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APPENDIX F:

PRELIMINARY GEOTECHNICAL SITE
ASSESSMENT

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Preliminary Geotechnical Site Assessment
Port of Longview
Barlow Point Terminal
Development
Longview, Washington

Prepared for
KPF Consulting Engineers

November 16, 2015
19132-00

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Prepared by
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1 Vicinity Map
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Field Explorations and Laboratory Results

ATTACHMENT B

Cone Penetrometer Test (CPT) Sounding Procedures

ATTACHMENT C

Explorations by Others

Port of Longview

Barlow Point Terminal Development

Longview, Washington

1.0 INTRODUCTION

Hart Crowser, Inc. is pleased to submit this report summarizing our preliminary geotechnical assessment of the proposed terminal development at Barlow Point for the Port of Longview (Port).

In 2010 the Port of Longview (Port) purchased the 282.5-acre property at Barlow Point for future Port industrial development. The property is located downstream of the current developed Port at approximately river mile 64 (RM 64), which is on the west side of the City of Longview (City). In order to better understand the full potential of the Barlow Point site, the Port determined that a comprehensive master planning process should occur. The first step in that process was to perform a due diligence study to assess the development feasibility of Barlow Point into a marine terminal, along with permit, timing requirements, and an order of magnitude of costs.

As part of the due-diligence study, Hart Crowser has completed a preliminary assessment of geotechnical conditions at the site to identify key geotechnical considerations and constraints to development.

We have organized this report into several sections. The first several pages provide an overview of the project information discussed in the text. The main body of the report presents our limited geotechnical engineering findings and recommendations. Figures are presented at the end of the text—the location of the site is shown on Figure 1, and the site layout showing the locations of our explorations is shown on Figure 2. Attachment A contains test pit logs and laboratory test results; Attachment B contains logs of our cone penetration test (CPT) probes and a description of the CPT equipment; and Attachment C contains historical explorations and laboratory test results, completed by others, from the project site.

2.0 SCOPE OF SERVICES

We completed the following scope of services as part of our preliminary geotechnical evaluation of the site.

- Reviewed existing published geotechnical information and geologic maps that covered the site and vicinity.
- Advanced four CPT probes to depths ranging between 51 and 152 feet below ground surface (bgs).
- Completed 18 test pits throughout the site to depths ranging between 6 and 12 feet bgs.

- Completed limited engineering analysis for seismic design, including liquefaction hazard analysis, riverfront seismic stability (including preliminary dredge configurations), and areal settlement imposed by potential mass filling of the site.
- Attended meetings at the Port and consulted with the design team regarding our findings.
- Prepared this report summarizing our findings, conclusions, and preliminary recommendations.

3.0 SITE CONDITIONS

3.1 Surface Conditions

The project area consists of an approximately 282.5-acre property located approximately 4 miles west of downtown Longview, as shown on Figure 1. The site is bound by Mt. Solo Landfill, Mt. Solo Road, and undeveloped fields on the north; former Reynolds Metals site on the east; and the Columbia River on the south and west. The site has historically been and is currently being used for agricultural purposes.

The site is a primarily a green field used for hay production, though has occasional small shrubs and a few stands of mature trees. A 900-foot-wide high voltage electrical transmission line easement, maintained by Bonneville Power Administration (BPA), is present on the east side of the property. A set of four high voltage transmission towers are present in the easement at the southeast corner of the site, one of which lies outside of the Barlow Point property. A drainage ditch and pump station, maintained and operated by the Consolidated Diking Improvement District No. 1 (CDID), are present at the east border of the site and the southeast corner of the site, respectively. A flood control dike, overseen by the CDID, is present at the riverfront and runs the entire length of the site.

Ground surface elevations are referenced to the NAVD 88 vertical datum for this project. The ground surface in the backland area of the site is generally level to gently rolling with an approximate average ground surface at elevation 10 feet. A drainage ditch with side slopes lying at approximately 4 horizontal to 1 vertical (4H:1V) is present between the site and the Mt. Solo Landfill. The flood control dike-side slopes lie at up to 4H:1V on the upland side of the dike and up to 6H:1V on the river side, with the dike crest at approximate average elevation 30 feet. Submerged slopes on the river side of the dike lie up to 1H:1V, but are typically shallower. The Columbia River shipping channel is adjacent to the site and is routinely dredged by the Army Corps of Engineers. Bathymetry provided to Hart Crowser indicates the bottom of the channel is at approximate elevation -45 to -65 feet at the time of bathymetry measurements.

3.2 Geologic and Soil Mapping

The geology of the site was mapped by the then named Washington Division of Geology and Earth Resources *Geological Map of the Astoria and Ilwaco Quadrangles, Washington and Oregon* (Walsh 1987). The map indicates that the geology of the site is dominated by young silty and sandy alluvium of the Columbia River overlying gravelly Troutdale Formation at depth. Site explorations have encountered materials that are consistent with the geologic mapping. However, the Troutdale formation has not been encountered at the site and could be over 200 feet bgs.

The near-surface soils at the site are mapped by the U.S. Department of Agriculture (USDA) in the *Soil Survey of Cowlitz County, Washington* (Soil Survey) as found on the *Web Soil Survey* (USDA 2006) website. The report generated by the Soil Survey for the site indicates several soil subtypes are mapped at the site, including Arents (Dike fill), 0 to 5 percent slopes; Caples silty clay loam, 0 to 3 percent slopes; Pilchuck loamy fine sand, 0 to 8 percent slopes; Riverwash (Outwash deposits on the riverside of the dikes); Schneider-Rock outcrop complex, 15 to 65 percent slopes; and Snohomish silty clay loam, 0 to 1 percent. The Schneider-Rock outcrop complex represents a small outcrop of bedrock near Mt. Solo Road and will not be discussed further.

The site is effectively split into two areas displaying similar soil types: 1) the inland portion of the site, covering nearly 75 percent of the land area, comprised of the Caples and Snohomish soil units; and 2) the dike portion of the site comprised of the Arents, Pilchuck, and Riverwash soil units. In general, for site planning purposes, the soils found at the site should be classified mainly as silt and sand with areas of organic silt or peat present in the Snohomish soil unit.

Inland Portion - Soils are typical alluvial deposits that form near bodies of water such as the Columbia River. Drainage characteristics of the soils range from moderately well drained in the Caples soil unit to poorly drained in the Snohomish. Permeability of the soil could range from 0.06 to 0.6 inches/hour, which is poorly draining.

Dike Portion - Soils range from typical alluvial deposits in the Riverwash and Pilchuck soil units, to man-made fills for the flood control dike in the Arents soil unit. Drainage characteristics of the soils range from somewhat poorly drained in the Riverwash soil unit, moderately well drained in the Arents soil unit, to somewhat excessively drained in the Pilchuck soil unit. Permeability of the soil could range from 0.6 and 20 inches/hour in the Pilchuck and Arents soil unit. Permeability estimates for Riverwash is not mapped at the site.

3.3 Subsurface Conditions

3.3.1 Recent Explorations by Hart Crowser

We completed a program of subsurface explorations at the site by advancing 4 CPT probes, designated CPT-1 through CPT-4, to depths ranging from 52 to 152 feet bgs on April 16, 2015, and 18 test pits, designated TP-1 through TP-18, to depths ranging from 6 to 12 feet bgs on April 29 and 30, 2015. The CPT and test pit locations are shown on Figure 2. Attachment A presents our test pit logs and Attachment B presents our CPT logs.

The soils encountered in the CPT explorations primarily consisted of alluvial silt and sand units. The silt unit was typically nearer the surface with the sand beneath and interbedded within the silt unit. Thin layers of cemented ash were also encountered interbedded with the silt. Dense sands and very stiff silts are present at great depths.

The test pits showed that the near-surface soils consist of soft fine-grained soils and, in some cases, soft organic-rich soils.

The soils encountered are discussed in more detail in the following paragraphs.

3.3.2 Explorations by Others

We reviewed a geotechnical report (GRI 1993) prepared for preliminary design of a proposed steel mill at the site. As part of that work, two borings and two CPT probes were advanced to depths of 149 to 189 feet bgs.

Subsurface conditions described by in the report are generally consistent with those observed in our explorations. As part of their geotechnical investigation, GRI conducted a suite of laboratory tests on selected soil samples. Included in their testing were five consolidation tests, which we used to help evaluate settlement/consolidation potential of the site soils.

Copies of the relevant explorations logs and laboratory tests are included in Attachment C.

3.3.3 Soil

3.3.3.1 Topsoil/Till Zone

As previously noted, the site has been agricultural in use, as such we anticipate that a 12- to 18-inch zone of tilled topsoil will be present throughout the property. We observed surficial topsoil with a heavy root zone between 1 and 12 inches thick, with an average thickness of 3 to 6 inches.

3.3.3.2 Fill

Within the last several years the Port has completed limited grading to remove dirt roads to prepare the site for farming and reduce the site's use by off-road vehicles. This grading generally included overall leveling of the site via bull dozer with cuts and fills less than 1 to 2 feet thick. No imported material was used for this work. Because the earthwork consisted of moving on-site soils, fills are generally indistinguishable from the *in situ* native soil and till zone. However, other fills are present at the site, such as at test pit TP-16 that encountered 2.5 feet of gravel, cobble, and boulder fill.

The flood control dike along the river also is comprised of fill. Current Army Corps of Engineer regulations and the project schedule did not allow for advancement of subsurface explorations through the dike; however, it appears as though one of the borings from GRI (1993) was advanced through the dike in the southeast corner of the site. Based on that boring, the dike fill appears to consist of stiff to soft gray silt with organics and loose gray sand, similar to the native materials found in the test pits throughout the site. It is not apparent from GRI's log at what depth the dike fill terminates. Based on the review of documents provided by the CDID, we understand the dike fill material was placed as hydraulic fill from dredging of the river and was also likely borrowed from the adjacent inland areas. One of the documents provided by the CDID includes summaries of the dike soil stratigraphy, based on borings that were completed in the dike alignment (CDID 1952). Based on the soil summaries it appears the dike fill soils at Barlow point consist of varying thicknesses of silt, sand, and clay, but is more typically silt and sand.

3.3.3.3 Alluvium

All explorations completed at the site encountered and were terminated in the alluvium. The alluvium consists of silt, clay, and sand in varying percentages. All the explorations at the site indicate the thickness and stratigraphy of the alluvial constituents is highly variable. Fine-grained (silt and clay) materials were found in very soft to very stiff consistencies, though more typically were soft to medium stiff. Coarse-grained (sand) materials were found in very loose to dense relative densities, though more typically were loose to medium dense.

Fine-grained layers from 125 to over 150 feet thick were encountered at the ground surface in CPT-1, CPT-2, and GRI's CPT probes P-1 and P-2. However, sand layers up to 60 feet thick were encountered underneath relatively thin silt layers at the ground surface and overlying thick silt layers in GRI borings B-1 and B-2. A sand layer over 50 feet thick was encountered underlying a relatively thin silt/clay layer in CPT-3. Interbedded sand layers ranging from about 1 to 8 feet were encountered in the silt/clay layers.

Probes CPT-1 through CPT-4 encountered very soft silt/clay layers ranging from approximately 1 to 38 feet thick at depths ranging from 3 to 15 feet bgs. The CPT data indicate that these are possibly sensitive fine-grained soils. We interpret these data as an indication that organic rich/highly compressible soils are present throughout the site. Test pit excavations encountered peat soils in test pits TP-9, TP-10, and TP-18 at depths ranging from 4.5 to 7.5 feet bgs. The thickness of peat soils in test pit TP-10 is 3.5 feet. However, test pits TP-10 and TP-18 did not extend fully through the peat soils.

A thin cemented ash layer was encountered in CPT-1, CPT-2, CPT-3, and GRI's probes P-1 at depths ranging from approximately 27 to 67 feet bgs.

The deeper (greater than 100 feet) probes and borings typically terminated in stiff to very stiff silt or dense sands at great depths.

3.3.3.4 Limitations

The subsurface information used for this study represents conditions at discrete locations across the project site. Actual conditions in other areas could vary. The nature and extent of any variations in subsurface conditions may not become evident until construction begins. If significant variations are observed at that time, we may need to modify our conclusions and recommendations accordingly to reflect actual site conditions.

3.3.4 Groundwater

Surface water and groundwater at the site are controlled by a series of private and public drainage ditches that run through and around the site. The CDID maintains ditches along the eastern side of the site and along SR 432, and operates a pump station off the southeast corner of the site. Private ditches run along the base of Mt. Solo with shallow ditches also present in the site interior. Surface runoff and groundwater are collected in the private ditches where it discharges to the CDID ditch network.

At the time of our exploratory work, groundwater was encountered in all the test pit explorations at depths ranging from 2 to 8 feet bgs. Based on our analysis of the CPT data, we estimate the static groundwater level was approximately 2 to 3 feet bgs (average elevation +8 feet) at the CPT locations.

However, groundwater conditions may fluctuate with time due to the levels of the Columbia River and drainage, rainfall, temperature, and other factors and may approach or build up above the existing ground surface during periods of heavy or prolong rainfall or flooding events.

3.3.5 Infiltration Characteristics

As previously noted, the majority of the site soils are mapped by USDA (2006) as being poorly draining with permeability values ranging from 0.06 to 0.20 inches/hour. Our test pit explorations confirmed the presence throughout the site of surficial fine-grained silts and clays which are not likely to infiltrate surface water.

Near the toe of the riverside dike (but not including the dike itself), some localized soils are mapped by USDA (2006) as having permeability values ranging from 6 to 20 inches/hour. However, our explorations in these areas (i.e., TP-8, TP-9, CPT-1, and CPT-2) encountered only a thin layer (approximately 1-foot thick) of sandy soil that could have limited infiltration capacity. Below the sandy soil, fine-grained, relatively impermeable silt and clay soils were present to great depths.

Groundwater was often encountered within 2 feet of the ground surface and is expected to rise to or above the ground surface late in the winter or during periods of heavy rain. The presence of such shallow groundwater does not allow sufficient separation (e.g., 5 feet) between the base of future infiltration systems and the groundwater table.

4.0 SEISMIC AND GEOLOGIC HAZARD ANALYSIS

4.1 Seismic Shaking

We evaluated potential seismic shaking at the site using data obtained from the U.S. Seismic Design Maps (USGS 2014). The expected peak bedrock acceleration having a 2 percent probability of exceedance in 50 years (2,475-year return period) is 0.44g. This value represents the peak acceleration on bedrock beneath the site and does not account for ground motion amplification due to site-specific effects. The peak ground acceleration (PGA) is determined by applying a Site Class factor to the peak bedrock acceleration. Refer to *Section 4.2 - Ground Motion Amplification (Site Class)* for a discussion of ground motion amplification.

We obtained a deaggregation of the seismic sources contributing to the expected peak bedrock acceleration shown above from the National Seismic Hazard Mapping Project website (USGS 2013). Seismic sources contributing to this potential ground shaking included intraplate (e.g., Cascadia Subduction Zone), interplate, and crustal faults. The data indicated that the “modal source” for shaking at the site is a magnitude 9.0 quake epicentered approximately 39 miles from the site. The modal source generally signifies the earthquake with the highest contribution to the site earthquake hazard. In this instance, a rupture of the Cascadia Subduction Zone will control the seismic hazard to the site.

4.2 Ground Motion Amplification (Site Class)

Thick sequences of unconsolidated, soft sediments typically amplify the shaking of long-period ground motions, such as those associated with subduction zone earthquakes; whereas, areas underlain by shallow soil profiles are not likely to amplify seismic waves.

The “Site Class” is a designation used by the 2012 International Building Code (IBC) (ICC 2012) to quantify ground motion amplification. The classification is based on the stiffness in the upper 100 feet of soil and bedrock materials at a site. At this site, the upper 100 feet of soil is generally loose or soft, with an average shear wave velocity of approximately 383 feet per second. This information, without regard for liquefaction potential (see below), leads us to classify the site as Site Class E.

Our analyses have identified a liquefaction hazard present at the site. The American Society of Civil Engineers (ASCE) 2010 Minimum Design Loads for Buildings and Other Structures (ASCE 7-10) (ASCE/SEI 2010) indicates that sites where a liquefaction hazard is identified should be represented as site class F and a site-specific ground response analysis be completed to determine the response spectrum for design. However, in accordance with ASCE 7-10, Site Class F soils vulnerable to potential failure or collapse under seismic loading, such as liquefiable soils, may be classified without regard for liquefaction, provided the structure under design has a fundamental period of vibration equal or less than 0.5 second. When a plan for further development to the site is prepared, it may be necessary to complete site-specific ground response analysis to better assess the effect of liquefaction on earthquake shaking on planned structures at the site.

It should be noted that we anticipate the seismic design of piers or wharves at the site will be governed by the ASCE/COPRI 61-14 Seismic Design of Piers and Wharves (ASCE/COPRI 61-14) or by the California State Building Code Chapter 31, Marine Oil Terminal Engineering and Maintenance Standards (MOTEMS). Both of these codes require a site-specific ground response analysis be completed for piers and wharves built in soils that are susceptible to liquefaction. A site-specific ground response analysis should be anticipated for the pier/wharf design phase of the project.

4.3 Seismic Design Parameters

Preliminary structural seismic design parameters are provided below in accordance with the 2012 IBC (ICC 2012). We anticipate seismic design parameters may change as the site development changes, and as more detailed analysis in accordance with ASCE/COPRI 61-14 or MOTEMS is completed. The parameters provided in Table 1 are appropriate for IBC code-level seismic design.

Table 1 – Seismic Design Parameters

Parameter	Value
Site Class	E (see discussion above)
Spectral Response Acceleration, S_S	0.976
Spectral Response Acceleration, S_1	0.457
Site Coefficient, F_a	0.929
Site Coefficient, F_v	2.400
Spectral Response Acceleration (Short Period), S_{DS}	0.604
Spectral Response Acceleration (1-Second Period), S_{D1}	0.732
Unfactored Peak Ground Acceleration, PGA	0.436
Site Coefficient, F_{PGA}	0.900
Maximum Considered Earthquake Geometric Mean PGA, PGA_M	0.392

4.4 Liquefaction Hazards

4.4.1 General

When cyclic loading occurs during an earthquake, the shaking can increase the pore pressure in loose to medium dense saturated sands and cause liquefaction. The rapid increase in pore water pressure reduces the effective normal stress between soil particles, resulting in the sudden loss of shear strength in the soil. Granular soils, which rely on interparticle friction for strength, are susceptible to liquefaction until the excess pore pressures can dissipate. Sand boils and flows observed at the ground surface after an earthquake are the result of excess pore pressures dissipating upwards, carrying soil particles with the draining water. In general, loose, saturated sand soils with low silt and clay contents are the most susceptible to liquefaction. Silty soils with low plasticity are moderately susceptible to liquefaction under relatively higher levels of ground shaking. For any soil type, the soil must be saturated for liquefaction to occur.

We performed site-specific liquefaction potential analysis on the sand soils underlying the site using procedures outlined in Youd, et al (2001). The analysis was conducted using data from our exploration CPT-3. In accordance with ASCE Minimum Design Loads for Buildings and Other Structures (ASCE 7-10), we completed the liquefaction hazard analysis using the site class adjusted Maximum Considered Earthquake Geometric Mean PGA (PGA_M). We used a PGA_M of 0.39 g and associated earthquake magnitude of 9.0 in our analysis.

The liquefaction potential of the silt soils underlying the site was evaluated based on the recommendations of Idriss and Boulanger (2008) and Robertson and Wride (1998). The Robertson and Wride reference provides a screening criteria, based on soil classification index, typically denoted I_c , which is evaluated from the CPT data obtained from a site. Robertson and Wride indicate that soils exhibiting an I_c greater than 2.6 can be treated as clay-like. Based on our evaluations, I_c values in the silt layer underlying the site typically vary from 2.8 to 3.4; therefore, we have treated the silt soils as clay-like. The methodology provided by the Idriss and Boulanger reference provides a liquefaction potential evaluation procedure similar to those of Youd, et al. However, a clay-like soil's ability to resist strength loss in the presence of earthquake shaking is dependent on the soil's *in situ* undrained shear strength.

Based on our analysis, it appears that liquefaction is likely to occur in the submerged sand soils interbedded within the silty soils found encountered at the site and encountered near the ground surface in CPT-3 during a design-level earthquake. Liquefaction of sand soils typically means strength loss and post-liquefaction settlement. Based on our analysis, it appears that liquefaction, typically termed “cyclic softening” for clay-like soils, is likely to occur in the submerged silt soils encountered throughout the site. Cyclic softening of silt soils typically means strength loss; however, unlike liquefied sand soils, cyclic softened clay-like soils do not typically manifest as large a magnitude of settlement as sandy soils.

4.4.2 Settlement

Post-liquefaction settlement results from densification of liquefiable sandy soils and remolding of cyclically softened clay-like soils following an earthquake. The permanent ground surface settlement is not typically uniform across the area and can result in significant differential settlement. Differential settlement will have the most significant effect on structures supported by shallow foundations.

Based on our analysis, we estimate up to approximately 1 foot of liquefaction-induced settlement could occur in the upper 50 feet of the soil column (primarily in sandy soils), with a theoretical additional foot of settlement below 50 feet. However, we consider these estimates to be conservative. It has not been well demonstrated that theoretical settlements at great depth (e.g., greater than 50 feet) will actually translate to the ground surface. Lastly, because the settlement will manifest as an area-wide ground deformation, differential settlement is expected to be about half of the total settlement. Therefore, we anticipate that the total ground settlement at the site will be on the order of 1 foot and that differential settlement will be on the order of 1/2 foot. Differential settlement is typically assumed to occur over a 50-foot distance.

Geotechnical seismic design is well established for estimating the post liquefaction settlement in sandy soils. However, no consensus has been reached for generally estimating the post cyclic softening settlement in clay-like soils. To better evaluate the post cyclic softened settlement potential in the site silty soils it may be necessary to conduct specialty seismic testing on select soil samples as development of the site continues.

4.4.3 Lateral Spreading and Flow Failure

Lateral spreading occurs when large blocks of ground are displaced down gentle slopes or towards the free face of river channels as a result of earthquake-induced inertial forces acting on the soil mass. Initiation of lateral spreading is often made worse when the soils within and beneath the soil mass liquefy or soften as a result of the shaking. Lateral spreading deformations can be experienced relatively far from a free face. Similar to lateral spread, flow failures result when large volumes of soil near the free face of river channels or lake bottoms displace vertically and laterally during or after earthquakes. As the ground begins to shake and the shearing resistance of liquefied soils decreases, ground displacement occurs in response to mainly static shear forces present within the soil mass and to a lesser extent earthquake-induced inertial forces. Flow failures typically manifest larger deformations than lateral spreading; however, the extent of the deformations is typically localized to the area behind the free face of the channel. Both lateral spreading and flow failures occurring within zones of deep or shallow foundations are destructive and pose a significant risk to the structures the foundations support.

We completed a preliminary river slope stability analysis to assess if flow failure should be anticipated at the site. Our analysis was completed using the program Slope/W by Geo-Slope International, Ltd. The Slope/W program performs two-dimensional limit equilibrium analysis to compute slope stability and determine a factor of safety against global failure. The factor of safety against global failure is simplistically defined as the ratio of the forces resisting slope movement (e.g., soil strength, soil mass, etc.) to the forces driving slope movement (e.g., gravity, earth pressure, earthquake shaking). The program predicts the location and geometry of “critical failure planes.” Critical failure planes are the zones with the lowest factors of safety. The results of our analysis show that earthquake-induced flow failures should be anticipated near the river’s edge as far back as the landward toe of the flood control dike.

We completed a preliminary lateral spreading analysis using the procedures of Youd, et al (2001). The results of the analysis indicate the entire site could be subject to lateral spreading deformations. However, the Youd, et al analysis is based on empirical relationships from a database of sites underlain by mainly sandy soil conditions where lateral spreading was observed post-earthquake, and does not consider site-specific effects such as complex soil stratigraphy or the presence of fine-grained soil units, such as those found at Barlow Point. Therefore, the presence of a lateral spreading hazard should be considered a conservative estimate for preliminary site development purposes. It should be acknowledged that the actual location and extent of flow failures and lateral spreading may change as the site development plan changes and more detailed site explorations and analyses are completed.

4.4.4 Seismic Strength Loss

As previously stated, the site is underlain by loose deposits of sand and relatively soft deposits of silt that we anticipate will undergo liquefaction or cyclic softening, respectively, during a design earthquake. In addition to the seismic hazards indicated above, our analyses indicate the soils at the site will weaken and exhibit significantly reduced shear strength during and after a design level earthquake. This will pose a risk to structures built at the site as the bearing capacity of both deep and shallow foundations will be significantly reduced during a design level earthquake. Structures will need to be designed with this in mind. Further preliminary recommendations regarding selection of foundation type are included in *Section 5.2 - Foundations* of this report.

