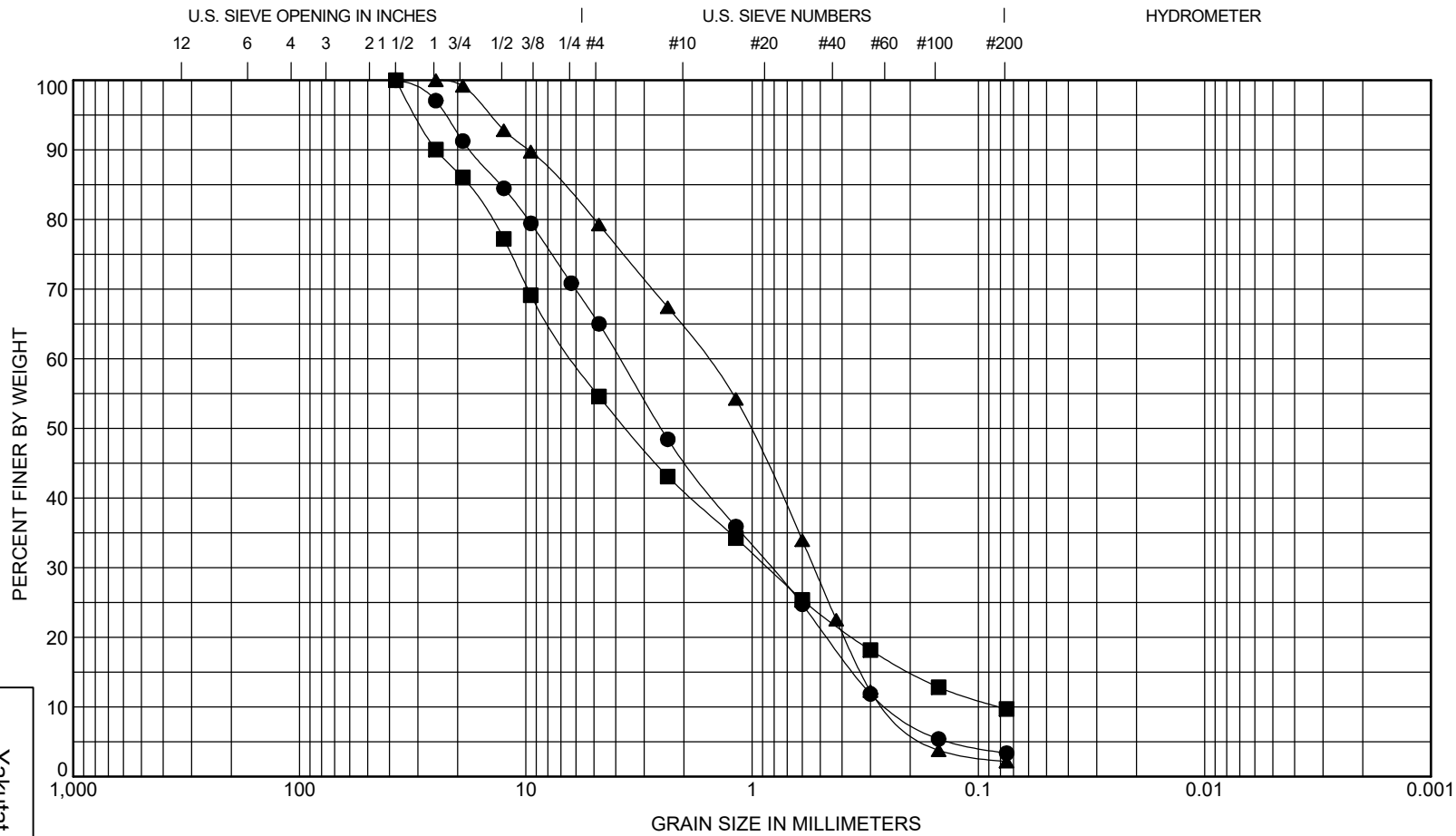
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		GRAVEL		SAND			SILT OR CLAY				
		coarse	fine	coarse	medium	fine					
Sample	Depth, Ft	USCS Classification					LL	PL	PI	Cc	Cu
● B-22 S2	2.5 - 4.5	Poorly Graded Gravel with Sand (GP)								0.2	60.3
■ B-23 S4	7.5 - 9.5	Poorly Graded Sand with Gravel (SP)								0.7	13.4
▲ B-24 S2	2.5 - 4.5	Well-Graded Gravel with Sand (GW)								2.8	83.7
Sample	Depth, Ft	D100	D60	D30	D10	%Gravel	%Sand	%Silt		%Clay	
● B-22 S2	2.5 - 4.5	50	9.54	0.53	0.16	50	45	5			
■ B-23 S4	7.5 - 9.5	37.5	2.71	0.63	0.2	30	66	4			
▲ B-24 S2	2.5 - 4.5	37.5	12.58	2.3	0.15	61	34	5			

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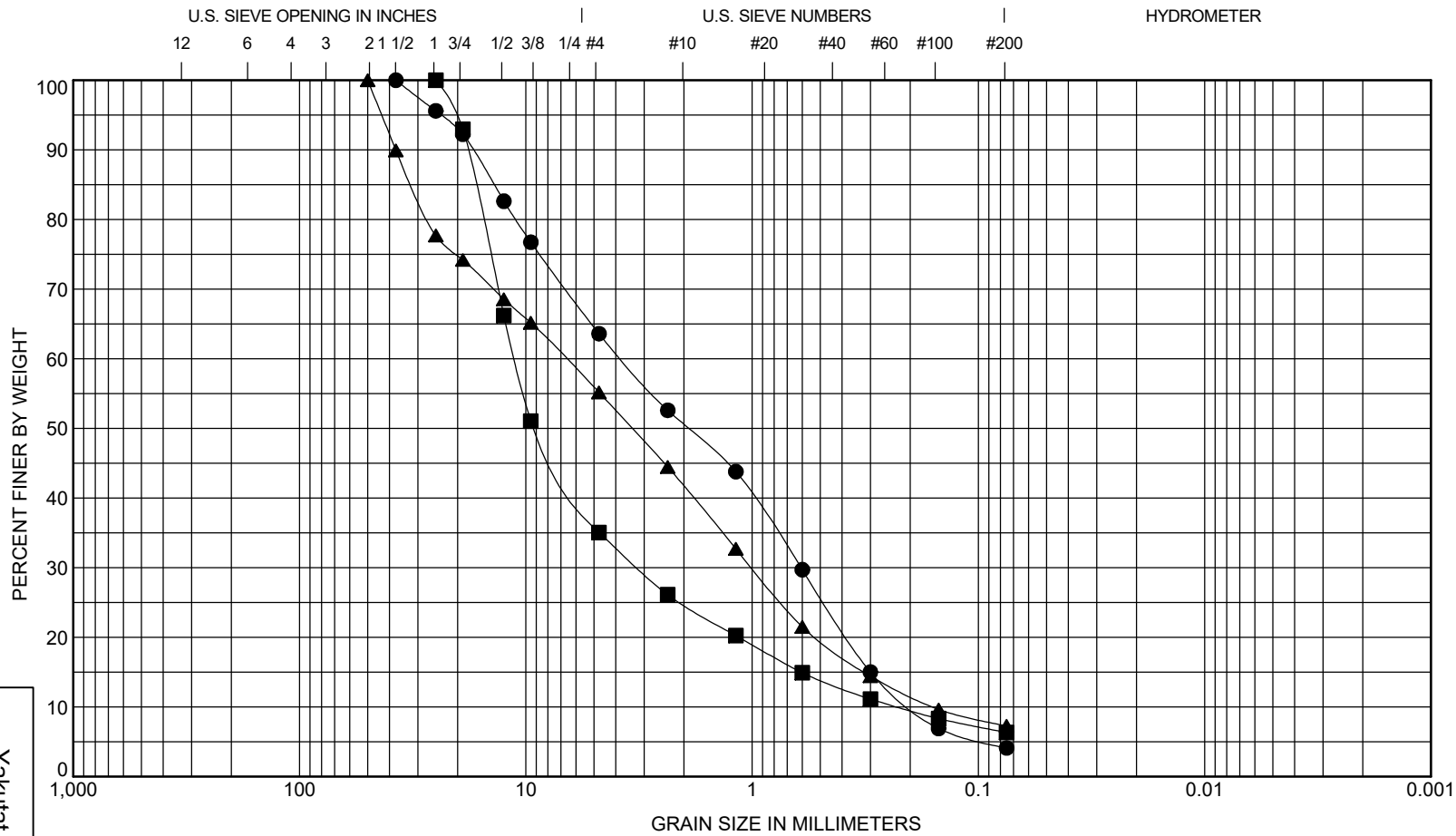