4.5 Ground Fault Rupture

Based on our review of available geologic maps (Personius 2002), the closest known fault is the Portland Hills Fault approximately 37 miles southeast of the site. Therefore, we anticipate the hazard from ground fault rupture to be low, unless occurring on an unmapped or unknown fault underlying the site.

4.6 Other Geologic Hazards

Review of the Washington State Department of Natural Resources (WSDNR) Washington State Geologic Information Portal (WSDNR 2015) indicates no other geologic hazards are present at the site. This includes tsunami hazard inundation effects, lahar or pyroclastic flow hazards, and landslide hazards. However, it is of note that a large, deep seated active landslide is present on the south facing

slope of Mt. Solo immediately north of Mt. Solo Road. Based on our review of the Washington Division of Geology and Earth Resources Report of Investigation 35, Digital Landslide Inventory for the Cowlitz County Urban Corridor, Washington (Wegmann 2006), the landslide does not appear to pose a direct hazard to the site. However, the landslide is moving slowly downslope. If the landslide becomes unstable, possibly during a design earthquake, a catastrophic landslide could impact the northeast corner of the site.

5.0 CONCLUSIONS

The following sections of the report presents our conclusions and planning level recommendations for the geotechnical aspects the project. We have developed our recommendations based on our current understanding of the project and the subsurface conditions encountered by our and previous site explorations. If the nature or location of the development is different than we have assumed, we should be notified so we can change or confirm our recommendations.

Based on our work to date, it is apparent the site is underlain by thick deposits of alluvial soils that are weak and compressible. Groundwater is found at shallow depths and will likely reach the ground surface occasionally. The alluvium is heterogeneous, though typical consists of interbedded loose to medium dense sands and soft to stiff silts. There are also localized zones of soft organic-rich soils near the ground surface. Relatively dense sands and stiff silts are not encountered until depths of 125 feet to over 150 feet bgs. Due to their weak nature, the upper soils are subject to seismically induced liquefaction, strength loss, and settlement.

The presence of these loose and soft alluvial soils will have significant impact on the geotechnical development of the site.

- The ground is compressible and will settle when loaded by the placement of fills and the construction of buildings. The use of surcharge fills, deep foundations, and/or ground improvements will be required to alleviate or accommodate the ground settlement.
- The soils are weak and heavily loaded structures will not be able to be supported on conventional spread footing foundation systems. The use of deep foundations (e.g., driven piles) and/or ground improvement measures (e.g., soil-cement mixing, stone columns, etc.) will be required to reduce settlement and carry structural loads.
- The river bank slopes are marginally stable due to the weak soils. Dredging of the river bank should be limited or significant in-water slope stabilization measures will likely be required.
- The seismic vulnerability of the soils will result in widespread instability of the river bank in the event of a strong earthquake. Significant ground improvement measures (e.g., deep soil mixing, driven shear piles, etc.) will likely be required to stabilize the river bank and in-water structures. Permitting issues related to in-water work may substantially guide the chosen option for ground improvements.

Due the presence of shallow groundwater and fine-grained soils near the ground surface, the ability to use infiltration facilities to discharge stormwater will be limited.

5.1 Settlement

As described above, the site is mantled with soft fine-grained and, in some cases, organic-rich soils that are susceptible to “static” ground settlement when loaded. Additionally, the site soils are susceptible to liquefaction-induced settlement during and after a design level earthquake. Settlement effects will be an important consideration for any improvements planned for the site.

5.1.1 Static Settlement Imposed by Areal Fills

We anticipate large scale filling may be an option to raise site grades to facilitate drainage. We have conducted preliminary analyses to evaluate the anticipated magnitude of settlement due to a large areal fill. The results of four areal fill placement scenarios are provided in Table 2.

Table 2 – Areal Fill Static Ground Settlement Estimates

Thickness of Fill (feet)	Estimated Settlement (feet)
2	0 to 0.5
4	0.5 to 1
8	1 to 2
12	3 to 5

Note: The settlement analyses assumed a unit weight of fill of 125 pound per cubic foot (pcf).

At this time, the rate of settlement has not been determined, but for planning purposes we recommend that it be assumed that settlement could take several years to complete if no special measures are taken to increase the rate of settlement. Such measures typically include the use of surcharge fills and/or wick drains. Surcharge fill is temporary fill that exceed the final fill and/or building loads to be applied to an area that are used to “pre-consolidate” and/or speed the rate of consolidation of soft soils. Wick drains consist of either sand or prefabricated elements installed vertically through or into compressible soils to create shortened drainage paths to reduce excess pore water pressures in soft soils that have been loaded, and therefore, accelerating the rate of primary consolidation. Wick drains are typically installed in a grid pattern throughout the area of interest.

5.1.2 Static Settlement Imposed by Dried Bulk Storage

The bulk storage of dried goods will induce ground settlement in addition to that caused by any areal fills. The unit weight of bulk dried goods is typically much smaller than compacted soil fill, though the height of the stored material can be great. For preliminary analyses we evaluated a 200-foot by 500-foot storage area with a stored height of 40 feet. We considered unit weights of approximately 25 pcf for wood pellets, 50 pcf for urea, and 75 pcf for potash, which correspond to equivalent surcharges for 1000, 2000, and 3000 pounds per square foot (psf), respectively.

The results of our dried bulk storage settlement analyses are provided in Table 3.

Table 3 – Dried Bulk Storage Static Ground Settlement Estimates

Material	Equivalent Surcharge (psf)	Estimated Settlement at Stored Pile (feet)
Wood Pellets	1,000	1 to 2
Urea	2,000	2 to 5
Potash	3,000	5 to 8

Due to the large magnitude of the settlements predicted, storage areas will likely require surcharging to reduce the potential for settlement induced by the stored materials to affect the proposed warehouse structures.

5.1.3 Static Settlement Imposed by Liquid Bulk Storage

The bulk storage of liquid products in tanks will also induce ground settlement in a similar manner to areal fills. The unit weight of most petroleum products ranges from 50 to 60 pcf. The magnitude of settlement will vary greatly, depending upon the diameter and height of the storage tanks and the layout of tank farms. Because of the great variability in factors, we have not completed an analysis of tank-induced settlement. For planning purposes, it should be assumed that surcharges, ground improvement, and/or deep foundations will likely be required to mitigate or accommodate ground settlement induced by tank loads.

5.1.4 Static Settlement Imposed by Footing and Slab Loads

We anticipate lightly loaded structures may be able to be supported by conventional spread footings. Such foundations and associated interior storage loads, which are typical of warehouse style structures employed at port sites, will load the underlying soils similarly to an areal fill. As foundation and floor loads are applied during and after construction, further static settlement will be experienced in and around the structures. We anticipate surcharging will be used throughout the site to reduce the magnitude of settlement imposed by footing and slab loads. See *Section 5.2 – Foundations* of this report for further discussion of surcharging.

5.1.5 Secondary/Creep Settlement and Surcharging

The site is mantled with soft and, in some places, organic-rich fine-grained soils. These soils can exhibit a large magnitude of secondary settlement or creep under areal fill, slab, and footing loads after completion of static settlement. If not properly accounted for during design, creep settlement can also cause overstressing and settlement of deep foundation systems installed through creep susceptible soils if overlying or adjacent improvements induce pressures into those soils.

Surcharge fills that induce pressures above the pressures induced by the areal fill loads, slab, and footing loads are typically employed to reduce the magnitude of post construction creep settlement. Surcharging in planned improvement areas should be anticipated throughout the site. Due to the presence of thick lenses of soft silt underlying the site, we anticipate the surcharging process could be prohibitively long as most construction must wait until consolidation of the underlying strata is complete. Wick drains may be needed to increase the speed of the surcharging process.

5.1.6 Seismic Settlement

As indicated above, the native sandy soils appear susceptible to liquefaction. Our exploration CPT-3 and GRI's exploration B-2 encountered loose sand lenses over 50 feet in thickness susceptible to liquefaction-induced settlement underlying the site. Based on our analysis, we estimate on the order of 1 foot and 1/2 foot of total and differential ground settlement, respectively, may occur as a result of liquefaction.

5.2 Foundations

Foundation systems will need to be chosen carefully due to the potential for large scale static and seismic ground settlements and seismically induced soil strength loss. Factors to consider when choosing a foundation system include structure type, structural loads, stored material loads, structural sensitivity to settlement, etc. Depending upon these considerations, a wide range of foundation systems may be used at the site, including spread footings, driven piles, stone columns, helical piles, etc. These systems and some considerations are discussed below.

5.2.1 Upland Structures

Lightly Loaded Structures – Lightly loaded structures, which are not movement sensitive, may be supported by a spread footing foundation system. However, in order to use such a foundation system, the building area may need to be surcharged and/or several feet of structural fill may be required. The building will also need to be able to withstand the liquefaction-induced ground settlements noted above. Using interlocking footings or grade beams is a common way to reduce the adverse impacted of differential settlement.

Moderately Loaded Structures – Moderately loaded structures, which are somewhat movement sensitive, may be supported by a spread footing foundation system that is underlain by ground reinforcement, such as aggregate piers. Such a system can oftentimes reduce static and seismic settlements to a level that a building can withstand.

Heavily Loaded Structures – Heavily loaded or movement sensitive structures will need to be supported by deep foundation systems (e.g., driven piles). We anticipate that this will be the most common foundation system used at the site. However, as previously noted, the site is underlain by thick units of weak silt and sand that are susceptible to seismically induced settlement and strength loss. Deep foundation systems less than 100 feet can be employed at the site to support static loads; however, these relatively shallow systems may experience significant settlement, or plunging, during a design level earthquake. Deep foundation systems over 150 feet and up to 200 feet long may be needed to prevent plunging during and after a design level earthquake.

5.2.2 In-Water Structures

In-water wharf and pier structures typically are required to withstand large vertical, lateral, and uplift structural loads. The use of deep, large diameter piles will be required to support these structures and loads. Furthermore, as discussed below, ground improvements will be required to stabilize the riverbank, so that failure of the bank does not damage the structures during an earthquake. We

anticipate that piles for in-water structures will consist of driven pipe piles that extend 150 to 200 feet below grade. Pile diameters of 24 inches or larger will likely be required.

5.3 Ground Improvement

Based on the potential for flow failure and lateral spreading hazards along the riverbank, we anticipate that a large scale program of ground improvement may be required to stabilize the riverbank.

Identifying and designing a ground improvement option for the site is beyond the scope of this report. However, due to the fine-grained soils encountered in subsurface investigations at the site, more conventional methods of ground improvement such as stone columns or vibrocompaction will be less effective than other techniques. Other techniques might consist of either deep soil mixing (DSM), driven compaction or “pinch” piles acting as slope reinforcement (i.e., “shear pins”), or both. DSM consists of blending cement with *in situ* soils with augers, creating a “soil-cement” column. Permitting issues may dictate the final ground improvement option chosen. It has been our experience that freshly mixed grout is not permitted in marine habitats; therefore, DSM installation will need to be performed in such a manner that will minimize the amount of grout that can enter the water column. Theoretically, this might require a coffer dam if placed below ordinary high water (OHW) level. Cofferdams may be prohibitively expensive to employ at the site due to the depth of mudline below OHW level. Conversely, driving piles over-water could require mitigation of pile driving hammer vibrations that carry through the water, adding increased cost to the pile option. Other ground improvement methods, such as stone columns or aggregate piers, may prove feasible, but might be cost inefficient. These types of ground improvement, such as stone columns, act to improve conditions by densifying the surrounding loose soils and also by providing a zone of high shear strength material. Because of the fine grained nature of the site soils, it will not be possible to gain substantial densification of the site soils. Therefore the technique will rely solely on the reinforcing effect of the angular stone that is inserted into the soil mass. This will result in the need for a greater than usual amount of stone. However, this approach would also reduce the environmental impacts associated with ground improvement.

For planning level estimation, we recommend assuming a 200- to 250-foot wide zone of ground improvement that extends up to 80 feet below finished grades. At a minimum, the treatment should extend laterally along the shoreline to “shadow” the envelope of new structures and at least 50 feet beyond. Additionally, ground improvement will be required wherever it is desired to stabilize the riverbank against flow failures or lateral spreading. Also, where significant dredging of the toe of the riverbank may occur, then ground improvement will be required. These conditions may require the ground improvement zone to extend between wharves/piers. We anticipate the treatment zone will extend into the dike right-of-way and/or below the OHW level, therefore; the impacts on permitting should also be considered.

The actual location, extent, and method of ground improvement will change as the site development plan changes and more detailed site explorations are undertaken.

5.4 Dredging

We completed preliminary slope stability analysis to evaluate the impact of dredging into the underwater riverbank on the stability of the flood control dike at the top of the riverbank. We understand the preliminary dimensions of the dredging include a cut to elevation -46 feet (Columbia River Datum [CRD]). The cut backslope will be graded at 3H:1V above elevation -46 feet (CRD). The results of our analysis show essentially no impact to the stability of the dike for dredging to elevation -46 feet outside a distance of 425 feet from the dike centerline. However, dredging and back cutting within the 425-foot distance decreases the stability of the riverbank and dike, and necessitates the need for ground improvement measures.

We note that the riverbank is only nominally stable in its current configuration and is unstable during seismic events (without ground improvements). The stability analysis completed was specifically geared toward evaluating the impact of dredging on the stability of the dike.

5.5 Earthwork Considerations

The majority of the surficial site soils are fine-grained and highly susceptible to moisture-related disturbance. Wet soil construction practices will be necessary throughout most of the year, particularly during periods of wet weather. Wet soil construction practices include using equipment, such as smooth excavator buckets and tracked equipment, to limit subgrade disturbance and using haul roads in areas of frequent heavy equipment traffic. The use of cement amendment to stabilize site soils and subgrades should be considered.

5.6 Stormwater Infiltration Considerations

Due the presence of relatively impermeable fine-grained soils and shallow groundwater, it is likely the use of stormwater infiltration facilities will be precluded at the site. We anticipate it will be necessary to collect, treat and detain stormwater runoff before discharging the water directly into the river or existing drainage ditches.

Theoretically, infiltration may be feasible if significant grading was completed to raise site elevations using permeable soils. However, even then study of groundwater mounding would be required to determine if infiltrated water would just perch and build up on the native fine-grained soils. We anticipate 10 or more feet of areal fill would be required to consider this alternative.

6.0 LIMITATIONS

We have prepared this report for the exclusive use of KPFF Consulting Engineers, the Port of Longview, and their authorized agents for the Barlow Point Terminal Development in Longview, Washington, in accordance with *Agreement for Subconsultant Services* (dated December 17, 2014) our November 11, 2014 memorandum detailing our scope and fee attached to the *Agreement for Subconsultant Services*. Our report is intended to provide our opinion of geotechnical parameters for preliminary design of the proposed project based on exploration locations that are believed to be representative of site conditions.

However, conditions can vary significantly between exploration locations and our conclusions should not be construed as a warranty or guarantee of subsurface conditions or future site performance.

Within the limitations of scope, schedule, and budget, our services have been executed in accordance with generally accepted practices in the field of geotechnical engineering in this area at the time this report was prepared. No warranty, express or implied, should be understood.

Any electronic form, facsimile, or hard copy of the original document (email, text, table, and/or figure), if provided, and any attachments are only a copy of the original document. The original document is stored by Hart Crowser and will serve as the official document of record.

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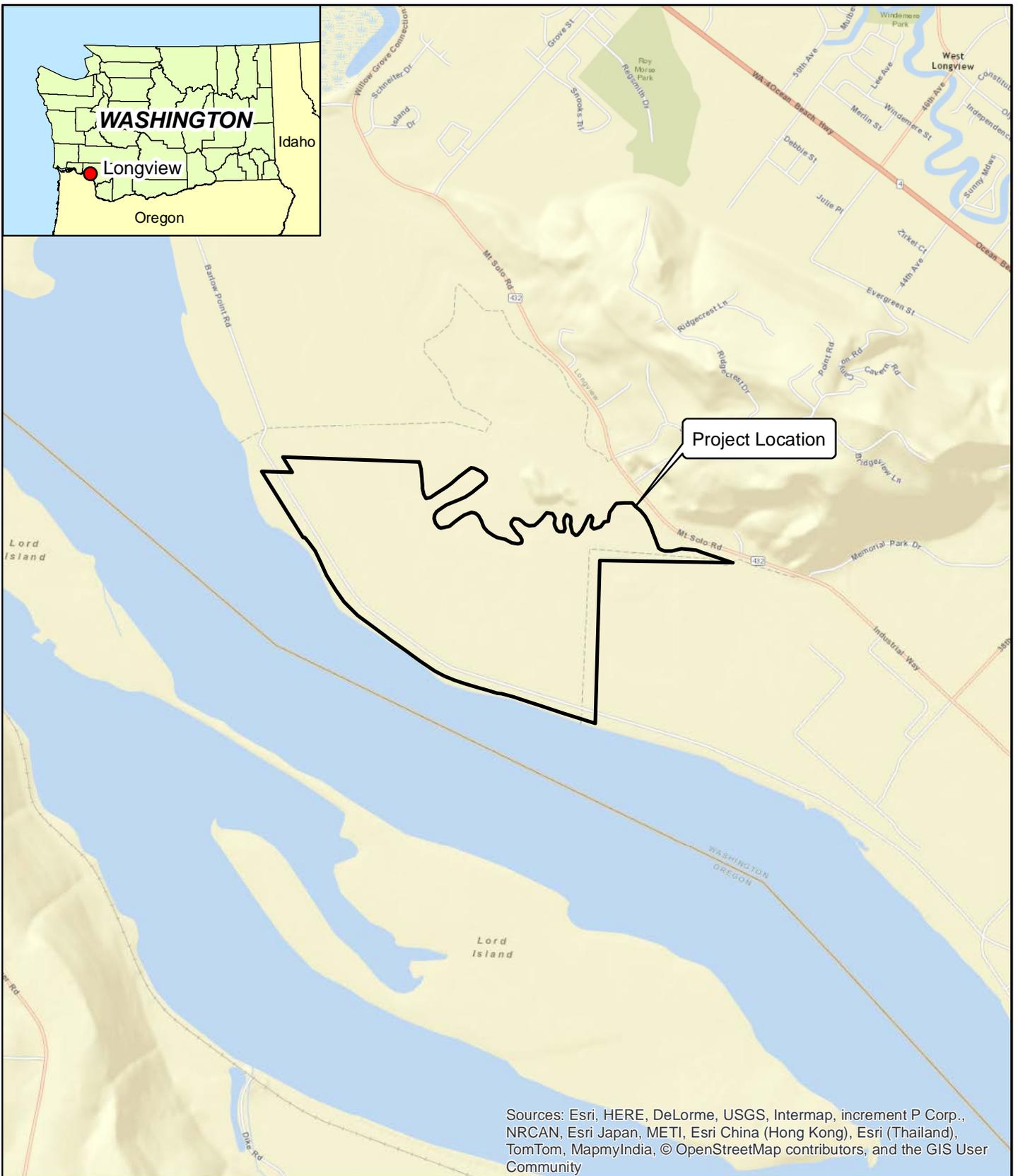
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Sources: Esri, HERE, DeLorme, USGS, Intermap, increment P Corp., NRCAN, Esri Japan, METI, Esri China (Hong Kong), Esri (Thailand), TomTom, MapmyIndia, © OpenStreetMap contributors, and the GIS User Community



Barlow Point Terminal Development
Longview, Washington

Vicinity Map

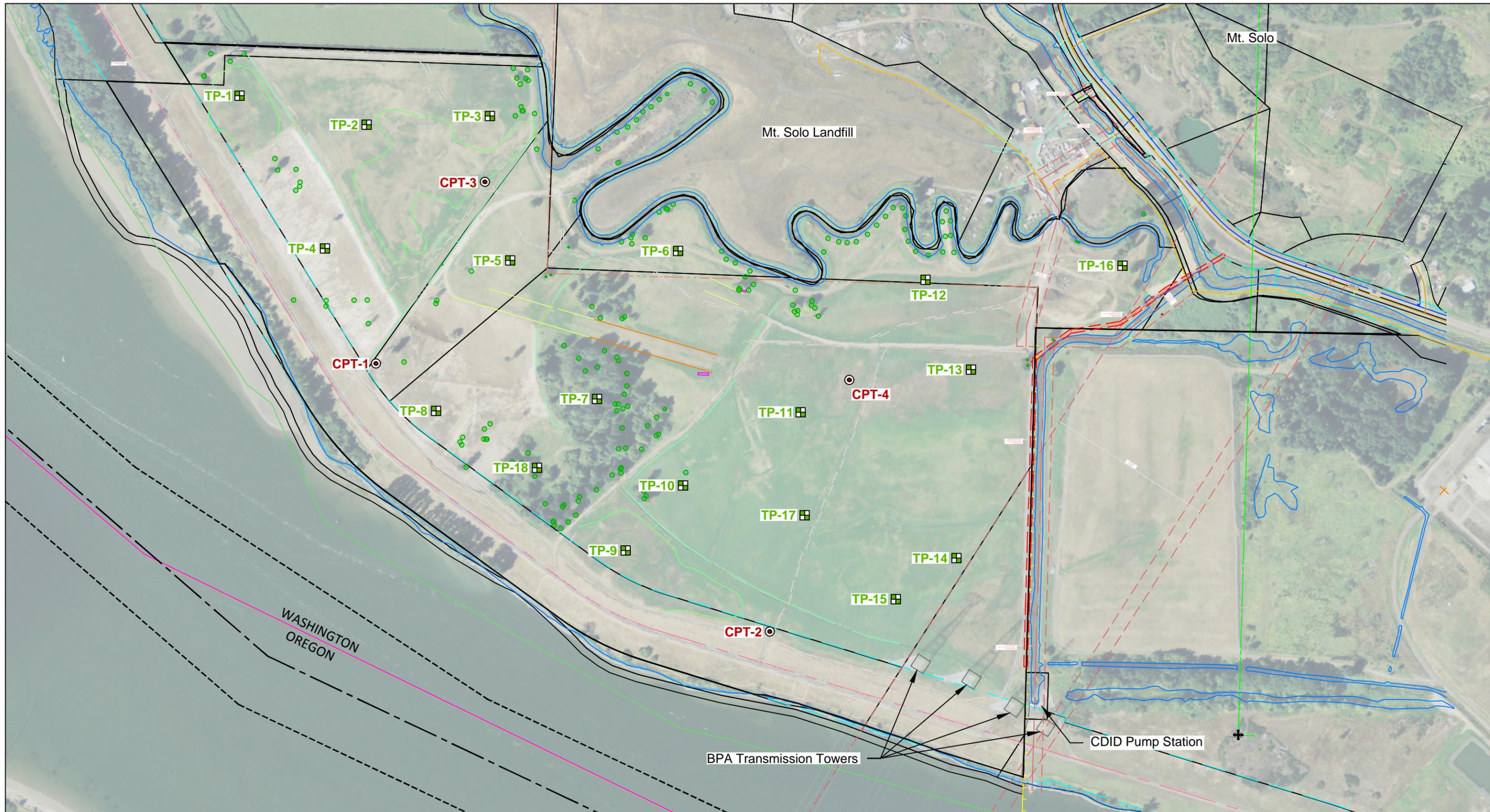
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11/15



Figure

1



LEGEND

- CPT-1** CPT Test
- TP-1** Test Pit

Note: Feature locations are approximate.

Sources: Base map prepared from AutoCAD files named ACAD-14195-X-Contours.dwg and 14195-SB-WSPC-S.dwg, provided by KPFF on April 6, 2015.



Barlow Point Terminal Development
Longview, Washington

Site Plan

19132-00

11/15



Figure

2

ATTACHMENT A
Field Explorations and Laboratory Results

ATTACHMENT A

Field Explorations and Laboratory Results

This attachment documents the processes Hart Crowser used to determine the nature (and quality) of the soil and groundwater underlying the project site addressed by this report.

Explorations and their Locations

The subsurface exploration for this supplemental included 18 test pits and 4 CPT probes. The logs of the test pits are included in this attachment, while the probe logs are in Attachment B.

The test pit exploration logs within this attachment shows our interpretation of the excavation, sampling, and laboratory test data. The logs indicate the depth where the soils change. Note that the change may be gradual. In the field, we classified the samples taken from the explorations according to the methods presented on Figure A-1, Key to Exploration Logs. This figure also provides a legend explaining the symbols and abbreviations used in the logs.

The locations of the explorations were determined using a hand-help GPS unit. The method used determines the accuracy of the location and elevation of the explorations.