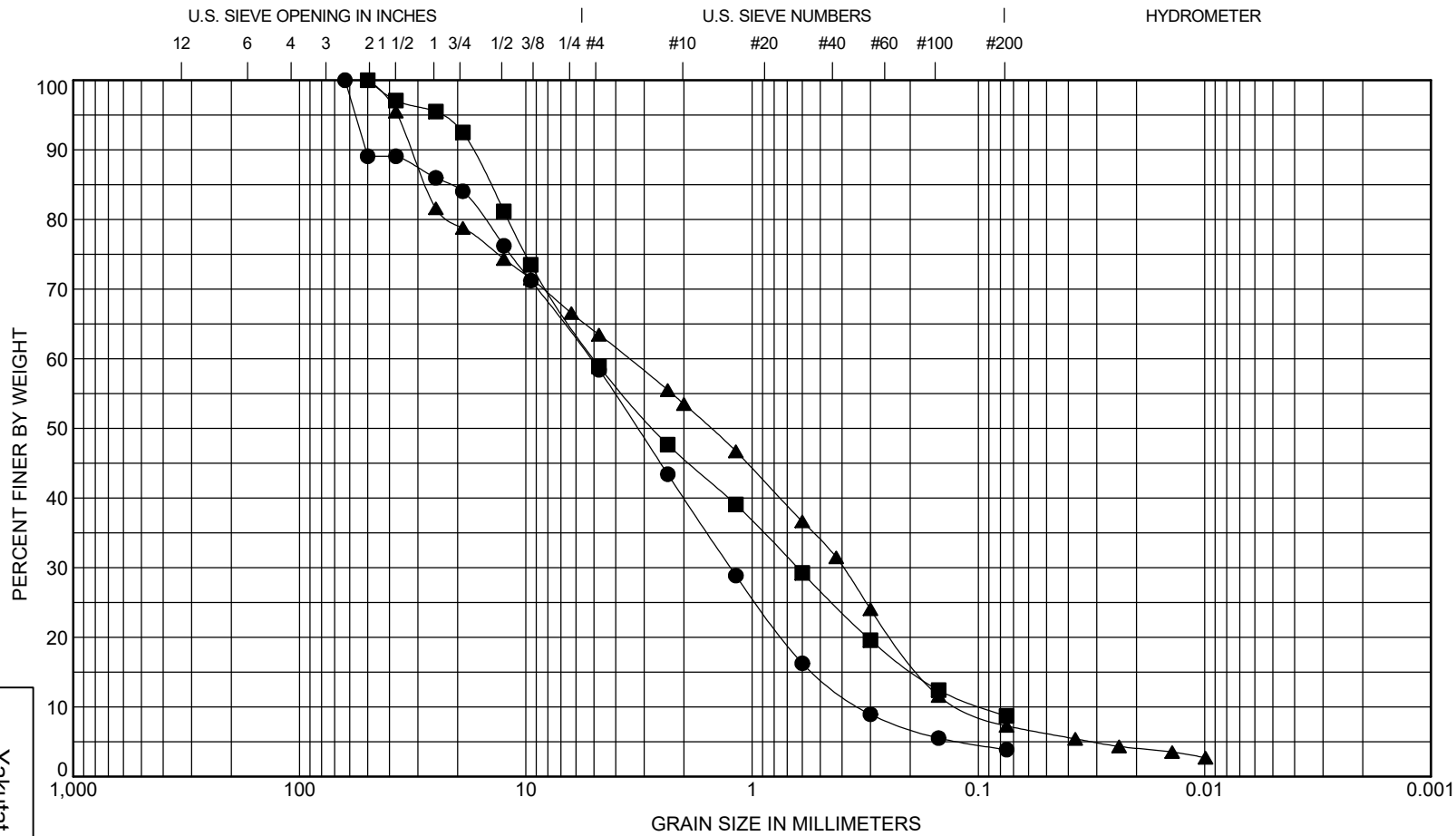
COBBLES			GRAVEL		SAND			SILT OR CLAY					
			coarse	fine	coarse	medium	fine						
Sample	Depth, Ft	USCS Classification						LL	PL	PI	Cc	Cu	
●	B-25 S3	5.0 - 7.0	Poorly Graded Sand with Gravel (SP)									0.7	15.7
■	B-26 S2	2.5 - 4.5	Well-Graded Gravel with Silt and Sand (GW-GM)									1.5	76.6
▲	B-26 S6	15.0 - 17.0	Poorly Graded Sand with Gravel (SP)									0.7	6.4
Sample	Depth, Ft	D100	D60	D30	D10	%Gravel	%Sand	%Silt		%Clay			
●	B-25 S3	5.0 - 7.0	37.5	3.84	0.83	0.25	35	62	3				
■	B-26 S2	2.5 - 4.5	37.5	6.15	0.85	0.08	45	45	10				
▲	B-26 S6	15.0 - 17.0	25	1.6	0.53	0.25	21	77	2				

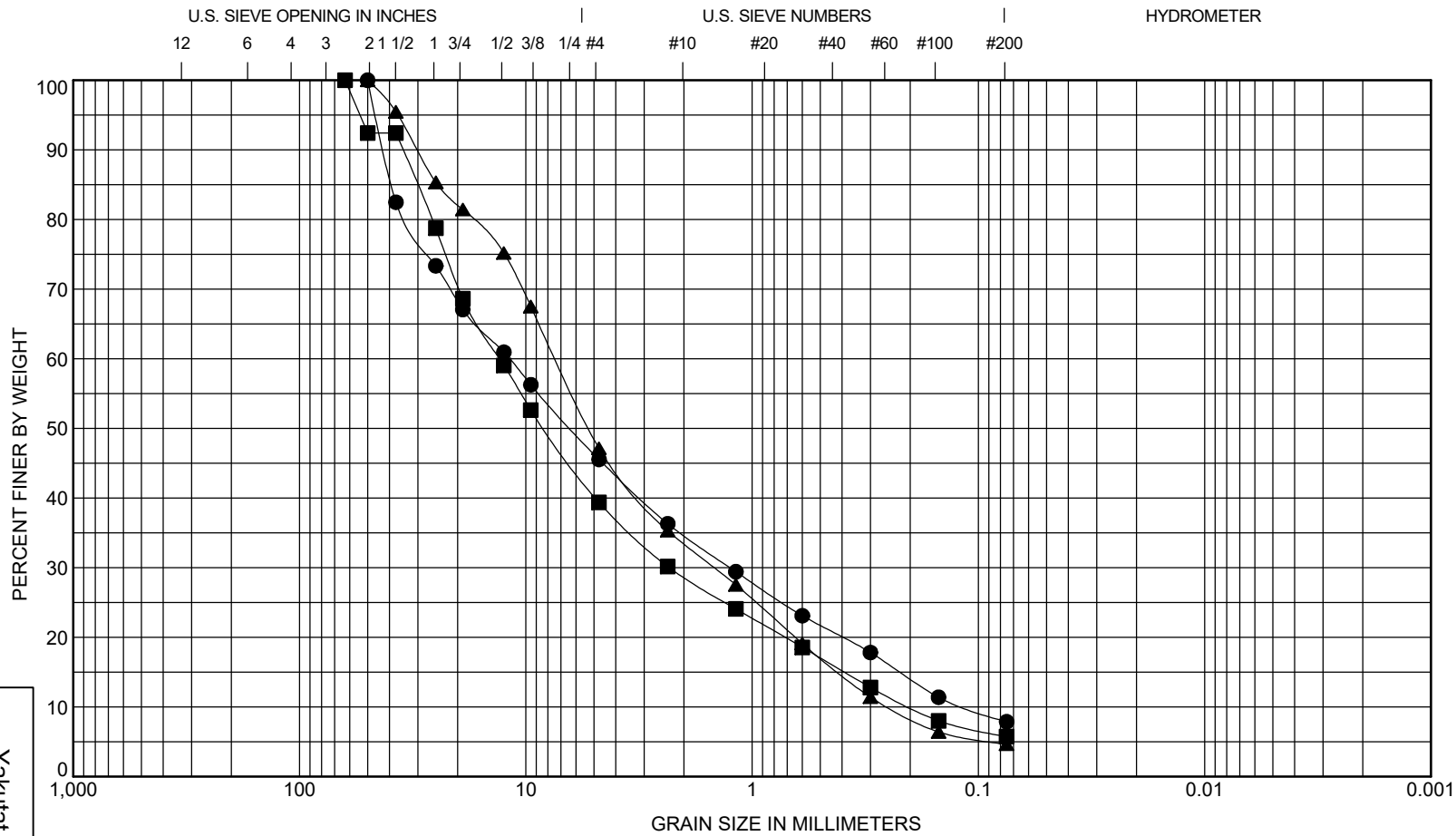
Yakutat Airport Water Line Extension
Yakutat, Alaska