Test Pit Excavation

Eighteen test pits, designated TP-1 through TP-18, were excavated to depths of 6 to 12 feet bgs on April 29 and 30, 2015. The test pits were excavated using a small-tracked excavator owned and operated by Dan J. Fischer Excavating Inc. of Forest Grove, Oregon. The excavations were continuously observed by engineering and geologic staff from Hart Crowser. Representative soil grab samples were obtained from the excavations, and detailed logs were prepared for each excavation. The test pit logs are presented on Figures A-2 through A-10 at the end of this attachment.

Soil Classification

Soil samples from the explorations were visually classified in the field and then taken to our laboratory where the classifications were verified in a relatively controlled laboratory environment. Field and laboratory observations include density/consistency, moisture condition, and grain size and plasticity estimates. The classifications of selected samples were checked by laboratory tests, such as moisture content determinations and grain size analyses. Classifications were made in general accordance with the Unified Soil Classification System (USCS) and ASTM International (ASTM) Test Method D 2487.

Moisture Content Testing

We tested the moisture content of selected soil samples in general accordance with ASTM D 2216. The moisture content is a ratio of the weight of the water to soil in a test sample and is expressed as a percentage. The moisture contents range from 27 to 195 percent, but are more typically between 27 and 82 percent, with an average value of approximately 53 percent. The high moisture contents (typically greater than approximately 40 or 50 percent) are generally indicative of a high organic content. The test results are indicated on the appropriate test pit log and on Table A-1 at the end of this attachment.

Fines Content Analyses

Fines content analyses were performed to determine the percentage of soils finer than the No. 200 sieve—the boundary between sand and silt size particles. The test was performed in general accordance with ASTM Test Method D 1140. The test results are indicated on the appropriate test pit log and on Table A-1 at the end of this attachment.

Organic Content

The organic content of four soil samples were determined in accordance with guidelines presented in ASTM D 2974. The moisture content of the samples were determined by drying the samples in a standard drying oven and expressed as a percentage of the sample weight. The organic content is determined by igniting the oven-dried sample in a muffle furnace. The resulting substance is ash, which is expressed as a percentage of the oven-dried sample. The results of the laboratory testing are displayed on Table A-1 at the end of this attachment.

KEY TO EXPLORATION LOGS



SOIL CLASSIFICATION CHART

MATERIAL TYPES	MAJOR DIVISIONS		GROUP SYMBOL	SOIL GROUP NAMES & LEGEND		OTHER MATERIAL SYMBOLS						
COARSE-GRAINED SOILS >50% RETAINED ON NO. 200 SIEVE	GRAVELS >50% OF COARSE FRACTION RETAINED ON NO 4. SIEVE	CLEAN GRAVELS <5% FINES	GW	WELL-GRADED GRAVEL		<table border="1"> <tr><td></td><td>Concrete</td></tr> <tr><td></td><td>Asphalt</td></tr> <tr><td></td><td>Topsoil</td></tr> </table>		Concrete		Asphalt		Topsoil
			Concrete									
			Asphalt									
			Topsoil									
	GRAVELS WITH FINES, >12% FINES	GP	POORLY-GRADED GRAVEL									
	SANDS >50% OF COARSE FRACTION PASSES ON NO 4. SIEVE	CLEAN SANDS <5% FINES	GM	SILTY GRAVEL								
		GRAVELS WITH FINES, >12% FINES	GC	CLAYEY GRAVEL								
	FINE-GRAINED SOILS >50% PASSES NO. 200 SIEVE	SANDS	CLEAN SANDS <5% FINES	SW	WELL-GRADED SAND							
POORLY-GRADED SAND			SP	POORLY-GRADED SAND								
SANDS AND FINES >12% FINES			SILTY SAND	SM	SILTY SAND							
			CLAYEY SAND	SC	CLAYEY SAND							
SILTS AND CLAYS LIQUID LIMIT <50		INORGANIC	LEAN CLAY	CL	LEAN CLAY							
			SILT	ML	SILT							
		ORGANIC	ORGANIC CLAY OR SILT	OL	ORGANIC CLAY OR SILT							
			SILTS AND CLAYS LIQUID LIMIT >50	INORGANIC	FAT CLAY	CH	FAT CLAY					
ELASTIC SILT	MH	ELASTIC SILT										
ORGANIC	ORGANIC CLAY OR SILT	OH		ORGANIC CLAY OR SILT								
	HIGHLY ORGANIC SOILS	PT		PEAT								

Note: Multiple symbols are used to indicate borderline or dual classifications

MOISTURE MODIFIERS

- Dry - Absence of moisture, dusty, dry to the touch
- Moist - Damp, but no visible water
- Wet - Visible free water or saturated, usually soil is obtained from below the water table

SEEPAGE MODIFIERS

- None -
- Slow - < 1 gpm
- Moderate - 1-3 gpm
- Heavy - > 3 gpm

CAVING MODIFIERS

- None -
- Minor - isolated
- Moderate - frequent
- Severe - general

MINOR CONSTITUENTS

- Trace - < 5% (silt/clay)
- Occasional - < 15% (sand/gravel)
- With - 5-15% (silt/clay) in sand or gravel
- 15-30% (sand/gravel) in silt or clay

SAMPLE TYPES

- Dames & Moore
- Standard Penetration Test (SPT)
- Shelby Tube
- Bulk or Grab

LABORATORY/ FIELD TESTS

- ATT - Atterberg Limits
- CP - Laboratory Compaction Test
- CA - Chemical Analysis (Corrosivity)
- CN - Consolidation
- DD - Dry Density
- DS - Direct Shear
- HA - Hydrometer Analysis
- OC - Organic Content
- PP - Pocket Penetrometer (TSF)
- P200 - Percent Passing No. 200 Sieve
- SA - Sieve Analysis
- SW - Swell Test
- TV - Torvane Shear
- UC - Unconfined Compression

GROUNDWATER SYMBOLS

- Water Level (at time of drilling)
- Water Level (at end of drilling)
- Water Level (after drilling)

STRATIGRAPHIC CONTACT

- Distinct contact between soil strata or geologic units
- Gradual or approximate change between soil strata or geologic units

Notes:

- Blowcount (N) is recorded for driven samplers as the number of blows required to advance sampler 12 inches (or distance noted) per ASTM D-1586. See exploration log for hammer weight and drop.
- When the Dames & Moore (D&M) sampler was driven with a 140-pound hammer (denoted on logs as D+M 140), the field blow counts (N-value) shown on the logs have been reduced by 50% to approximate SPT N-values.
- Soil density/consistency in borings is related primarily to the Standard Penetration Resistance. Soil density/consistency in test pits and probes is estimated based on visual observation and is presented parenthetically on the logs.
- Refer to the report text and exploration logs for a proper understanding of subsurface conditions. Descriptions on the logs apply only at the exploration locations at the time the explorations were made. The logs are not warranted to be representative of the subsurface conditions at other locations or times.

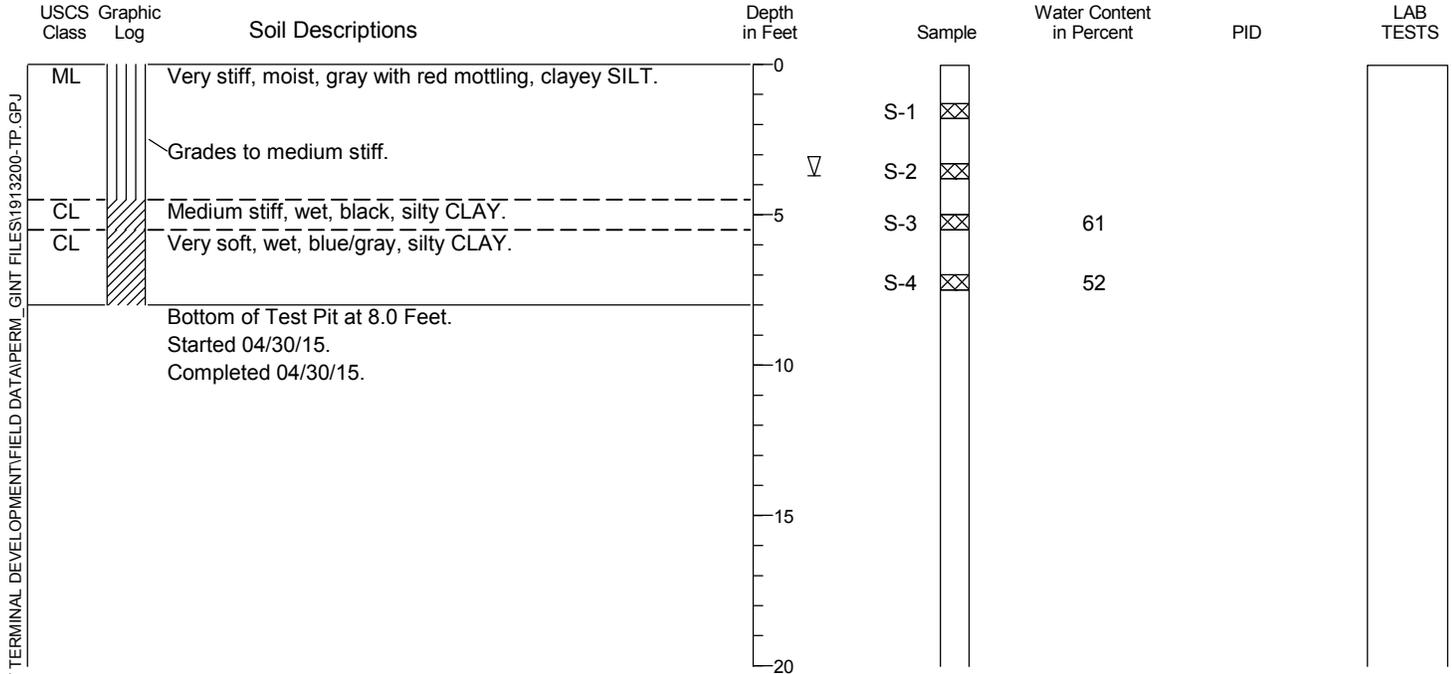
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Figure A-1

Test Pit Log TP- 1

Location: N 46.1506 E -123.0329
 Approximate Ground Surface Elevation: 9 Feet
 Logged By: J. Strobel/G. Wade Reviewed By: J. Alders

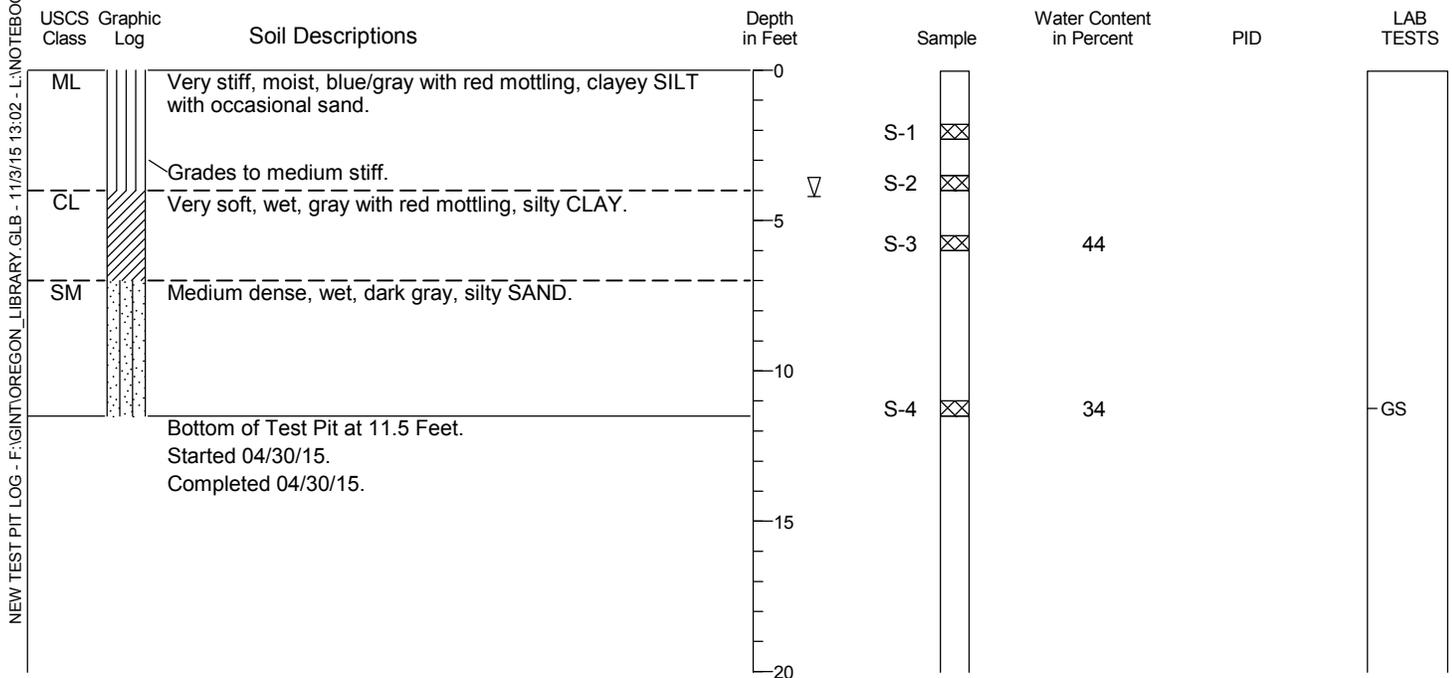
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 Vertical Datum: NAVD88



Test Pit Log TP- 2

Location: N 46.1503 E -123.0301
 Approximate Ground Surface Elevation: 7 Feet
 Logged By: J. Strobel/G. Wade Reviewed By: J. Alders

Horizontal Datum: WGS84
 Vertical Datum: NAVD88



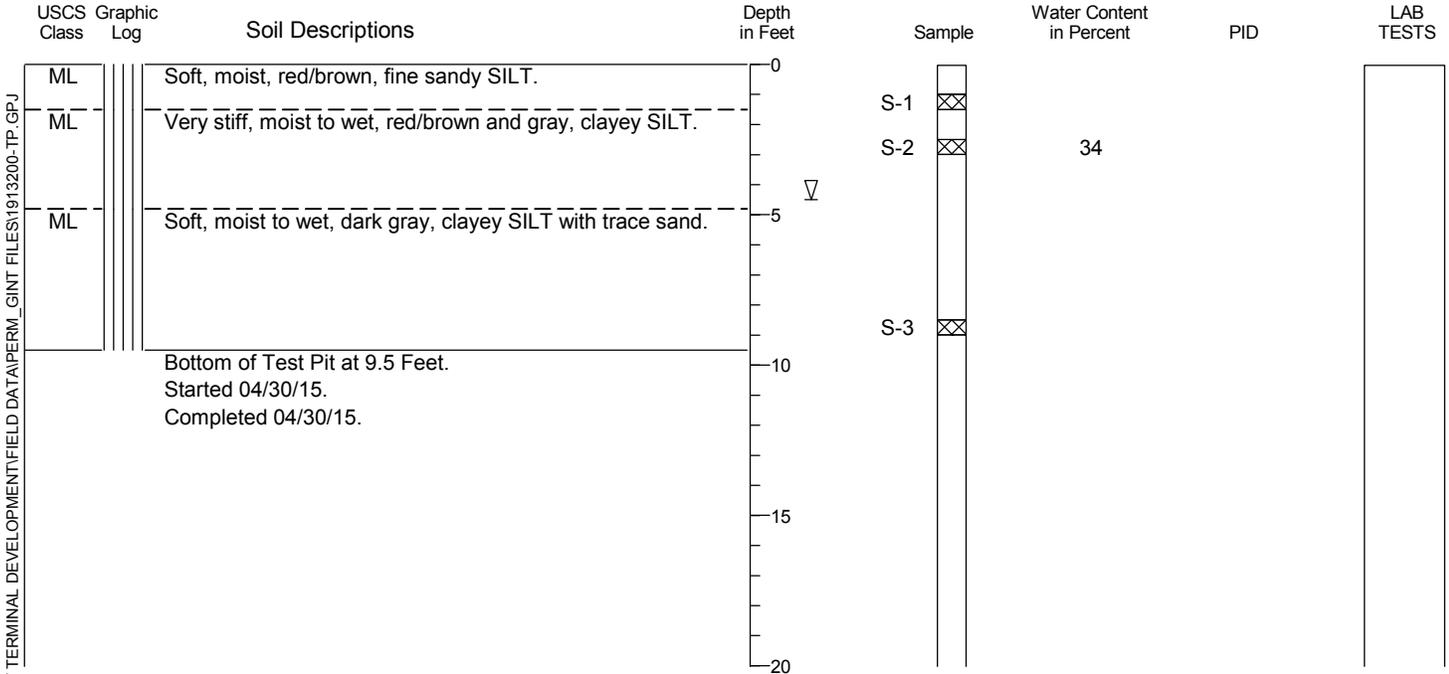
1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

NEW TEST PIT LOG - F:\GINT\OREGON_LIBRARY.GLB - 11/3/15 13:02 - L:\NOTEBOOKS\1913200_BARLOW POINT TERMINAL DEVELOPMENT\FIELD DATA\PERM_GINT FILES\1913200-TP.GPJ

Test Pit Log TP- 3

Location: N 46.1504 E -123.0275
 Approximate Ground Surface Elevation: 6 Feet
 Logged By: J. Strobel/G. Wade Reviewed By: J. Alders

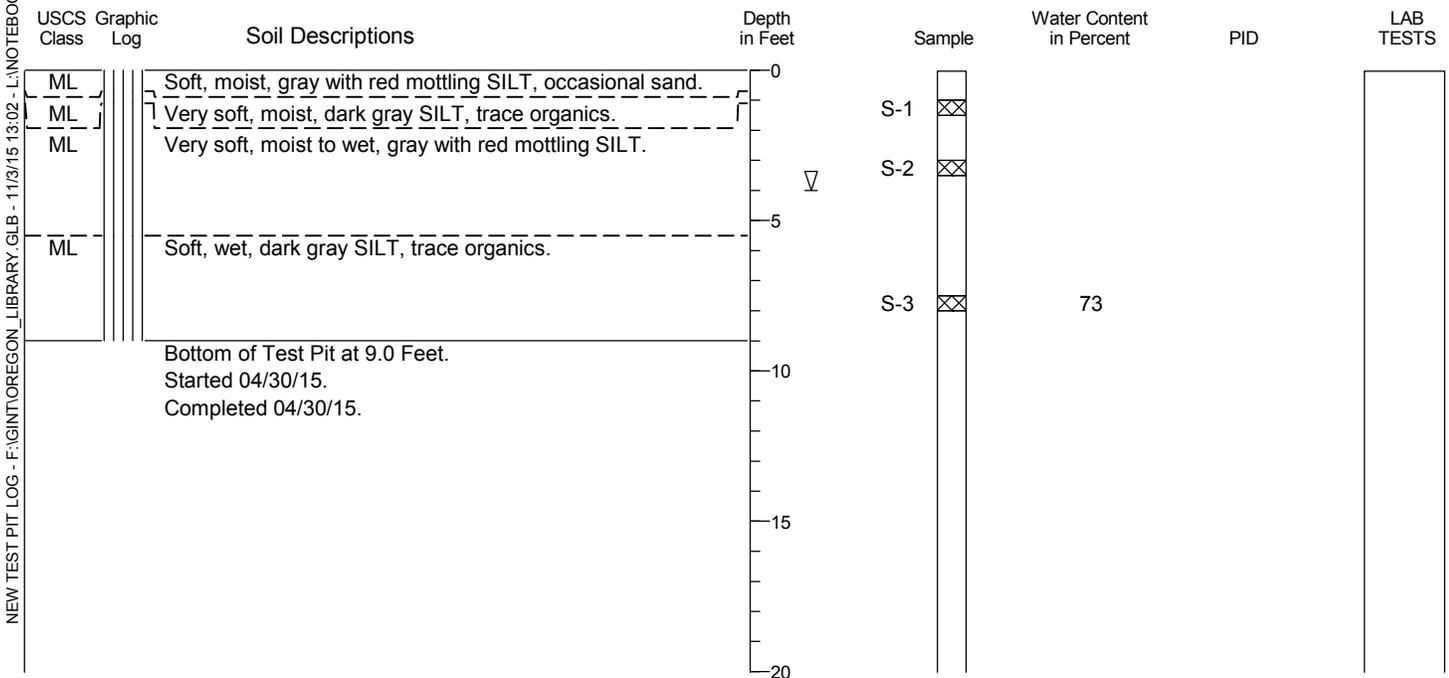
Horizontal Datum: WGS84
 Vertical Datum: NAVD88



Test Pit Log TP- 4

Location: N 46.1484 E -123.0309
 Approximate Ground Surface Elevation: 7 Feet
 Logged By: J. Strobel/G. Wade Reviewed By: J. Alders

Horizontal Datum: WGS84
 Vertical Datum: NAVD88



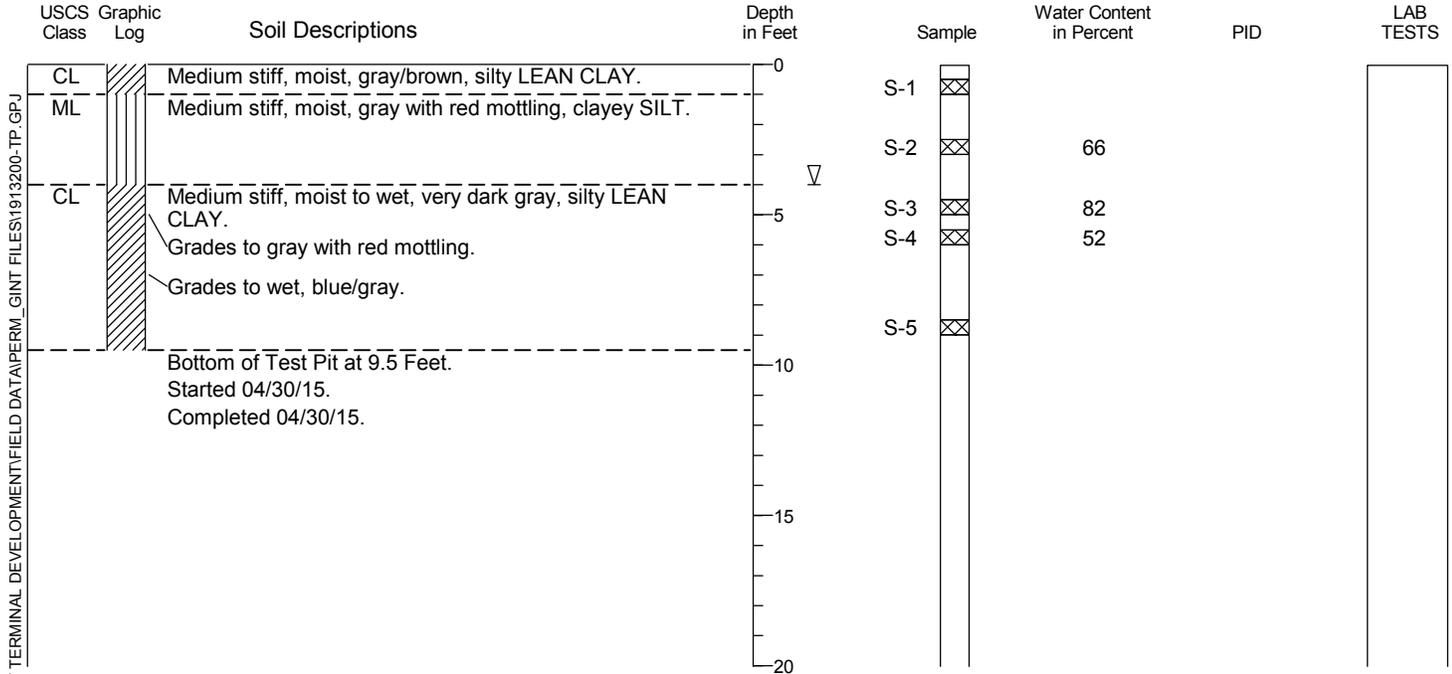
1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

NEW TEST PIT LOG - F:\GINT\OREGON_LIBRARY.GLB - 11/3/15 13:02 - L:\NOTEBOOKS\1913200_BARLOW POINT TERMINAL DEVELOPMENT\FIELD DATA\PERM_GINT FILES\1913200-TP.GPJ

Test Pit Log TP- 5

Location: N 46.1483 E -123.027
 Approximate Ground Surface Elevation: 8 Feet
 Logged By: J. Strobel/G. Wade Reviewed By: J. Alders