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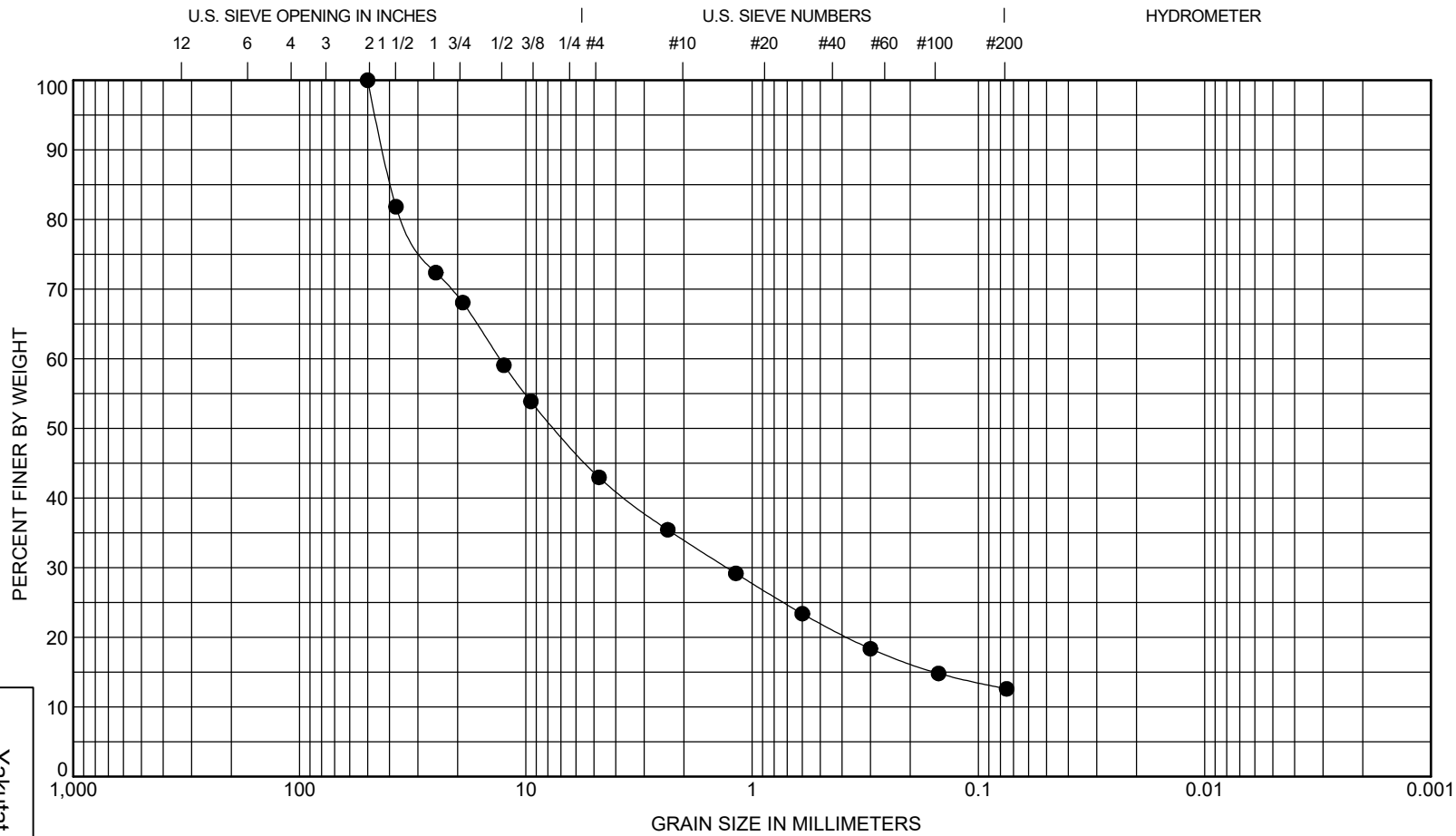




COBBLES		GRAVEL		SAND			SILT OR CLAY				
		coarse	fine	coarse	medium	fine					
Sample	Depth, Ft	USCS Classification					LL	PL	PI	Cc	Cu
● B-29 S3	5.0 - 7.0	Well-Graded Gravel with Silt and Sand (GW-GM)								1.2	103.7
■ B-29 S5	10.0 - 12.0	Well-Graded Gravel with Silt and Sand (GW-GM)								2.1	65.2
▲ B-29 S8	25.0 - 27.0	Well-Graded Gravel with Sand (GW)								1.2	29.9
Sample	Depth, Ft	D100	D60	D30	D10	%Gravel	%Sand	%Silt		%Clay	
● B-29 S3	5.0 - 7.0	50	11.83	1.25	0.11	54	38	8			
■ B-29 S5	10.0 - 12.0	63	13.04	2.32	0.2	61	34	6			
▲ B-29 S8	25.0 - 27.0	50	7.36	1.47	0.25	53	42	5			

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Yakutat, Alaska

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COBBLES		GRAVEL		SAND			SILT OR CLAY				
		coarse	fine	coarse	medium	fine					
Sample	Depth, Ft	USCS Classification					LL	PL	PI	Cc	Cu
● Perc S2	1.5 - 3.0	Silty Gravel with Sand (GM)									
Sample	Depth, Ft	D100	D60	D30	D10	%Gravel	%Sand	%Silt		%Clay	
● Perc S2	1.5 - 3.0	50	13.05	1.29		57	30	13			

Yakutat Airport Water Line Extension
Yakutat, Alaska

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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 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.

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

IMPORTANT INFORMATION

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.

The preceding paragraphs are based on information provided by the ASFE/Association of Engineering Firms Practicing in the Geosciences, Silver Spring, Maryland