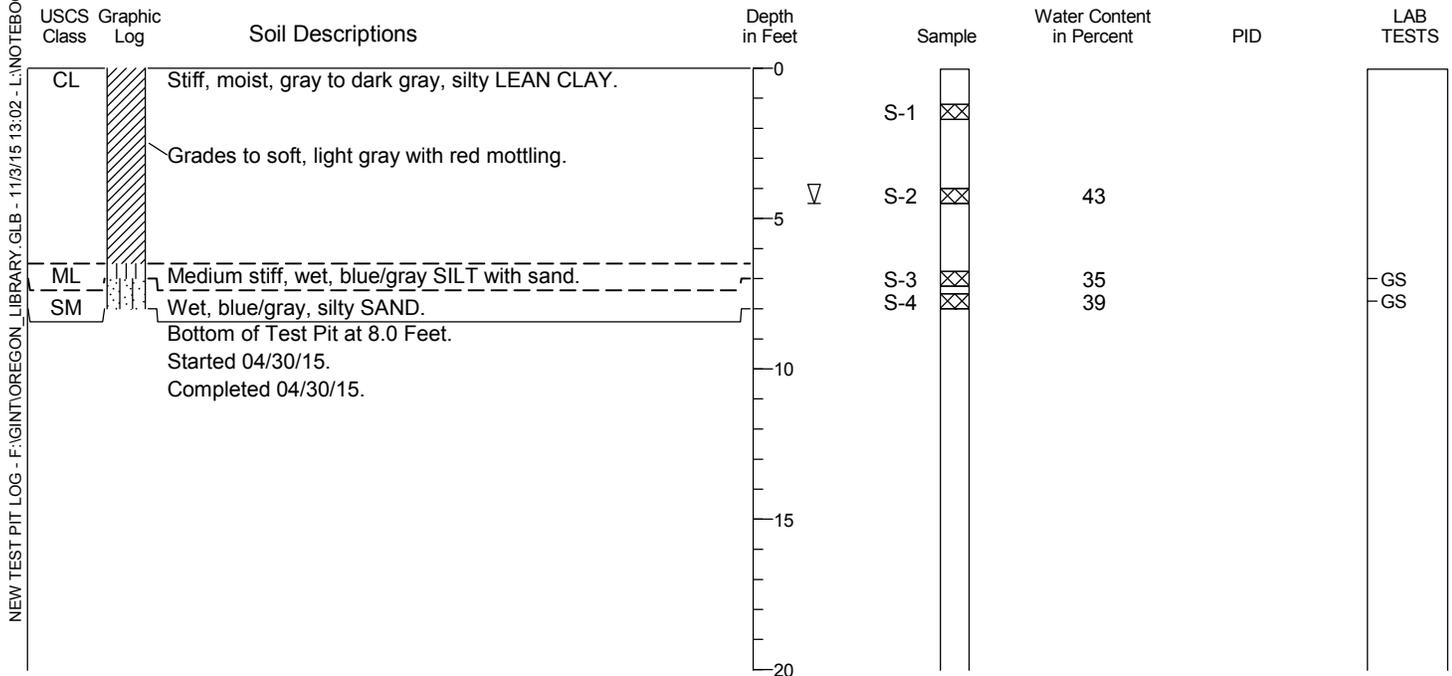
Horizontal Datum: WGS84
 Vertical Datum: NAVD88



Test Pit Log TP- 6

Location: N 46.1486 E -123.0234
 Approximate Ground Surface Elevation: 7 Feet
 Logged By: J. Strobel/G. Wade Reviewed By: J. Alders

Horizontal Datum: WGS84
 Vertical Datum: NAVD88



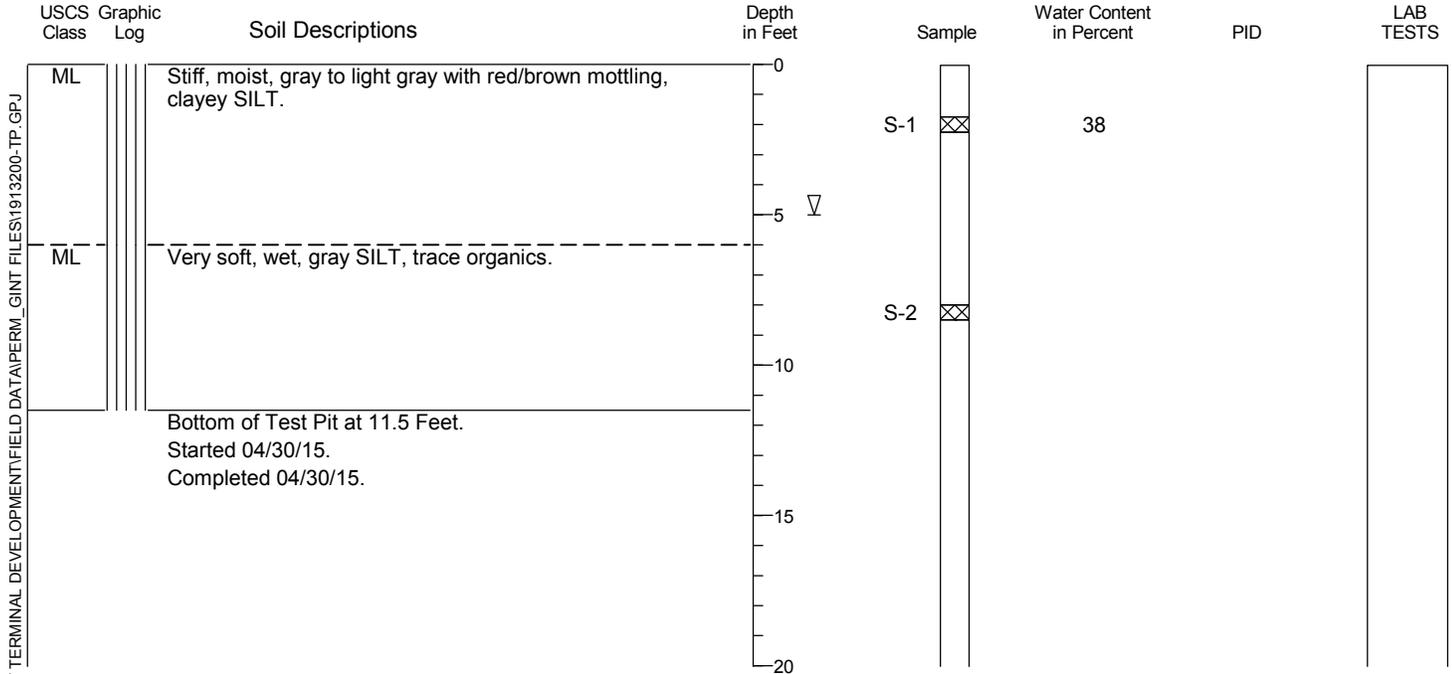
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1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

Test Pit Log TP- 7

Location: N 46.1463 E -123.0251
 Approximate Ground Surface Elevation: 9 Feet
 Logged By: J. Strobel/G. Wade Reviewed By: J. Alders

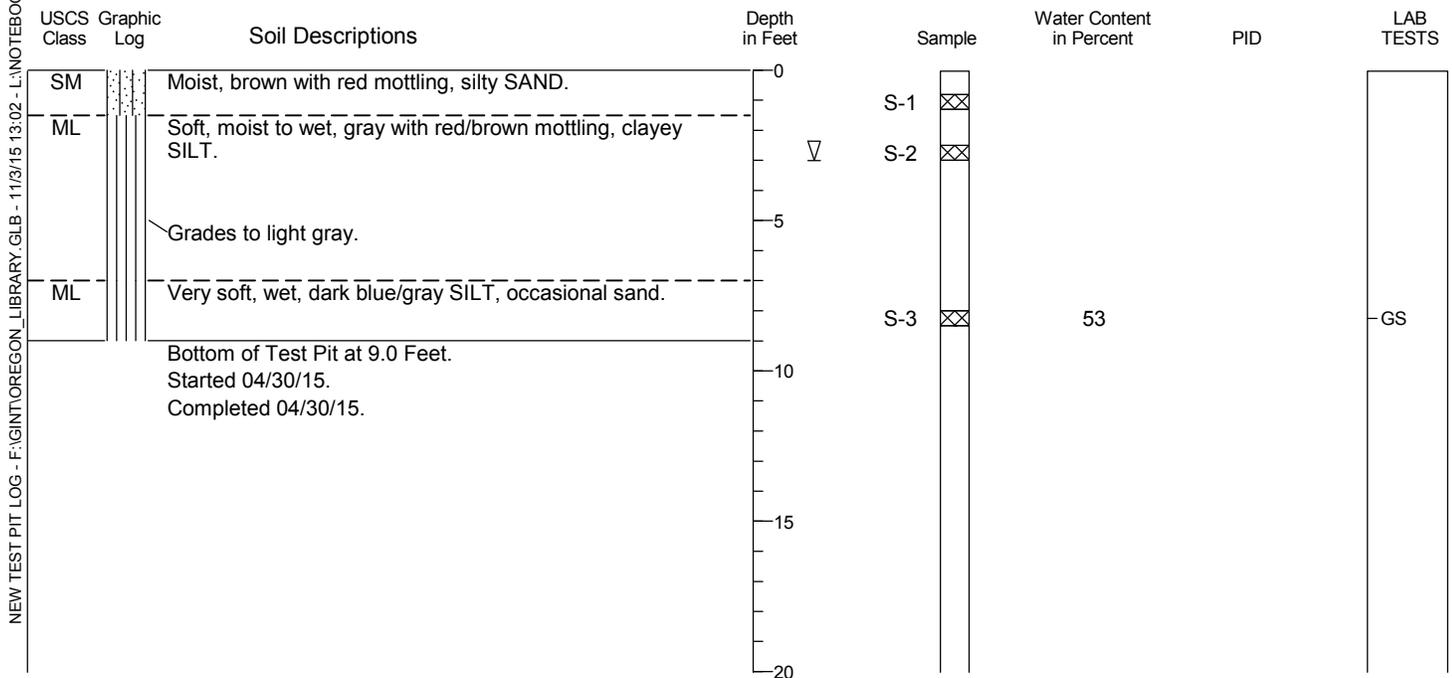
Horizontal Datum: WGS84
 Vertical Datum: NAVD88



Test Pit Log TP- 8

Location: N 46.1461 E -123.0285
 Approximate Ground Surface Elevation: 11 Feet
 Logged By: J. Strobel/G. Wade Reviewed By: J. Alders

Horizontal Datum: WGS84
 Vertical Datum: NAVD88



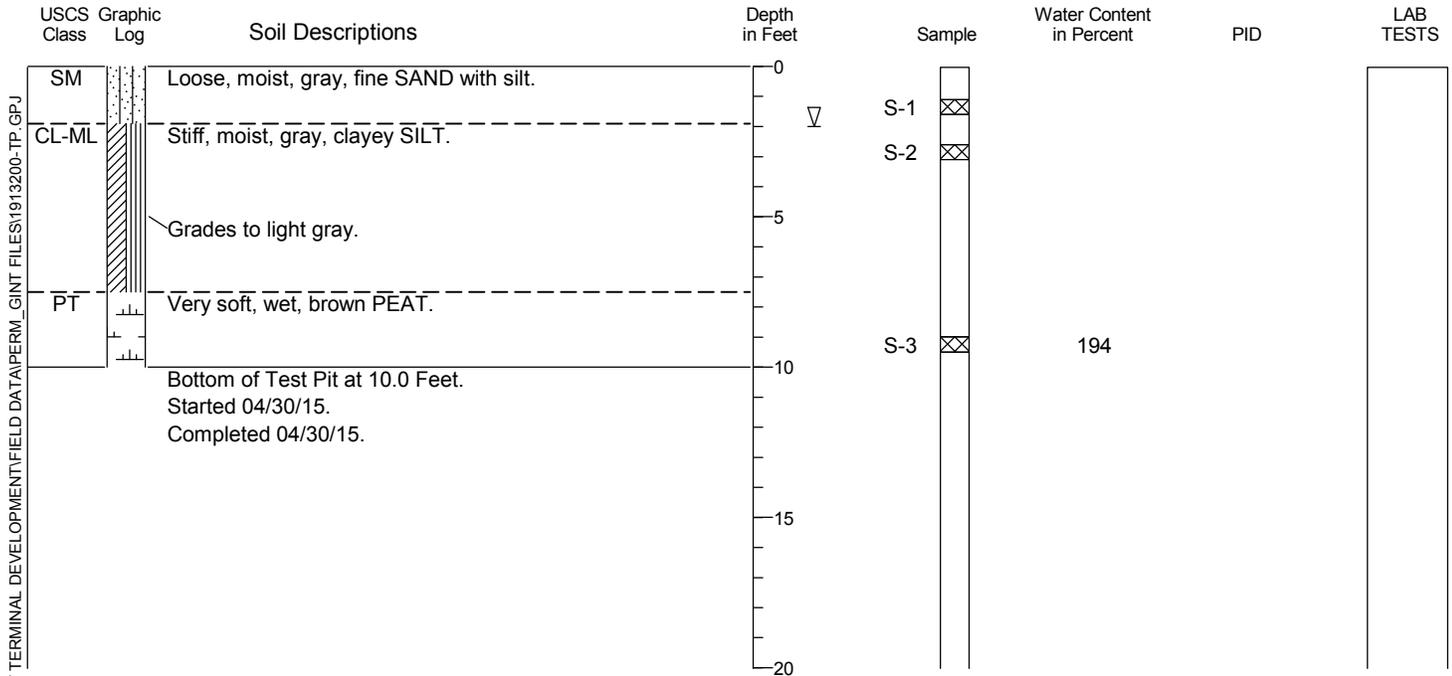
NEW TEST PIT LOG - F:\GINT\OREGON_LIBRARY.GLB - 11/3/15 13:02 - L:\NOTEBOOKS\1913200_BARLOW POINT TERMINAL DEVELOPMENT\FIELD DATA\PERM_GINT FILES\1913200-TP.GPJ

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

Test Pit Log TP- 9

Location: N 46.1441 E -123.0243
 Approximate Ground Surface Elevation: 11 Feet
 Logged By: J. Strobel/G. Wade Reviewed By: J. Alders

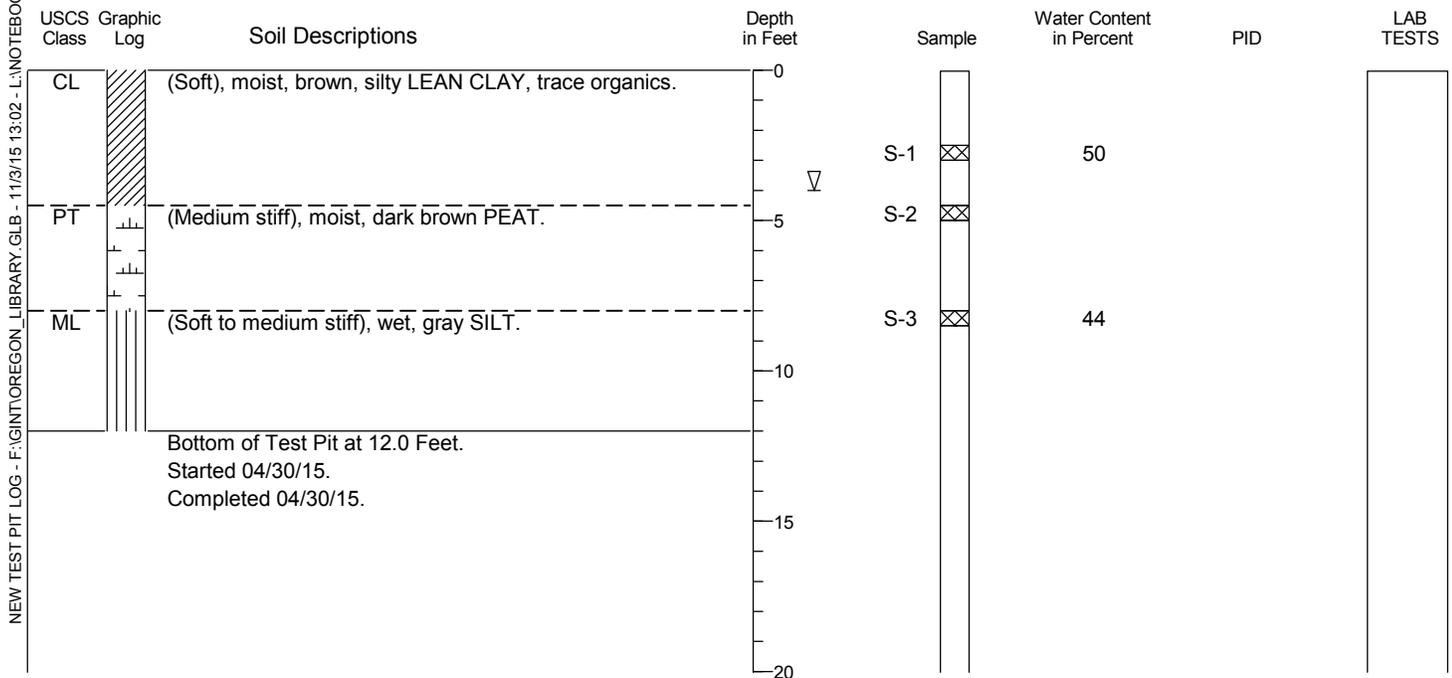
Horizontal Datum: WGS84
 Vertical Datum: NAVD88



Test Pit Log TP-10

Location: N 46.1451 E -123.0232
 Approximate Ground Surface Elevation: 8 Feet
 Logged By: J. Strobel/G. Wade Reviewed By: J. Alders

Horizontal Datum: WGS84
 Vertical Datum: NAVD88



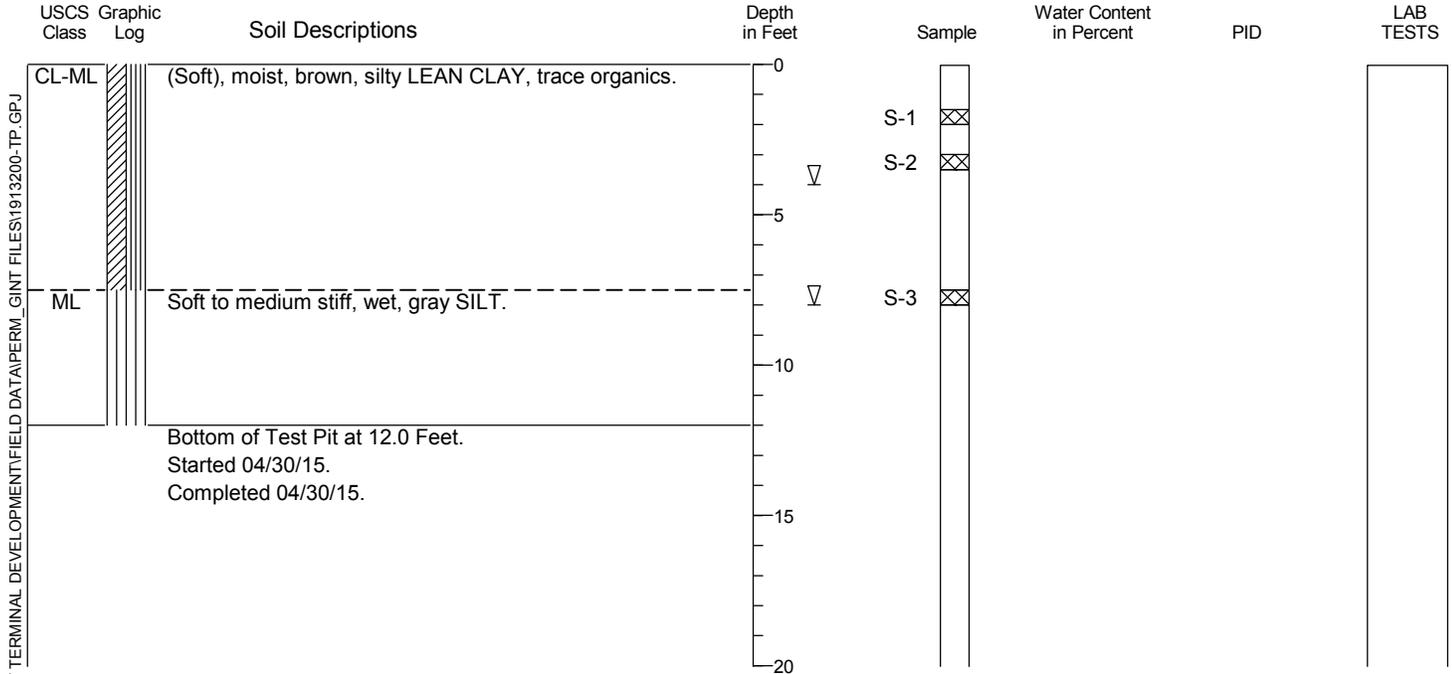
NEW TEST PIT LOG - F:\GINT\OREGON_LIBRARY.GLB - 11/3/15 13:02 - L:\NOTEBOOKS\1913200_BARLOW POINT TERMINAL DEVELOPMENT\FIELD DATA\PERM_GINT_FILES\1913200-TP.GPJ

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

Test Pit Log TP-11

Location: N 46.1462 E -123.0207
 Approximate Ground Surface Elevation: 7 Feet
 Logged By: J. Strobel/G. Wade Reviewed By: J. Alders

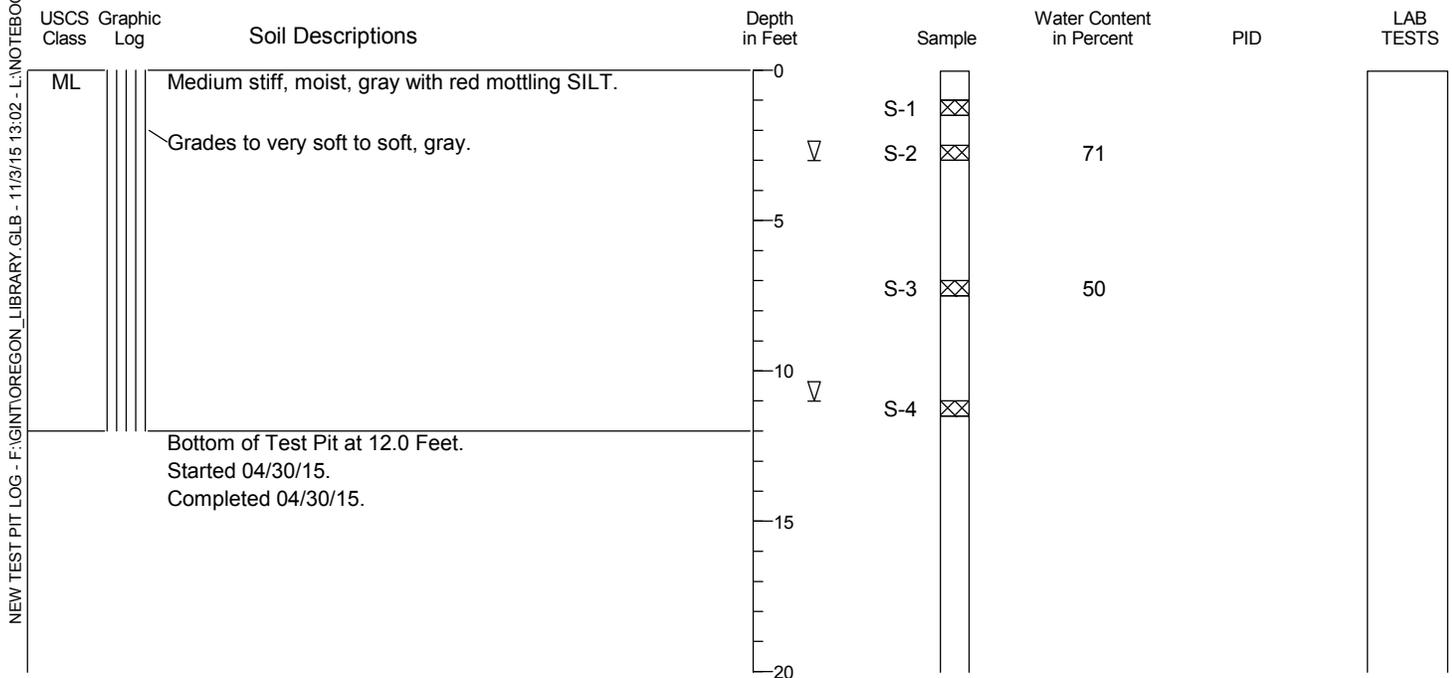
Horizontal Datum: WGS84
 Vertical Datum: NAVD88



Test Pit Log TP-12

Location: N 46.1482 E -123.0182
 Approximate Ground Surface Elevation: 8 Feet
 Logged By: J. Strobel/G. Wade Reviewed By: J. Alders

Horizontal Datum: WGS84
 Vertical Datum: NAVD88



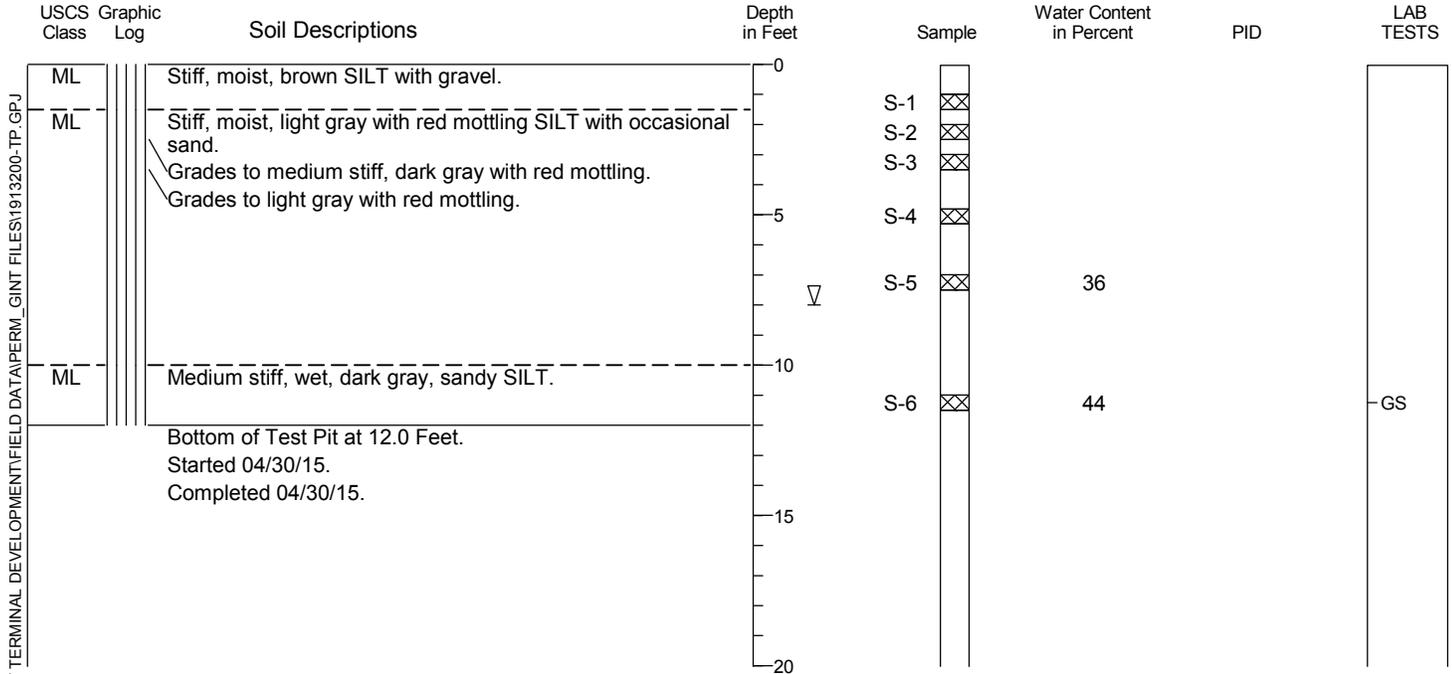
NEW TEST PIT LOG - F:\GINT\OREGON_LIBRARY.GLB - 11/3/15 13:02 - L:\NOTEBOOKS\1913200_BARLOW POINT TERMINAL DEVELOPMENT\FIELD DATA\PERM_GINT_FILES\1913200-TP.GPJ

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

Test Pit Log TP-13

Location: N 46.1469 E -123.0171
 Approximate Ground Surface Elevation: 7 Feet
 Logged By: J. Strobel/G. Wade Reviewed By: J. Alders

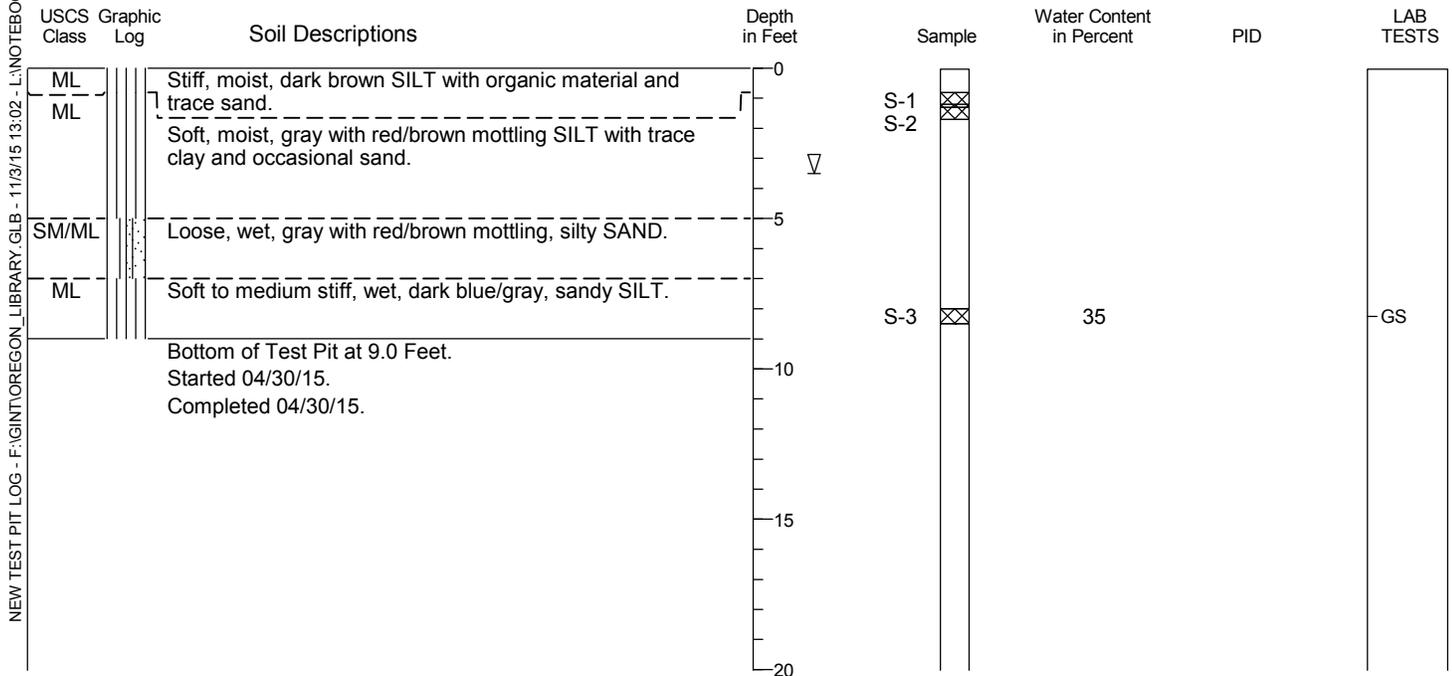
Horizontal Datum: WGS84
 Vertical Datum: NAVD88



Test Pit Log TP-14

Location: N 46.1442 E -123.0173
 Approximate Ground Surface Elevation: 8 Feet
 Logged By: J. Strobel/G. Wade Reviewed By: J. Alders

Horizontal Datum: WGS84
 Vertical Datum: NAVD88



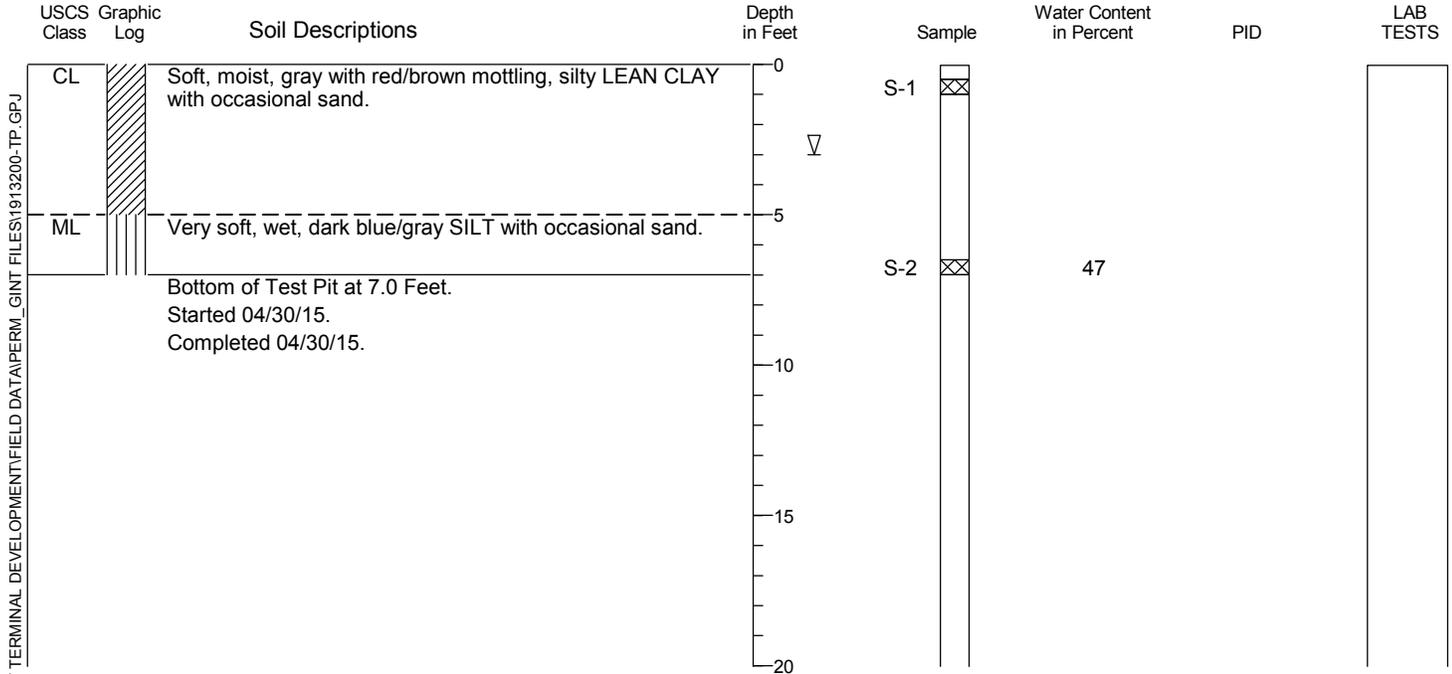
1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

NEW TEST PIT LOG - F:\GINT\OREGON_LIBRARY.GLB - 11/3/15 13:02 - L:\NOTEBOOKS\19132000_BARLOW POINT TERMINAL DEVELOPMENT\FIELD DATA\PERM_GINT_FILES\19132000-TP.GPJ

Test Pit Log TP-15

Location: N 46.1435 E -123.0186
 Approximate Ground Surface Elevation: 8 Feet
 Logged By: J. Strobel/G. Wade Reviewed By: J. Alders

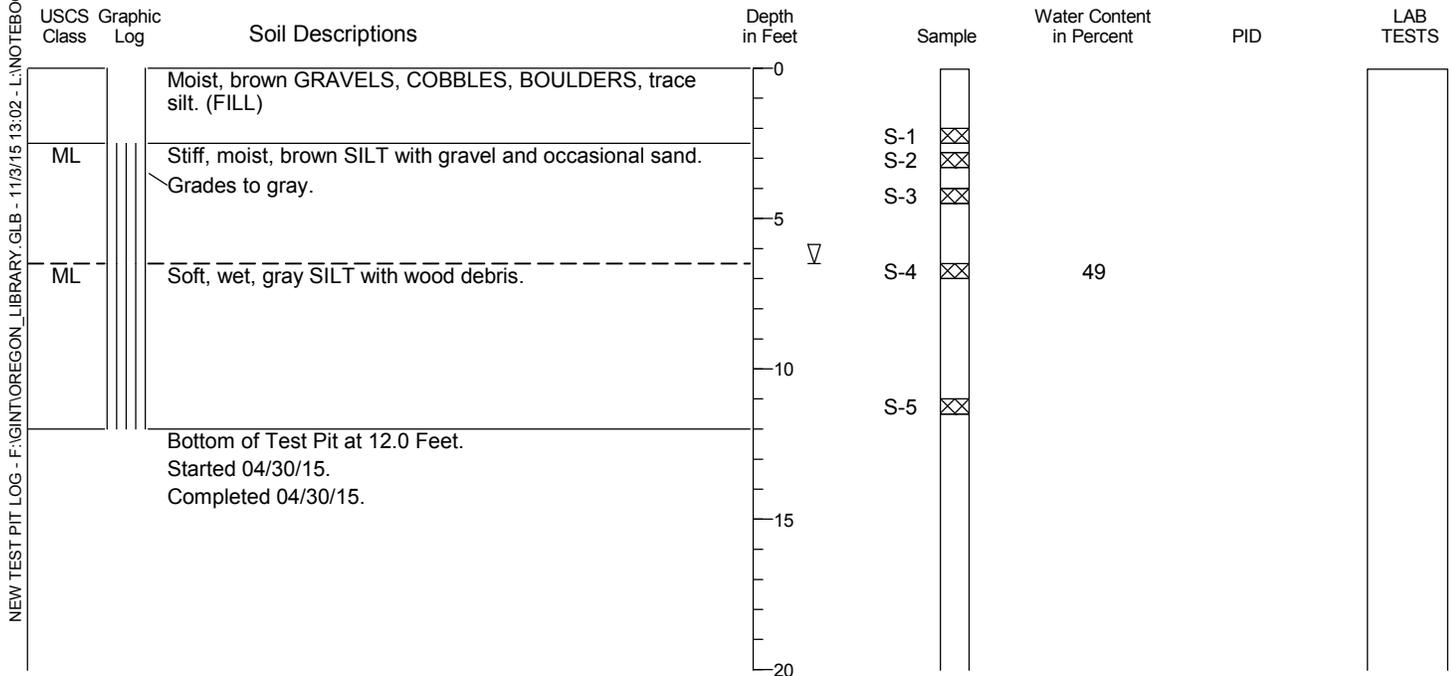
Horizontal Datum: WGS84
 Vertical Datum: NAVD88



Test Pit Log TP-16

Location: N 46.1485 E -123.014
 Approximate Ground Surface Elevation: 9 Feet
 Logged By: J. Strobel/G. Wade Reviewed By: J. Alders

Horizontal Datum: WGS84
 Vertical Datum: NAVD88



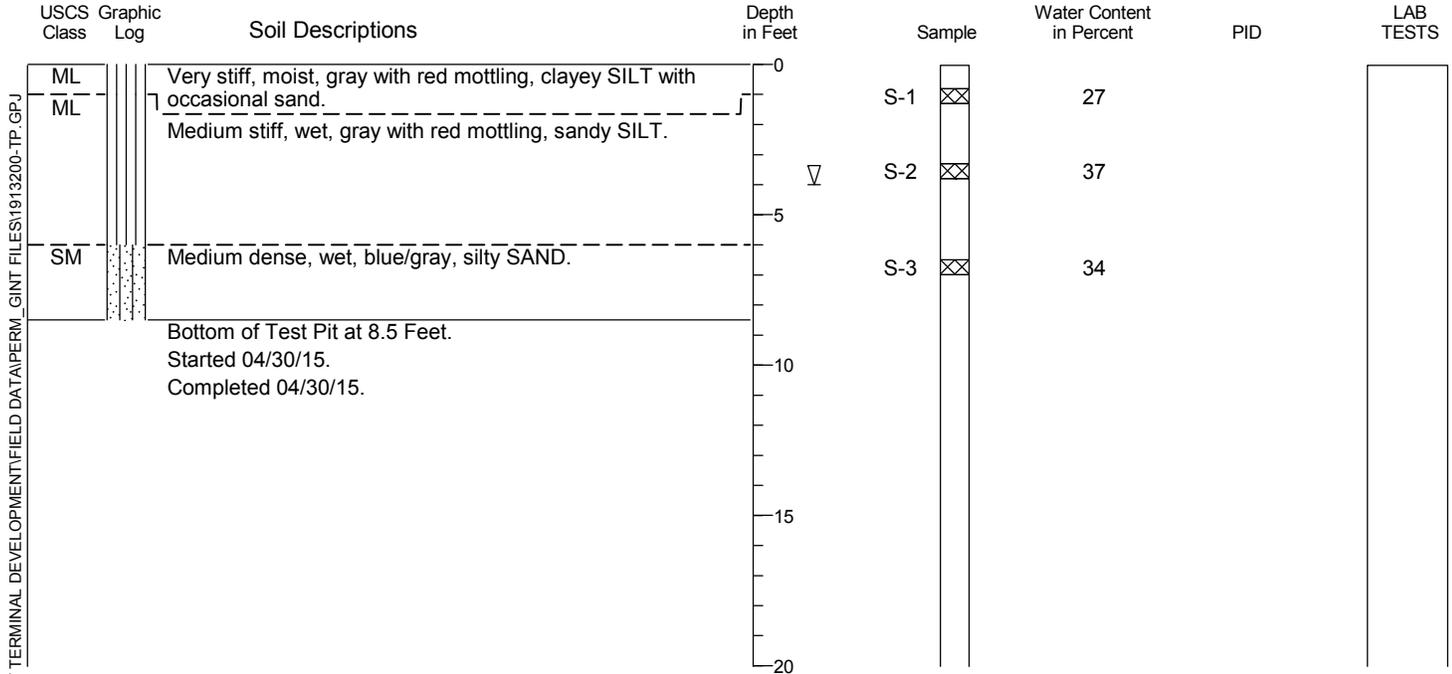
1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

NEW TEST PIT LOG - F:\GINT\OREGON_LIBRARY.GLB - 11/3/15 13:02 - L:\NOTEBOOKS\1913200_BARLOW POINT TERMINAL DEVELOPMENT\FIELD DATA\PERM_GINT FILES\1913200-TP.GPJ

Test Pit Log TP-17

Location: N 46.1447 E -123.0206
 Approximate Ground Surface Elevation: 8 Feet
 Logged By: J. Strobel/G. Wade Reviewed By: J. Alders

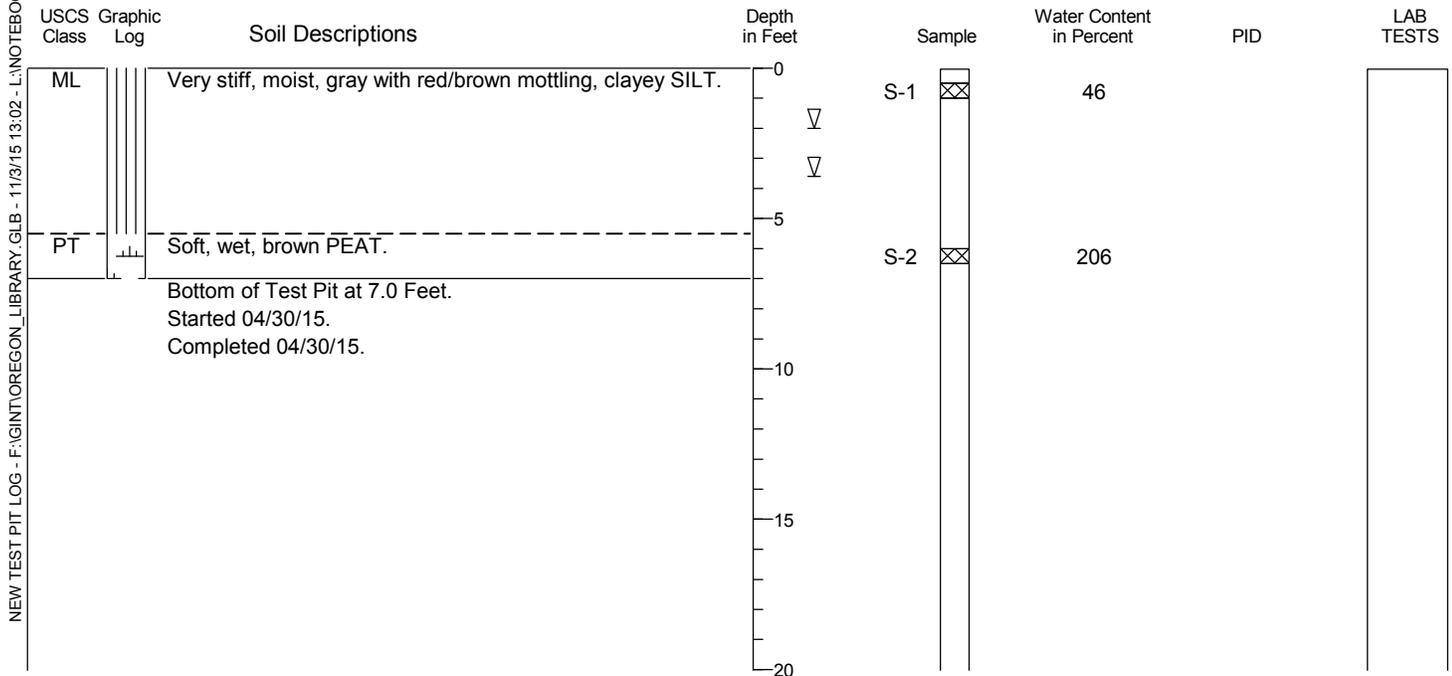
Horizontal Datum: WGS84
 Vertical Datum: NAVD88



Test Pit Log TP-18

Location: N 46.1453 E -123.0263
 Approximate Ground Surface Elevation: 9 Feet
 Logged By: J. Strobel/G. Wade Reviewed By: J. Alders

Horizontal Datum: WGS84
 Vertical Datum: NAVD88



NEW TEST PIT LOG - F:\GINT\OREGON_LIBRARY_GLB - 11/3/15 13:02 - L:\NOTEBOOKS\1913200_BARLOW POINT TERMINAL DEVELOPMENT\FIELD DATA\PERM_GINT FILES\1913200-TP.GPJ

1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.



SUMMARY OF LABORATORY RESULTS

CLIENT KPFF Consulting Engineers

PROJECT NAME Barlow Point Terminal Development

PROJECT NUMBER 19132-00

PROJECT LOCATION Longview, Washington

Borehole	Sample ID	Depth	% Gravel	% Sand	% Fines	Liquid Limit	Plastic Limit	Water Content (%)	USCS Group Symbol	Soil Description
TP- 1	S-1	1.3								
TP- 1	S-2	3.3								
TP- 1	S-3	5.0						61.1		
TP- 1	S-4	7.0						51.5		
TP- 2	S-1	1.8								
TP- 2	S-2	3.5								
TP- 2	S-3	5.5						44.0		
TP- 2	S-4	11.0	0.0	56.1	43.9			34.1	SM	silty SAND
TP- 3	S-1	1.0								
TP- 3	S-2	2.5						33.5		
TP- 3	S-3	8.5								
TP- 4	S-1	1.0								
TP- 4	S-2	3.0								
TP- 4	S-3	7.5						72.9		
TP- 5	S-1	0.5								
TP- 5	S-2	2.5						65.9		
TP- 5	S-3	4.5						82.1		
TP- 5	S-4	5.5						51.8		
TP- 5	S-5	8.5								
TP- 6	S-1	1.2								
TP- 6	S-2	4.0						43.1		
TP- 6	S-3	6.8	0.5	15.5	84.0			34.9	ML	SILT with sand
TP- 6	S-4	7.5	0.0	79.9	20.1			39.5	SM	silty SAND
TP- 7	S-1	1.8						37.9		
TP- 7	S-2	8.0								
TP- 8	S-1	0.8								
TP- 8	S-2	2.5								
TP- 8	S-3	8.0	0.0	10.7	89.3			53.3	ML	SILT, occasional sand
TP- 9	S-1	1.1								
TP- 9	S-2	2.6								
TP- 9	S-3	9.0						194.5		%OC = 32.8
TP-10	S-1	2.5						49.8		
TP-10	S-2	4.5								
TP-10	S-3	8.0						43.8		
TP-11	S-1	1.5								
TP-11	S-2	3.0								
TP-11	S-3	7.5								
TP-12	S-1	1.0								
TP-12	S-2	2.5						70.9		
TP-12	S-3	7.0						49.9		
TP-12	S-4	11.0								
TP-13	S-1	1.0								
TP-13	S-2	2.0								

SELECT SUMMARY WITH DESC.MOD01 - GINT STD US LAB.GDT - 06/15/10-43 - C:\USERS\MELISSASCHWEITZER\DESKTOP\PERM_GINT FILES\1913200-TP.GPJ

FIGURE A-11



SUMMARY OF LABORATORY RESULTS

CLIENT KPFF Consulting Engineers

PROJECT NAME Barlow Point Terminal Development

PROJECT NUMBER 19132-00

PROJECT LOCATION Longview, Washington

Borehole	Sample ID	Depth	% Gravel	% Sand	% Fines	Liquid Limit	Plastic Limit	Water Content (%)	USCS Group Symbol	Soil Description
TP-13	S-3	3.0								
TP-13	S-4	4.8								
TP-13	S-5	7.0						36.5		%OC = 2.2
TP-13	S-6	11.0	0.0	41.7	58.3			44.5	ML	sandy SILT
TP-14	S-1	0.8								
TP-14	S-2	1.2								
TP-14	S-3	8.0	0.0	38.1	61.9			35.0	ML	sandy SILT
TP-15	S-1	0.5								
TP-15	S-2	6.5						47.1		
TP-16	S-1	2.0								
TP-16	S-2	2.8								
TP-16	S-3	4.0								
TP-16	S-4	6.5						48.7		%OC = 5.4
TP-16	S-5	11.0								
TP-17	S-1	0.8						27.1		
TP-17	S-2	3.3						37.1		
TP-17	S-3	6.5						34.0		
TP-18	S-1	0.5						45.9		%OC = 5.9
TP-18	S-2	6.0						205.7		%OC = 25.4

SELECT SUMMARY WITH DESC.MOD01 - GINT STD US LAB.GDT - 06/15 10:43 - C:\USERS\MELISSASCHWEITZER\DESKTOP\PERM_GINT FILES\1913200-TP.GPJ

FIGURE A-11

ATTACHMENT B
Cone Penetrometer Test (CPT) Sounding Procedures

ATTACHMENT B

Cone Penetrometer Test (CPT) Sounding Procedures

CPT Soundings

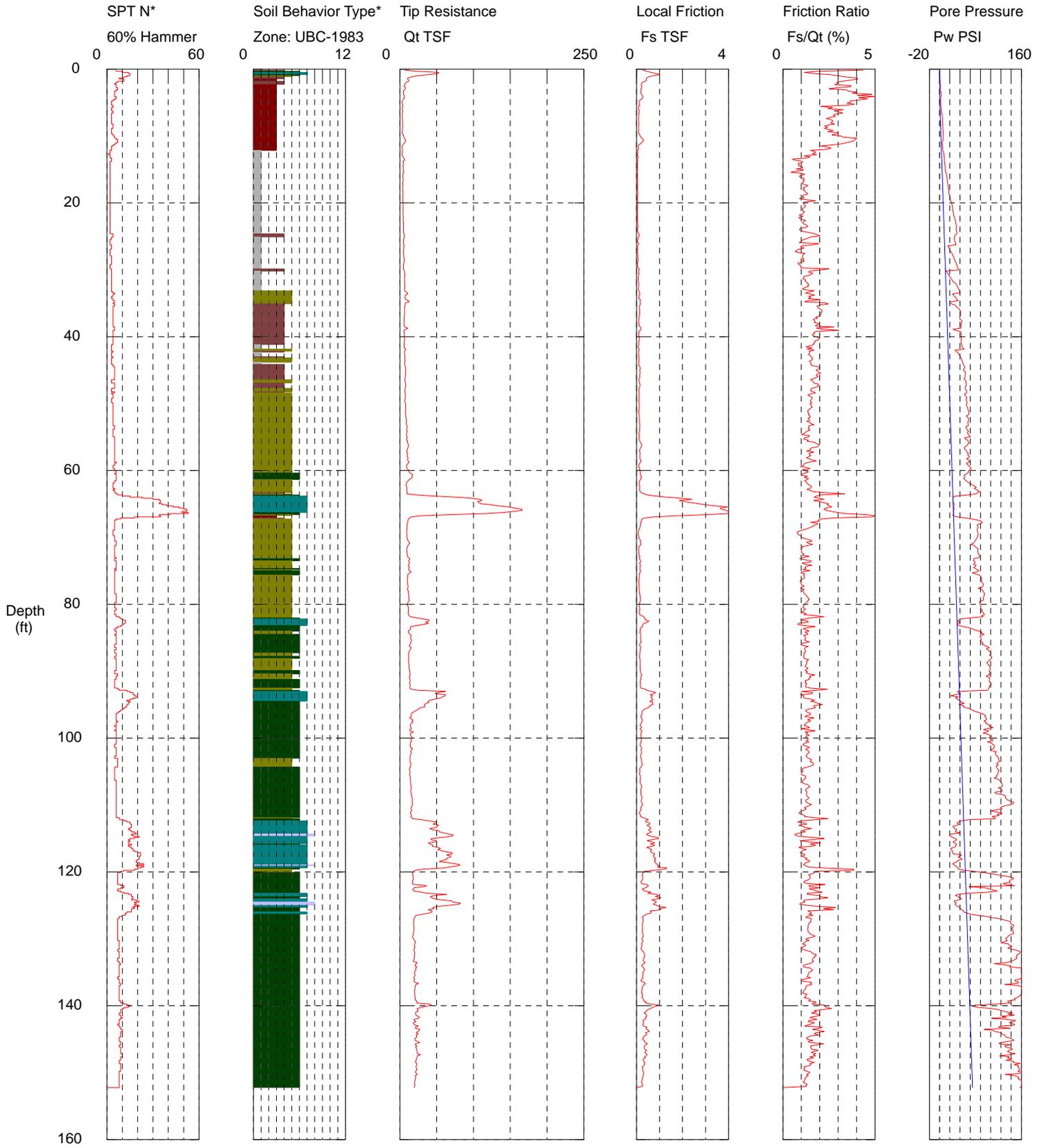
Four CPT soundings were advanced in general accordance with ASTM D 5778 using a seismic electronic cone penetrometer manufactured by Hogentogler & Company, Inc. and operated by Oregon Geotechnical Explorations of Keizer, Oregon. The CPT probe is an *in situ* test that provides assistance in characterizing subsurface stratigraphy. The test included advancing a 35.6-mm-diameter cone equipped with a load cell, friction sleeve, strain gages, porous stone, and geophone through the soil profile. The cone was advanced at a rate of approximately 2 centimeters per second. Tip resistance, sleeve friction, and pore pressure were recorded at 0.1-meter intervals. Shear wave velocity of the subsurface soils was measured at increments of 2 meters in sounding CPT-2.

This attachment presents the results of the CPT soundings completed for this project.

Hart Crowser / CPT-1 / Barlow Point Port of Longview

Operator: OGE TAJ
 Sounding: CPT-1
 Cone Used: DDG1323

CPT Date/Time: 4/16/2015 11:58:18 AM
 Location: Hart Crowser / CPT-1 / Barlow Point Port of Longview
 Job Number: 15031 / Hart Crowser / CPT-1 / Barlow Point Port of Longview



Maximum Depth = 152.23 feet

Depth Increment = 0.164 feet

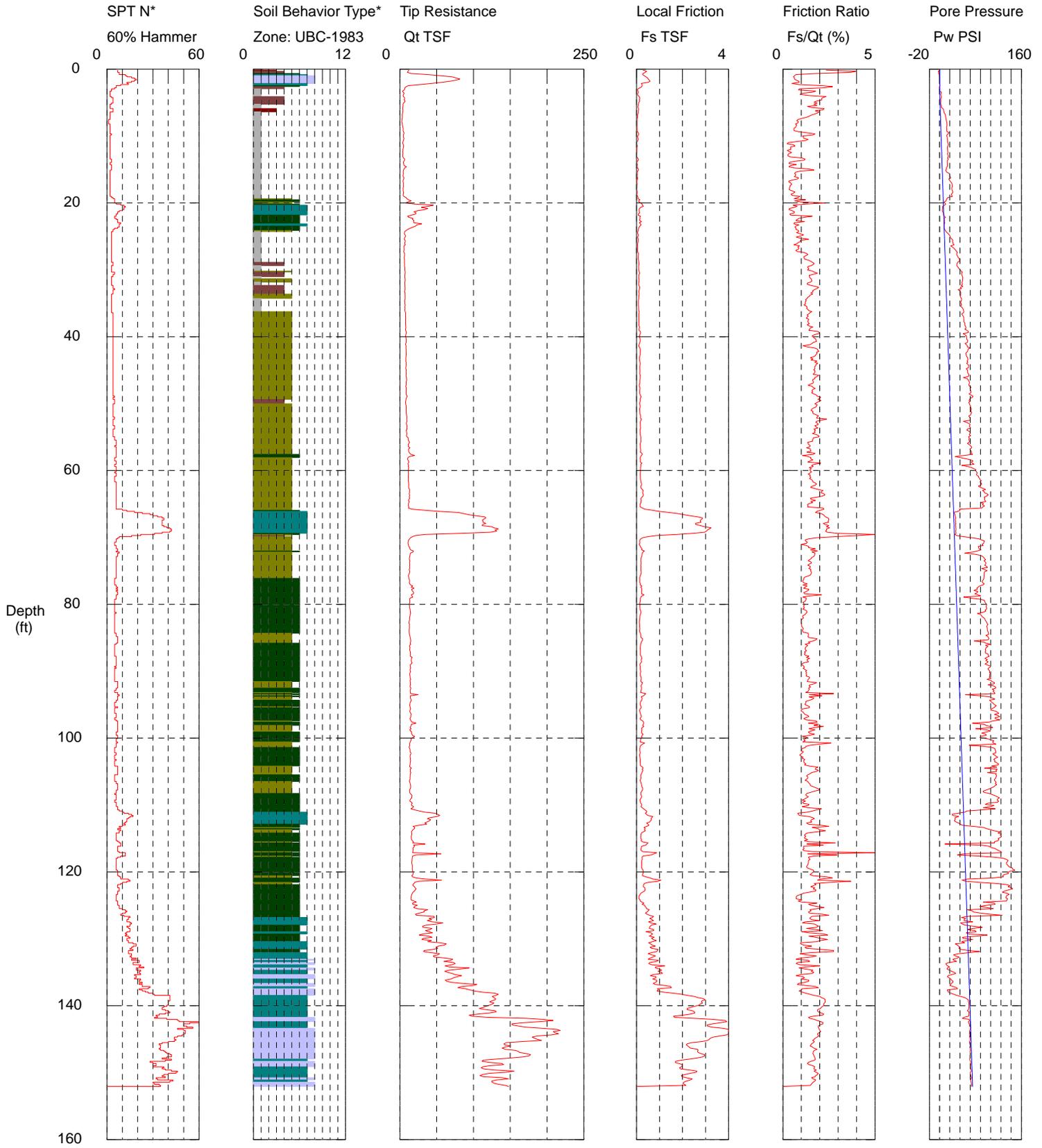
- | | | | |
|--------------------------|-----------------------------|----------------------------|--------------------------------|
| 1 sensitive fine grained | 4 silty clay to clay | 7 silty sand to sandy silt | 10 gravelly sand to sand |
| 2 organic material | 5 clayey silt to silty clay | 8 sand to silty sand | 11 very stiff fine grained (*) |
| 3 clay | 6 sandy silt to clayey silt | 9 sand | 12 sand to clayey sand (*) |

*Soil behavior type and SPT based on data from UBC-1983

Hart Crowser / CPT-2 / Barlow Point Port of Longview

Operator: OGE TAJ
 Sounding: CPT-2
 Cone Used: DDG1323

CPT Date/Time: 4/16/2015 9:06:39 AM
 Location: Hart Crowser / CPT-2 / Barlow Point Port of Longview
 Job Number: 15031 / Hart Crowser / CPT-2 / Barlow Point Port of Longview



Maximum Depth = 152.07 feet

Depth Increment = 0.164 feet

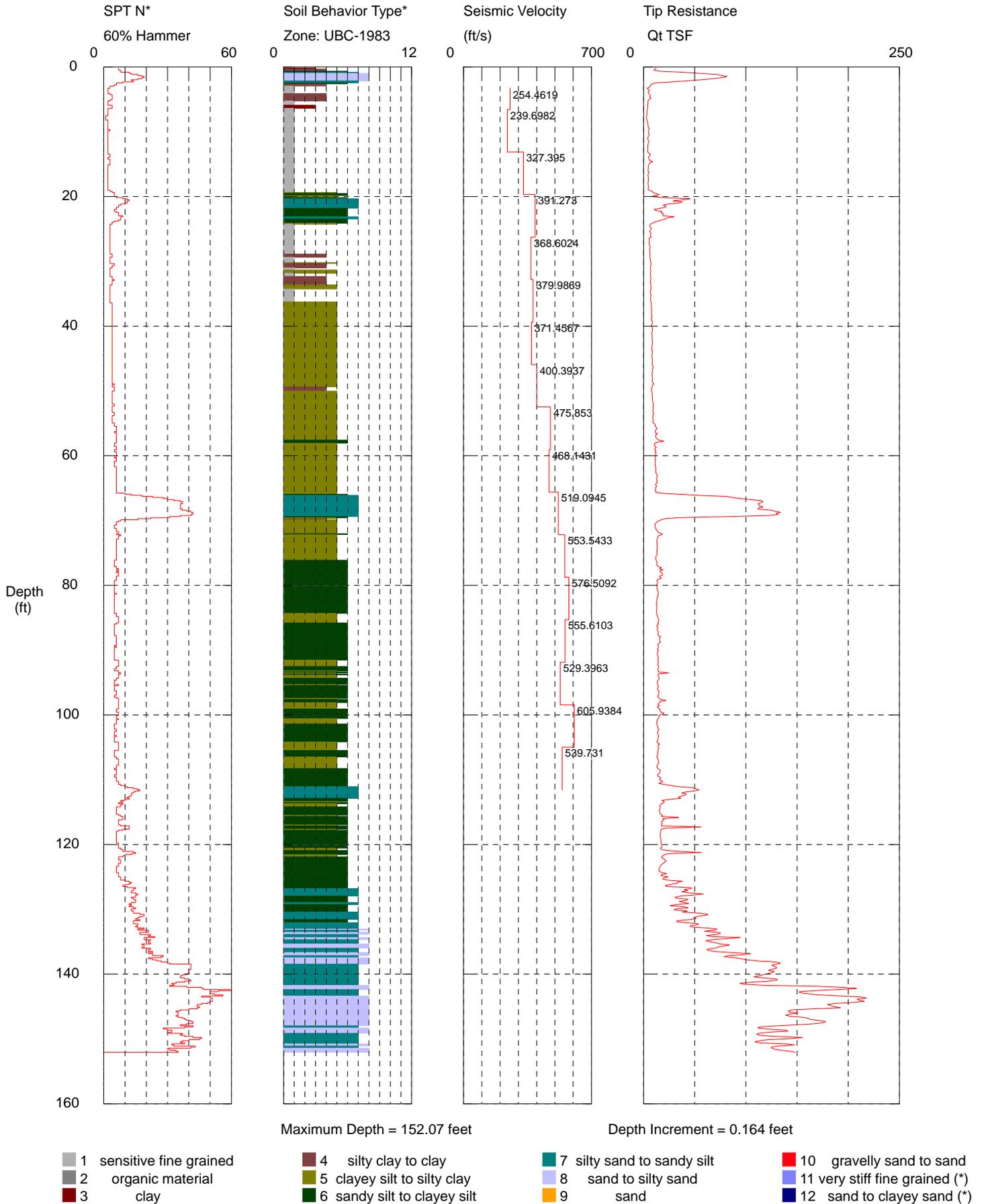
- | | | | |
|--------------------------|-----------------------------|----------------------------|--------------------------------|
| 1 sensitive fine grained | 4 silty clay to clay | 7 silty sand to sandy silt | 10 gravelly sand to sand |
| 2 organic material | 5 clayey silt to silty clay | 8 sand to silty sand | 11 very stiff fine grained (*) |
| 3 clay | 6 sandy silt to clayey silt | 9 sand | 12 sand to clayey sand (*) |

*Soil behavior type and SPT based on data from UBC-1983

Hart Crowser / CPT-2 / Barlow Point Port of Longview

Operator: OGE TAJ
 Sounding: CPT-2
 Cone Used: DDG1323

CPT Date/Time: 4/16/2015 9:06:39 AM
 Location: Hart Crowser / CPT-2 / Barlow Point Port of Longview
 Job Number: 15031 / Hart Crowser / CPT-2 / Barlow Point Port of Longview

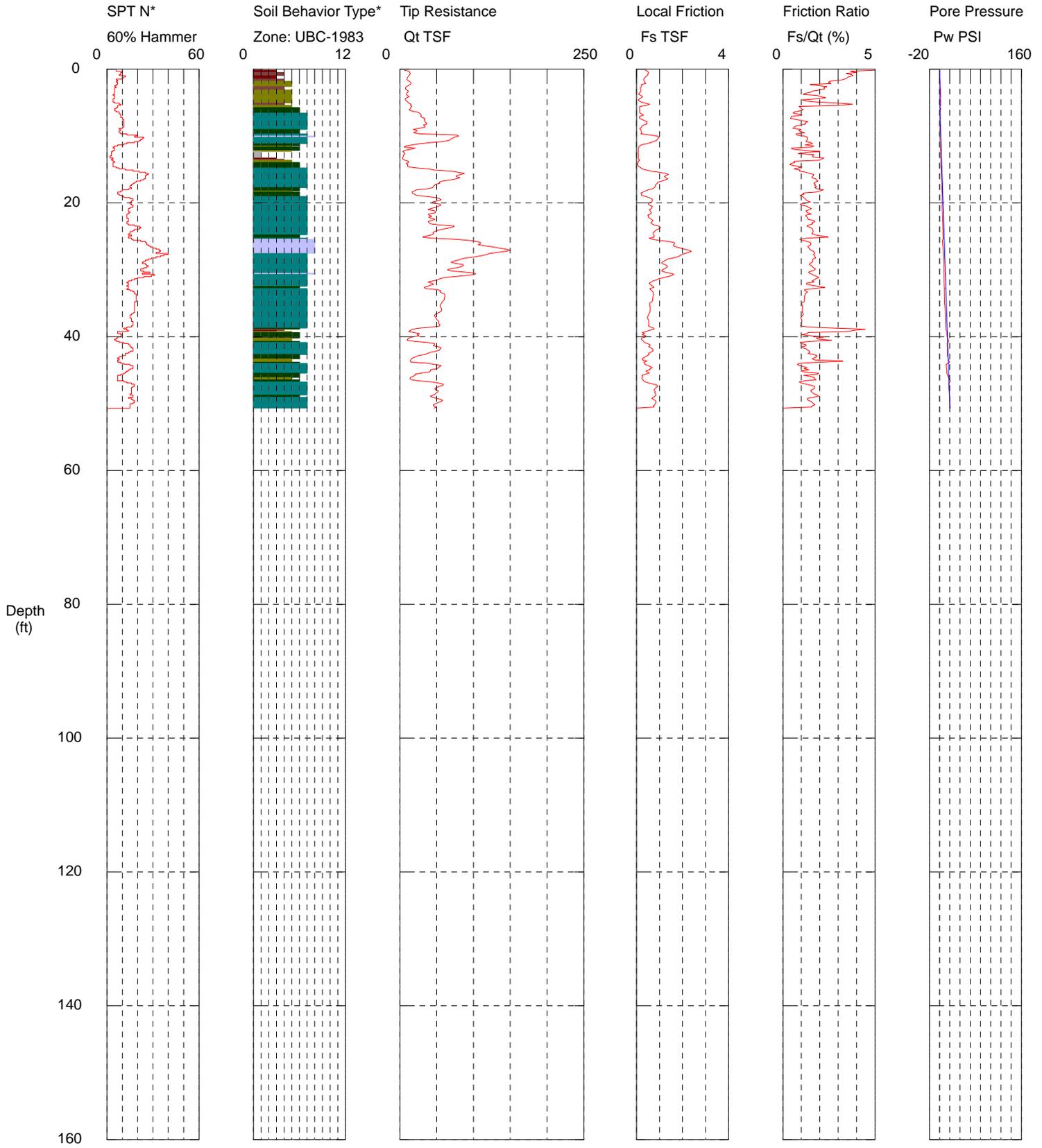


*Soil behavior type and SPT based on data from UBC-1983

Hart Crowser / CPT-3 / Barlow Point Port of Longview

Operator: OGE TAJ
 Sounding: CPT-3
 Cone Used: DDG1323

CPT Date/Time: 4/16/2015 2:27:32 PM
 Location: Hart Crowser / CPT-3 / Barlow Point Port of Longview
 Job Number: 15031 / Hart Crowser / CPT-3 / Barlow Point Port of Longview



Maximum Depth = 50.69 feet

Depth Increment = 0.164 feet

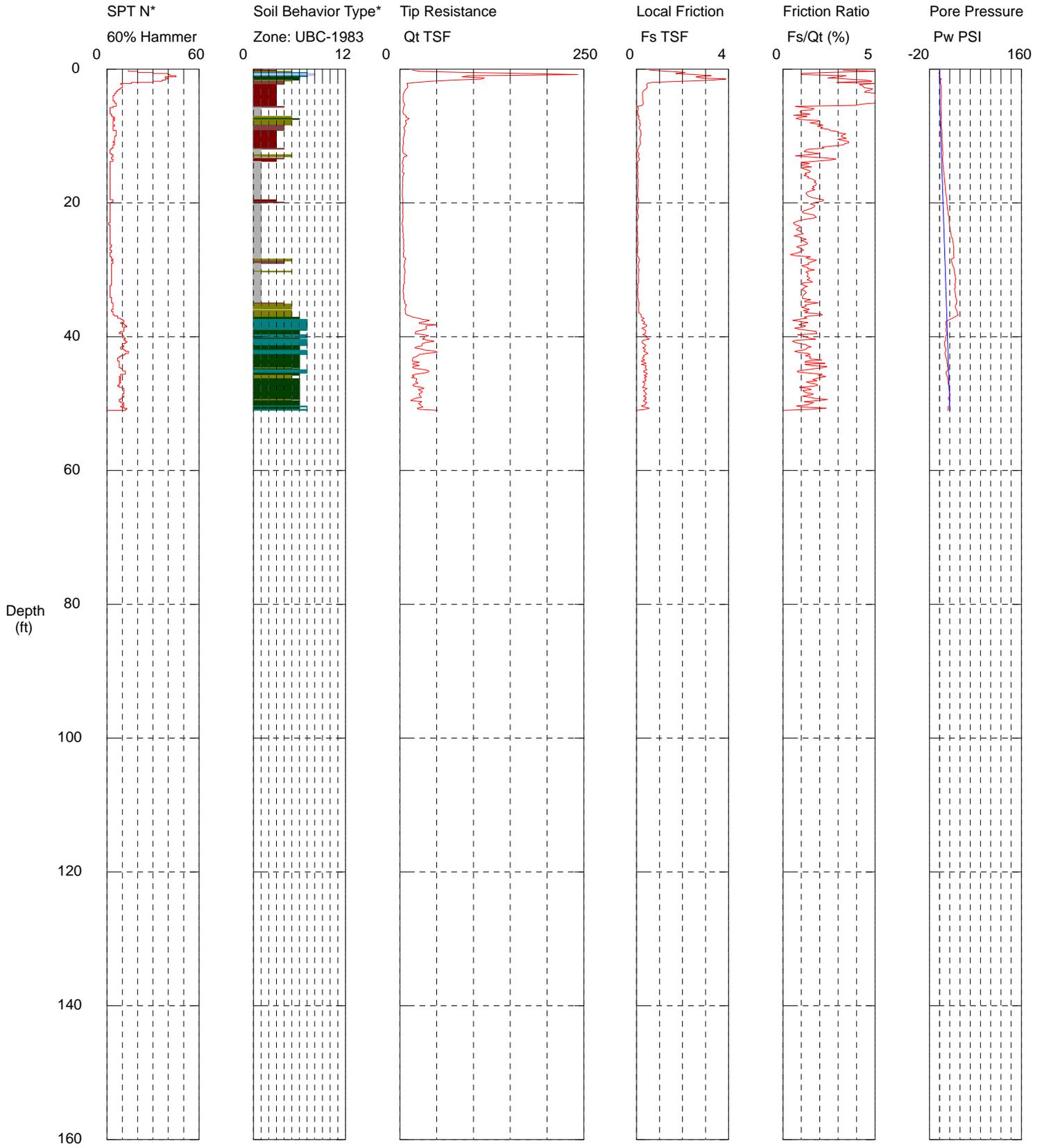
- | | | | |
|--------------------------|-----------------------------|----------------------------|--------------------------------|
| 1 sensitive fine grained | 4 silty clay to clay | 7 silty sand to sandy silt | 10 gravelly sand to sand |
| 2 organic material | 5 clayey silt to silty clay | 8 sand to silty sand | 11 very stiff fine grained (*) |
| 3 clay | 6 sandy silt to clayey silt | 9 sand | 12 sand to clayey sand (*) |

*Soil behavior type and SPT based on data from UBC-1983

Hart Crowser / CPT-4 / Barlow Point Port of Longview

Operator: OGE TAJ
 Sounding: CPT-4
 Cone Used: DDG1323

CPT Date/Time: 4/16/2015 3:52:17 PM
 Location: Hart Crowser / CPT-4 / Barlow Point Port of Longview
 Job Number: 15031 / Hart Crowser / CPT-4 / Barlow Point Port of Longview



Maximum Depth = 51.02 feet

Depth Increment = 0.164 feet

- | | | | |
|--------------------------|-----------------------------|----------------------------|--------------------------------|
| 1 sensitive fine grained | 4 silty clay to clay | 7 silty sand to sandy silt | 10 gravelly sand to sand |
| 2 organic material | 5 clayey silt to silty clay | 8 sand to silty sand | 11 very stiff fine grained (*) |
| 3 clay | 6 sandy silt to clayey silt | 9 sand | 12 sand to clayey sand (*) |

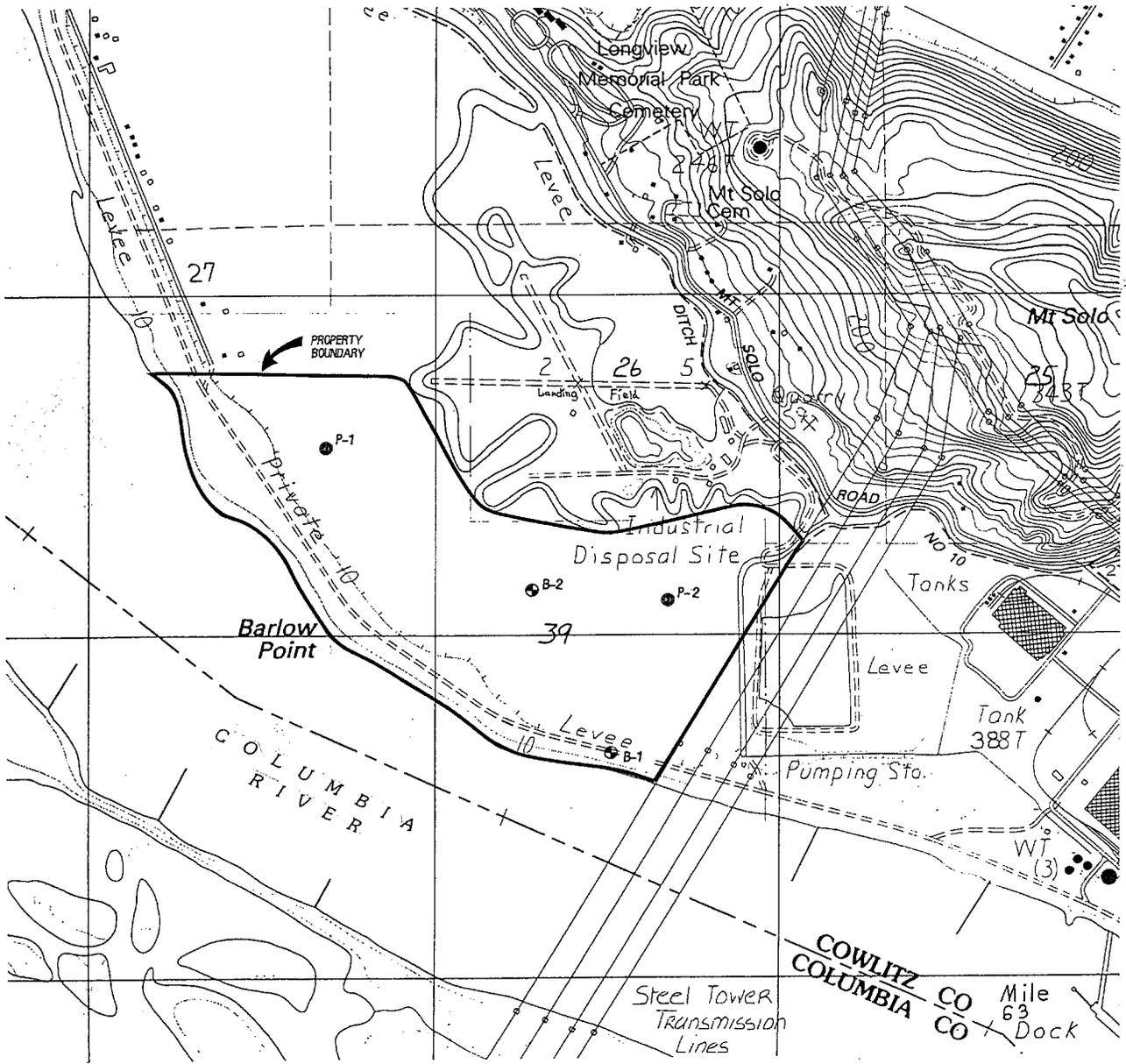
*Soil behavior type and SPT based on data from UBC-1983

ATTACHMENT C
Explorations by Others

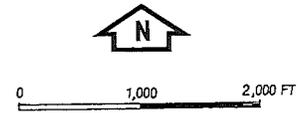
ATTACHMENT C

Explorations by Others

This attachment contains copies of the boring and CPT logs from a geotechnical investigation completed by GRI (GRI 1993) for preliminary design of a steel mill at the site. A site plan showing the exploration locations is also provided.



● BORING MADE BY GEOTECHNICAL RESOURCES, INC. (MAY 1993)
 ● CONE PENETRATION TEST MADE BY GEOTECHNICAL RESOURCES, INC. (MAY 1993)
 SITE PLAN FROM USGS TOPOGRAPHIC MAP, COAL CK, WA



NUCOR STEEL
 GEOTECHNICAL INVESTIGATION/BARLOW POINT, WASH.
GRI
SITE PLAN

APPENDIX A

FIELD EXPLORATIONS AND LABORATORY TESTING

FIELD EXPLORATIONS

General

Subsurface materials and conditions were investigated between May 11 and 19, 1993, with two borings, designated B-1 and B-2, and two cone penetration test (CPT) probes, designated P-1 and P-2. The approximate locations of the borings and probes are shown on the Site Plan, Figure 2. The borings were drilled to depths of 149 to 149.5 ft, and the CPT probes extended to depths of 175 to 189 ft.

Borings

The borings were drilled with a truck-mounted, Mobile B-61 drill rig using mud-rotary techniques. The drill rig was provided and operated by Subterranean, Inc. of Sumner, Washington. The drilling was observed by an engineer provided by our firm who maintained a detailed log of the conditions and materials encountered in each boring. Logs of borings are provided on Figures 1A and 2A.

Disturbed and undisturbed samples were typically obtained from the borings at 5- to 10-ft intervals. Disturbed samples were obtained using a standard split-spoon sampler. At the time of sampling, the Standard Penetration Test was conducted. This test is performed by driving a split-spoon sampler into the soil a total distance of 18 in. using the weight of a 140-lb hammer dropped 30 in. The number of blows required to drive the sampler the last 12 in. is called the standard penetration resistance, or N-value. The N-value provides a measurement of the relative density of granular soils, such as sand, and the relative consistency of cohesive soils, such as silt.

Relatively undisturbed samples of fine-grained, cohesive soils were obtained by pushing a 3-in.-O.D. Shelby tube into the undisturbed soil a distance of about 24 in. using the hydraulic ram of the drill rig. The soil exposed in the end of the Shelby tubes was examined and classified in the field. After classification, the ends of the tubes were sealed with rubber caps and tape to preserve the natural moisture content of the soils. All of the samples were returned to our laboratory for further examination and testing.

Observation Standpipes

A standpipe was installed in boring B-1 to permit measurement of the depth to the groundwater level. The standpipe consisted of a slotted 1-in.-diameter plastic riser pipe inserted to a depth of about 20 ft in the open borehole which was then backfilled with silica sand and capped with bentonite. Groundwater enters the pipe through the slots and rises to a static level. The groundwater level is measured using an electrical probe lowered inside the riser pipe.

Logs of Borings

Logs of the borings are provided on Figures 1A and 2A. Each log presents a descriptive summary of the various types of materials encountered in the boring and notes the depth where the materials and/or characteristics of the materials change. To the right of the descriptive summary, the numbers and types of samples taken during the drilling operation are indicated. Farther to the right, N-values are shown graphically, along with the natural moisture contents and shear strength values. The terms used to describe the soils are defined in Table 1A.

Cone Penetration Test Probes

The cone penetration tests were performed and interpreted and the soil classified by Subterranean, Inc. of Sumner, Washington. The soil classifications were reviewed by GRI. The cone penetration test (CPT) consists of forcing a hardened steel cone vertically into the soil at a constant rate of penetration. The thrust required to cause penetration at a constant rate is related to the bearing capacity of the soil immediately surrounding the point of the penetrometer cone. This value is known as the cone penetration resistance. After making the cone thrust measurement, a measurement is obtained of the magnitude of thrust required to force a special friction sleeve, attached above the cone, through the soil. The thrust required to move the friction sleeve can be related to the undrained shear strength of fine-grained soils. The dimensionless ratio of sleeve friction to point bearing capacity provides an indication of the type of soil penetrated. The cone penetration resistance and the sleeve friction are determined at 8-in. intervals in the probe hole and can be used to evaluate the relative density of cohesionless soils and the relative consistency of cohesive soils.

Logs of Cone Penetration Test Probes

The logs of CPT probes P-1 and P-2 are provided on Figures 3A and 4A. The logs show the values of cone penetration resistance and sleeve friction. To the right, the friction ratio (i.e., sleeve friction divided by the cone penetration resistance) is given, as well as an interpretation of the data with respect to the basic type of soil penetrated. Qualitative descriptions of relative consistency, based on cone penetration resistance and sleeve friction, are also presented on the logs. The terms used to describe the soils are defined in Table 2A. All soil classifications were reviewed by GRI.

LABORATORY TESTING

General

The samples obtained from the borings were examined in our laboratory where the physical characteristics of the samples were noted and the field classifications were modified where necessary. At the time of classification, the natural moisture content of each sample was determined. Additional testing included determining Torvane shear strength and unconfined compressive strength, consolidation characteristics, and dry unit weight of representative soil samples. The following paragraphs describe the testing program in more detail.

Natural Moisture Content

Natural moisture content determinations were made in conformance with ASTM D 2216. The results are presented on the Boring Logs, Figures 1A and 2A.

Torvane Shear Strength

The approximate undrained shear strength of relatively undisturbed soil samples was determined using a Torvane shear device. The Torvane is a hand-held apparatus with vanes which are inserted into the soil. The torque required to fail the soil in shear around the vanes is measured using a calibrated spring. The results of the Torvane shear tests are shown on the Boring Logs, Figures 1A and 2A.

Unconfined Compression

Unconfined compression tests were performed on selected undisturbed samples. The samples were tested using the constant-rate-of-strain procedure in conformance with ASTM D 2166. The data from the unconfined compression test are used to evaluate the relative consistency, i.e., softness or stiffness, of fine-grained, cohesive soils. Undrained shear strengths, based on unconfined compressive strength, $q_u/2$, are shown graphically on the Boring Logs, Figures 1A and 2A.

One-Dimensional Consolidation Tests

Five consolidation tests were performed in accordance with ASTM D 2435 to obtain data on the compressibility characteristics of the fine-grained soils. The consolidation testing was performed on specimens obtained from various depths. Results of the tests are summarized on Figures 5A through 9A in the form of curves showing effective stress versus percent strain. The initial and final moisture content and dry unit weight of the samples were determined in conjunction with the consolidation tests and are summarized at the top of each figure.

Dry Unit Weight

The unit weight of undisturbed soil samples was determined in the laboratory in accordance with ASTM D 2937 by cutting a cylindrical specimen of soil from a Shelby tube sample. The dimensions of the specimen were carefully measured, the volume calculated, and the specimen weighed. After oven drying, the specimen was reweighed and the water content calculated. The dry unit weight was then computed. The dry unit weights are summarized below.

SUMMARY OF DRY UNIT WEIGHTS

<u>Boring</u>	<u>Sample</u>	<u>Depth, ft</u>	<u>Dry Unit Weight, pcf</u>	<u>Moisture Content, %</u>	<u>Soil Type</u>
B-2	S-2	9.0	80	38	Silt
B-2	S-4	18.0	90	32	Sand
B-2	S-15	74.0	73	47	Silt
B-2	S-18	89.0	74	47	Silt
B-2	S-22	108.5	60	44	Silt
B-2	S-30	148.0	70	50	Silt
B-3	S-3	13.5	50	86	Silt
B-3	S-5	23.0	87	37	Sand
B-3	S-10	48.0	79	39	Sand
B-3	S-15	73.5	79	40	Silt
B-3	S-19	93.0	79	51	Silt
B-3	S-23	113.8	82	38	Silt
B-3	S-29	143.5	69	50	Silt

Table 1A

GUIDELINES FOR CLASSIFICATION OF SOIL

Description of Relative Density for Granular Soil

<u>Relative Density</u>	<u>Standard Penetration Resistance (N-values) blows per foot</u>
very loose	0 - 4
loose	4 - 10
medium dense	10 - 30
dense	30 - 50
very dense	over 50

Description of Consistency for Fine-Grained (Cohesive) Soils

<u>Consistency</u>	<u>Standard Penetration Resistance (N-values) blows per foot</u>	<u>Torvane Undrained Shear Strength, tsf</u>
very soft	2	less than 0.125
soft	2 - 4	0.125 - 0.25
medium stiff	4 - 8	0.25 - 0.50
stiff	8 - 15	0.50 - 1.0
very stiff	15 - 30	1.0 - 2.0
hard	over 30	over 2.0

Sandy silt materials which exhibit general properties of granular soils are given relative density description.

Grain-Size Classification

Modifier for Subclassification

<u>Grain-Size Classification</u>	<u>Adjective</u>	<u>Percentage of Other Material In Total Sample</u>
<i>Boulders</i> 12 - 36 in.		
<i>Cobbles</i> 3 - 12 in.	clean	0 - 1.5
<i>Gravel</i> $\frac{1}{4}$ - $\frac{3}{4}$ in. (fine) $\frac{3}{4}$ - 3 in. (coarse)	trace some	1.5 - 10 10 - 30
<i>Sand</i> No. 200 - No. 40 sieve (fine) No. 40 - No. 10 sieve (medium) No. 10 - No. 4 sieve (coarse)	sandy, silty, clayey, etc.	30 - 50
<i>Silt/Clay</i> - pass No. 200 sieve		

Table 2A

**SOIL CLASSIFICATION
BASED ON CONE PENETRATION TEST**

<u>Friction Ratio (Percent)</u>	<u>Soil Classification</u>
0 to 2	Clean sand or slightly silty sand
2 to 5	Silty sand, clayey sand, or silt
> 5	Clayey silt, silty clay, or clay

COHESIVE SOILS

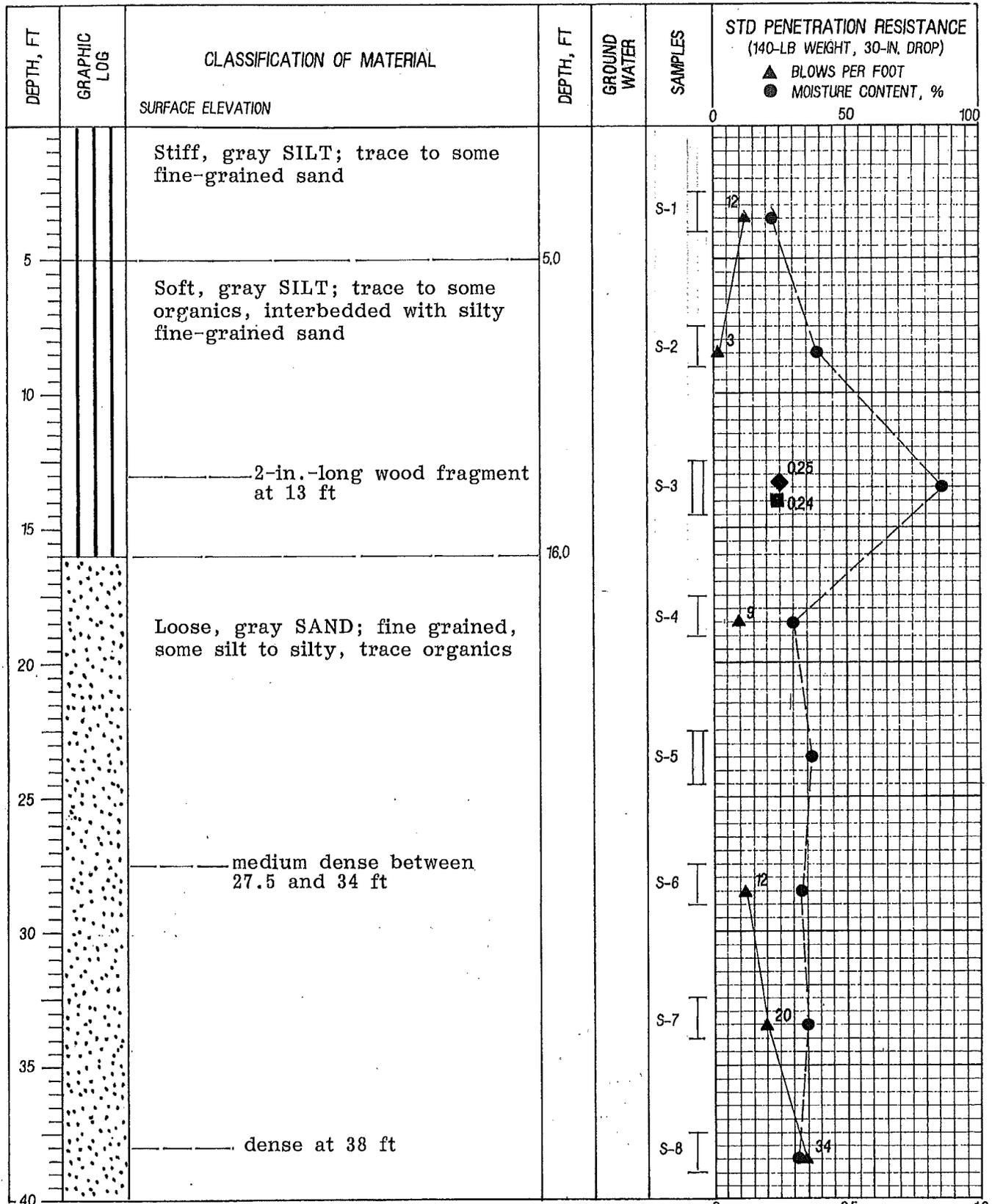
<u>Sleeve Friction, tsf</u>	<u>Relative Consistency</u>
< 0.12	Very Soft
0.12 to 0.25	Soft
0.25 to 0.50	Medium Stiff
0.50 to 1.00	Stiff
1.00 to 2.00	Very Stiff
> 2.00	Hard

COHESIONLESS SOILS

<u>Relative Density</u>	<u>Soil Type*</u>			
	<u>ML, SM</u>	<u>SM, SP, SW</u>	<u>SP, SW, GW</u>	<u>SW, GP</u>
	<u>Cone Penetration Resistance, tsf</u>			
Very Loose	0 - 8	0 - 14	0 - 20	0 - 24
Loose	8 - 20	14 - 35	20 - 50	24 - 60
Med. Dense	20 - 60	35 - 105	50 - 150	60 - 180
Dense	60 - 100	105 - 175	150 - 250	180 - 300
Very Dense	> 100	> 175	> 250	> 300

* Unified Soil Classification System

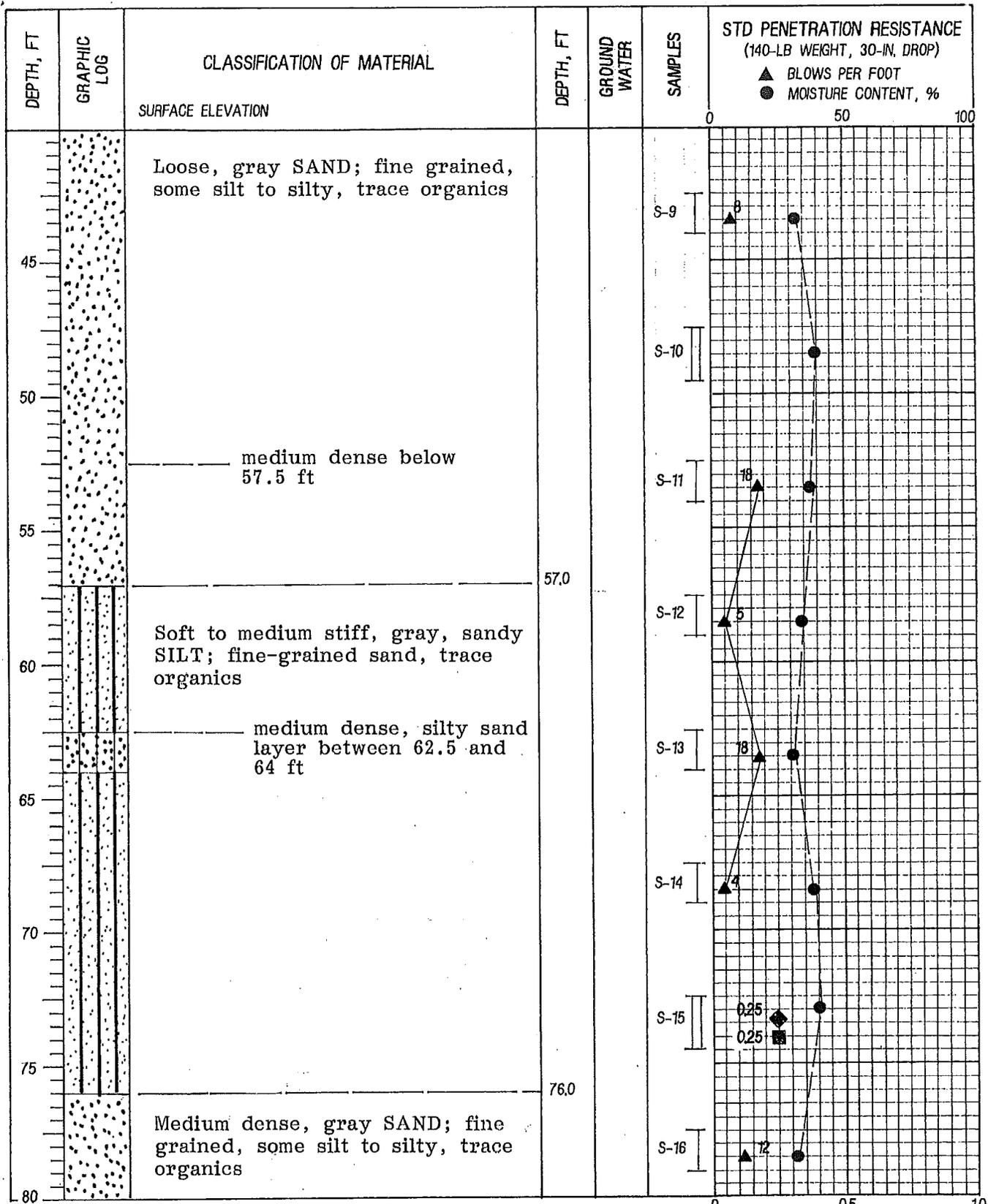
- 1) Friction ratio is equal to sleeve friction (tsf) divided by cone penetration (tsf) expressed as a percent.
- 2) Cone penetration test performed and interpreted by Subterranean, Inc. of Sumner, Washington.



- I 2-IN.-OD SPLIT-SPOON SAMPLER
- II 3-IN.-OD THIN-WALLED SAMPLER
- G GRAB SAMPLE OF DRILL CUTTINGS
- NX CORE RUN
- ▬ SLOTTED PVC PIPE
- ▼ WATER LEVEL (DATE)
- ◆ TORVANE SHEAR STRENGTH, TSF
- UNDRAINED SHEAR STRENGTH, TSF
- LIQUID LIMIT
- MOISTURE CONTENT
- PLASTIC LIMIT



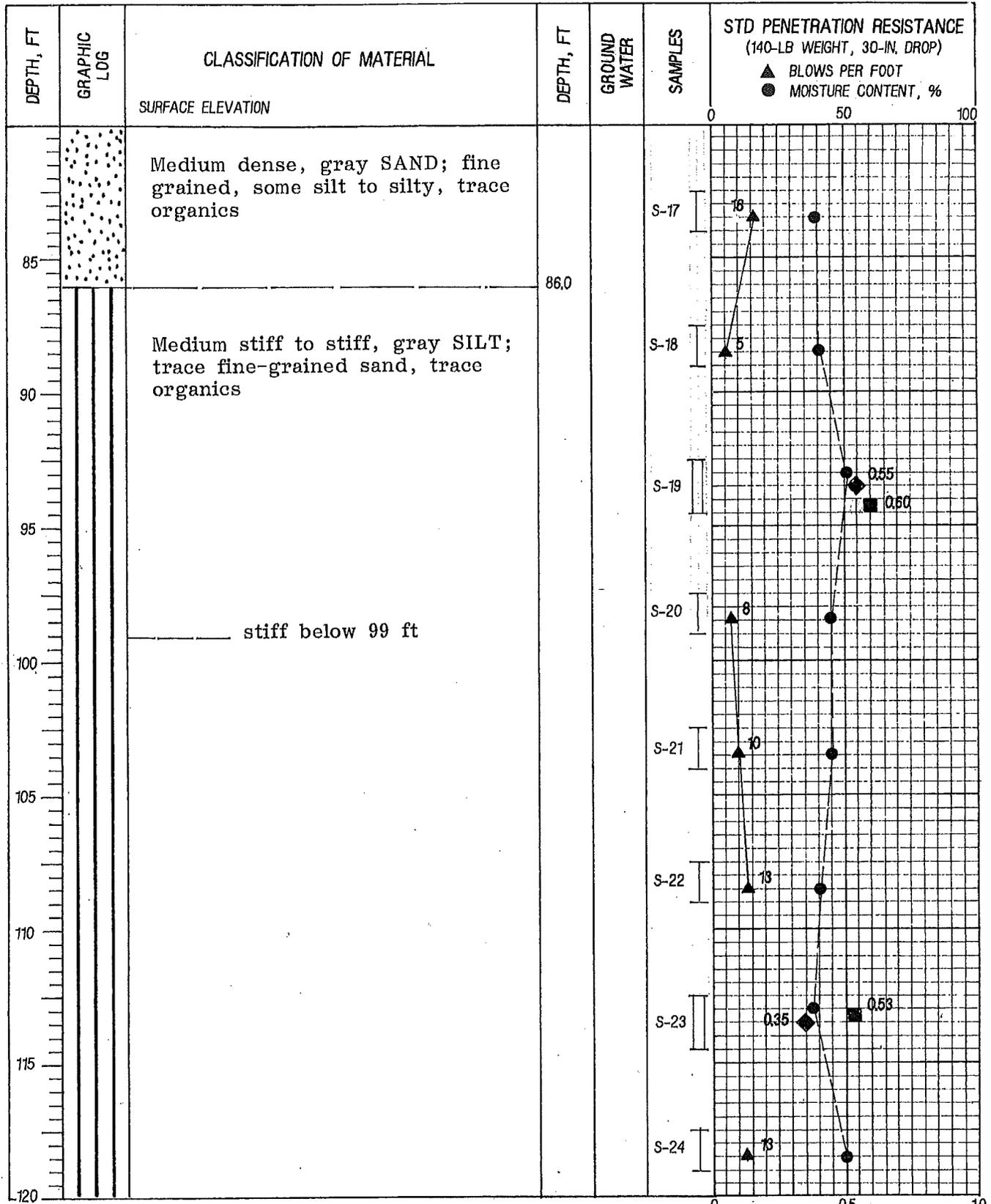
BORING B-1



- I 2-IN.-OD SPLIT-SPOON SAMPLER
- II 3-IN.-OD THIN-WALLED SAMPLER
- G GRAB SAMPLE OF DRILL CUTTINGS
- NX CORE RUN
- SLOTTED PVC PIPE
- ▼ WATER LEVEL (DATE)
- ◆ TORVANE SHEAR STRENGTH, TSF
- UNDRAINED SHEAR STRENGTH, TSF
- LIQUID LIMIT MOISTURE CONTENT
- PLASTIC LIMIT



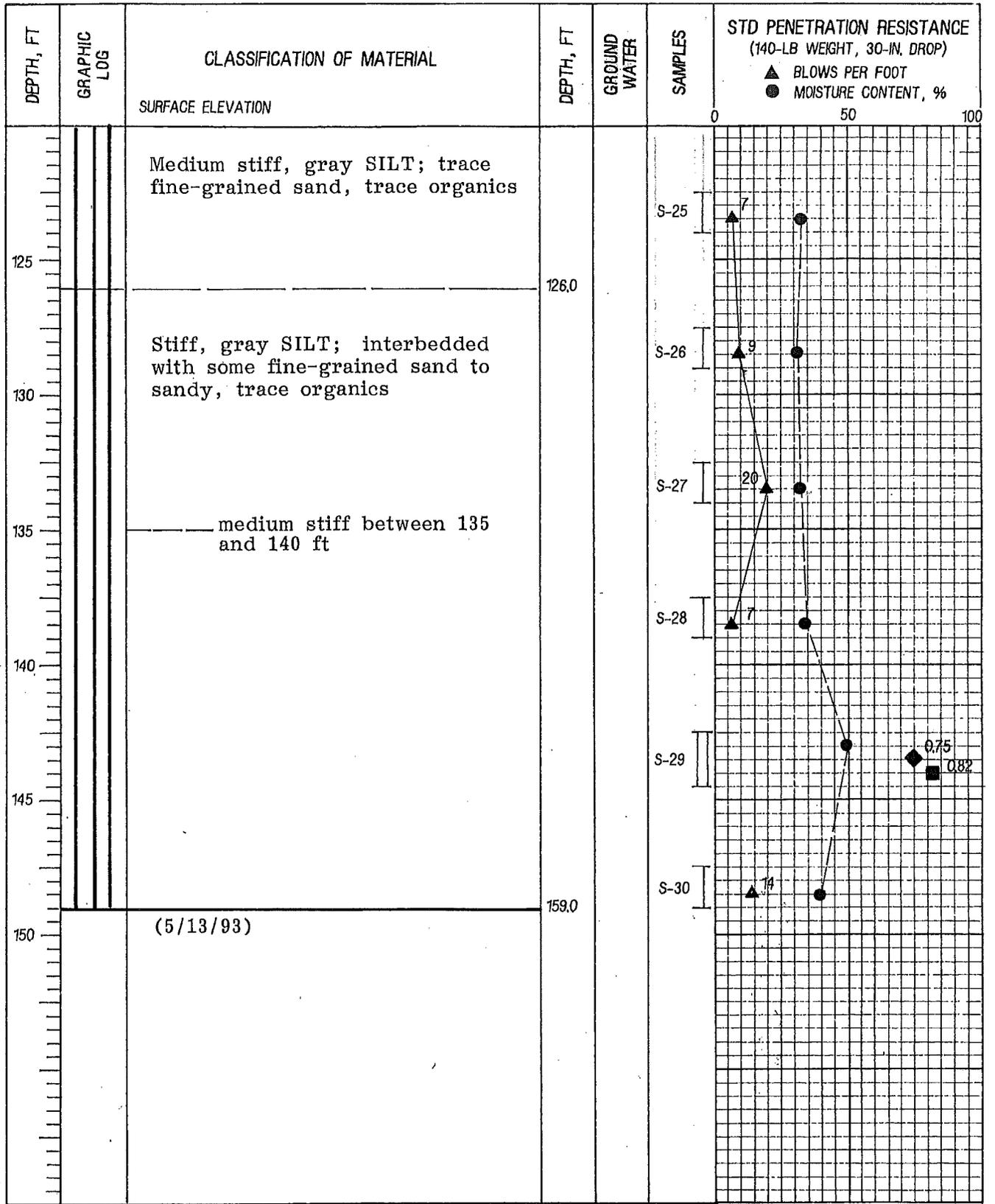
BORING B-1 (CONT.)



- I 2-IN.-OD SPLIT-SPOON SAMPLER
- II 3-IN.-OD THIN-WALLED SAMPLER
- G GRAB SAMPLE OF DRILL CUTTINGS
- NX CORE RUN
- SLOTTED PVC PIPE
- ▼ WATER LEVEL (DATE)
- ◆ TORVANE SHEAR STRENGTH, TSF
- UNDRAINED SHEAR STRENGTH, TSF
- LIQUID LIMIT
- MOISTURE CONTENT
- PLASTIC LIMIT



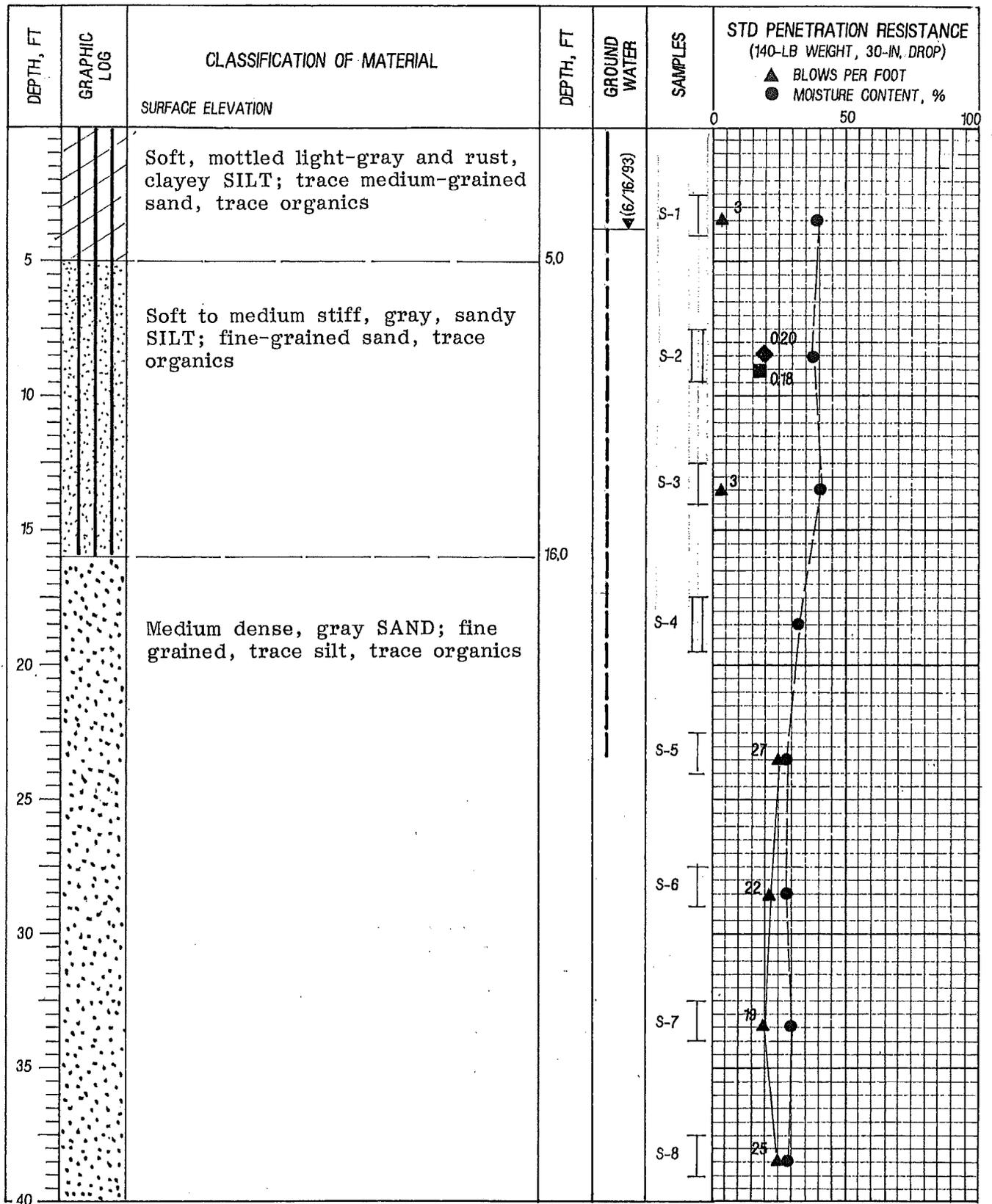
BORING B-1 (CONT.)



- I 2-IN.-OD SPLIT-SPOON SAMPLER
- II 3-IN.-OD THIN-WALLED SAMPLER
- G GRAB SAMPLE OF DRILL CUTTINGS
- NX CORE RUN
- SLOTTED PVC PIPE
- ▼ WATER LEVEL (DATE)
- ◆ TORVANE SHEAR STRENGTH, TSF
- UNDRAINED SHEAR STRENGTH, TSF
- LIQUID LIMIT
- MOISTURE CONTENT
- PLASTIC LIMIT



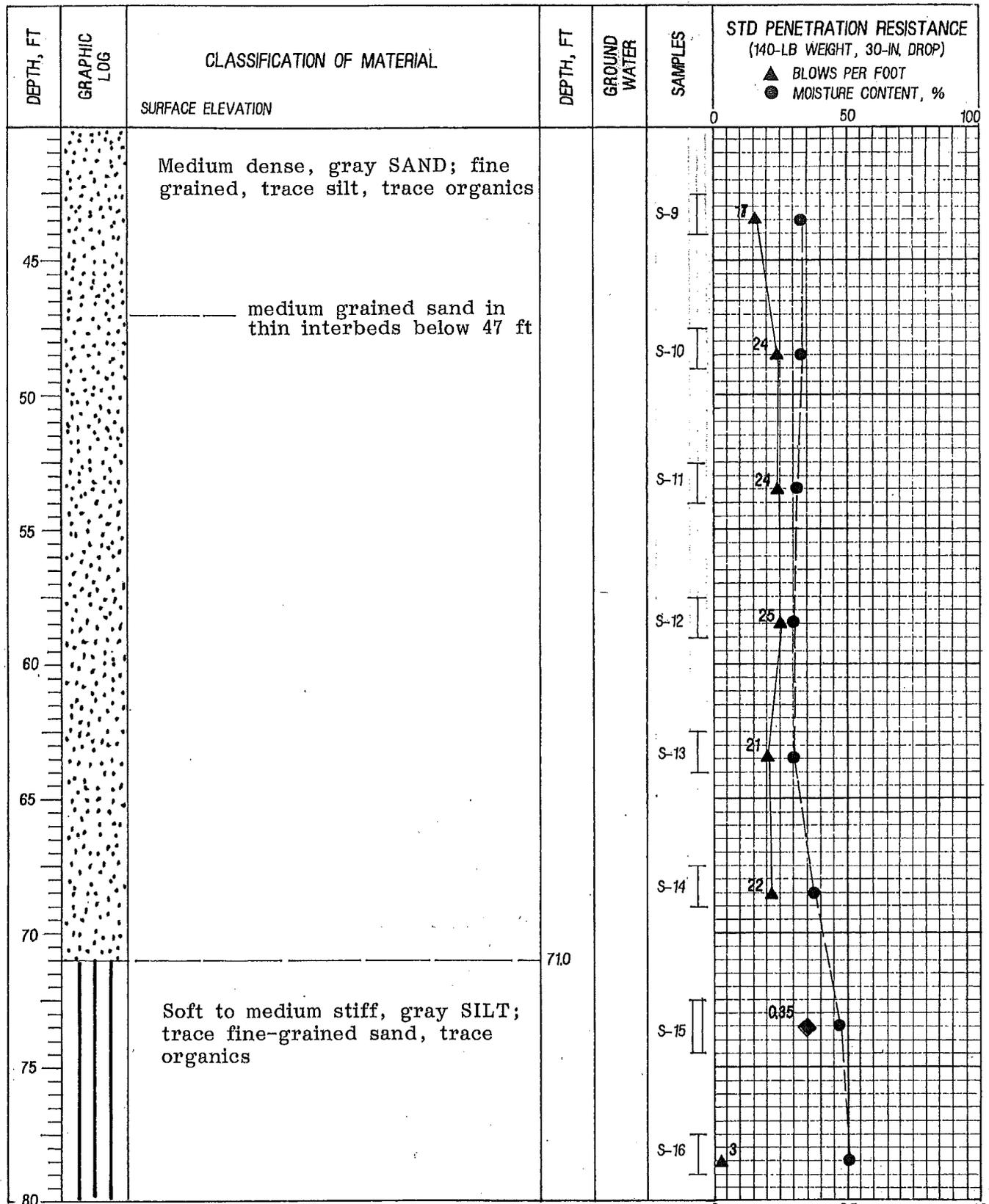
BORING B-1 (CONT.)



- I 2-IN.-OD SPLIT-SPOON SAMPLER
- II 3-IN.-OD THIN-WALLED SAMPLER
- G GRAB SAMPLE OF DRILL CUTTINGS
- NX CORE RUN
- SLOTTED PVC PIPE
- ▼ WATER LEVEL (DATE)
- ◆ TORVANE SHEAR STRENGTH, TSF
- UNDRAINED SHEAR STRENGTH, TSF
- LIQUID LIMIT
- MOISTURE CONTENT
- PLASTIC LIMIT



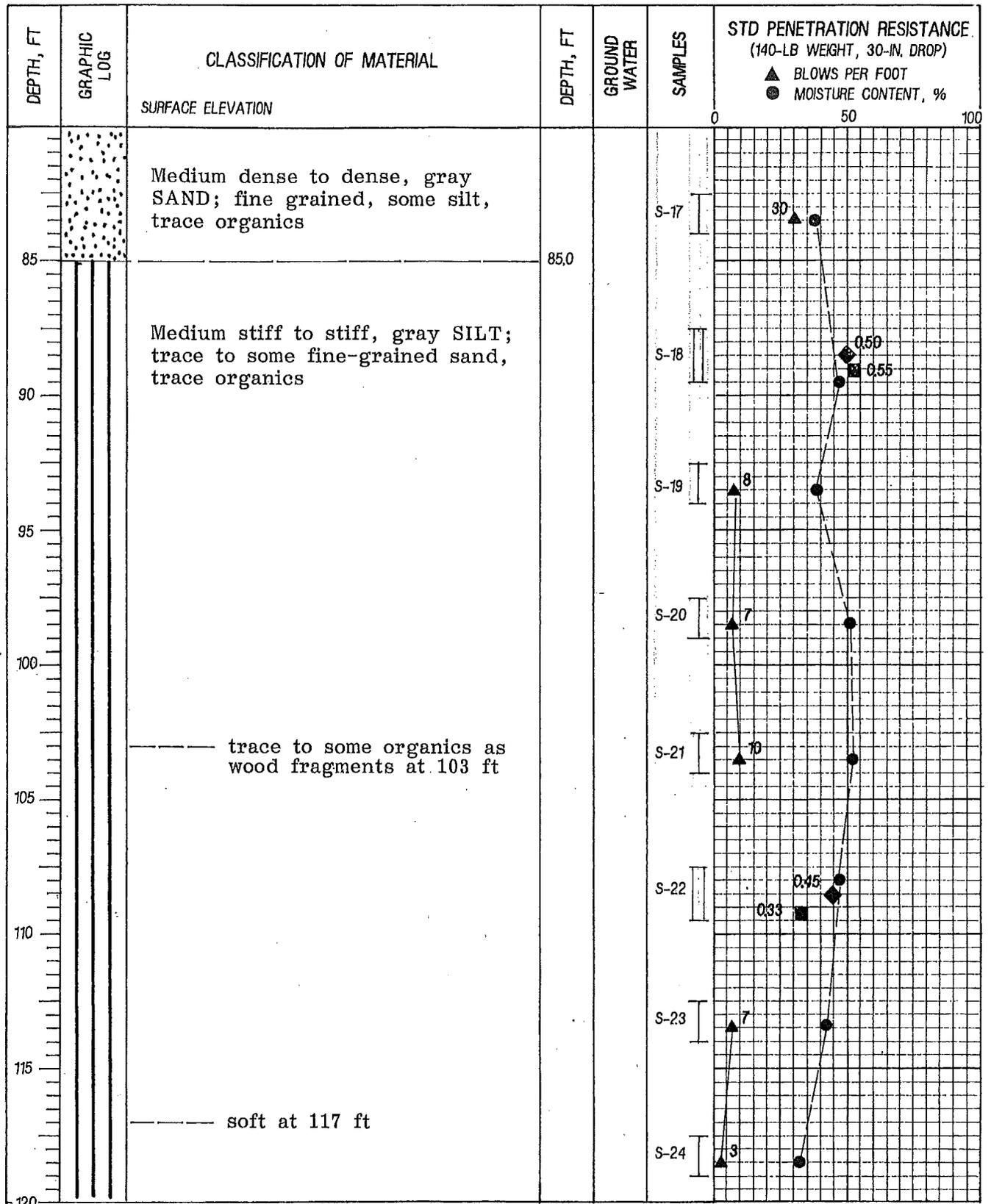
BORING B-2



- I 2-IN.-OD SPLIT-SPOON SAMPLER
- II 3-IN.-OD THIN-WALLED SAMPLER
- G. GRAB SAMPLE OF DRILL CUTTINGS
- NX CORE RUN
- ▬ SLOTTED PVC PIPE
- ▼ WATER LEVEL (DATE)
- ◆ TORVANE SHEAR STRENGTH, TSF
- UNDRAINED SHEAR STRENGTH, TSF
- LIQUID LIMIT
- MOISTURE CONTENT
- PLASTIC LIMIT



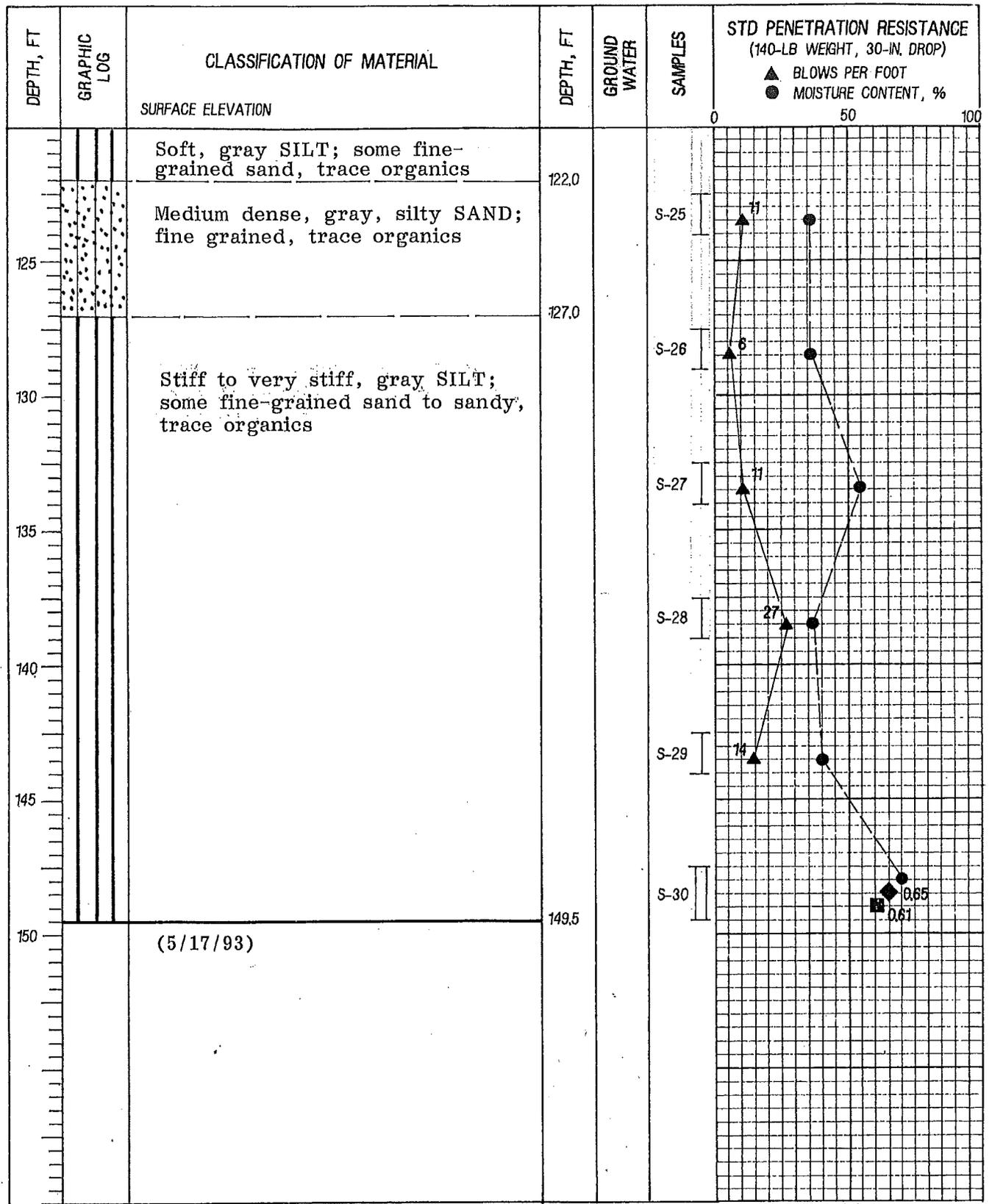
BORING B-2 (CONT.)



- I 2-IN.-OD SPLIT-SPOON SAMPLER
- II 3-IN.-OD THIN-WALLED SAMPLER
- G GRAB SAMPLE OF DRILL CUTTINGS
- NX CORE RUN
- SLOTTED PVC PIPE
- ▼ WATER LEVEL (DATE)
- ◆ TORVANE SHEAR STRENGTH, TSF
- UNDRAINED SHEAR STRENGTH, TSF
- LIQUID LIMIT MOISTURE CONTENT
- PLASTIC LIMIT



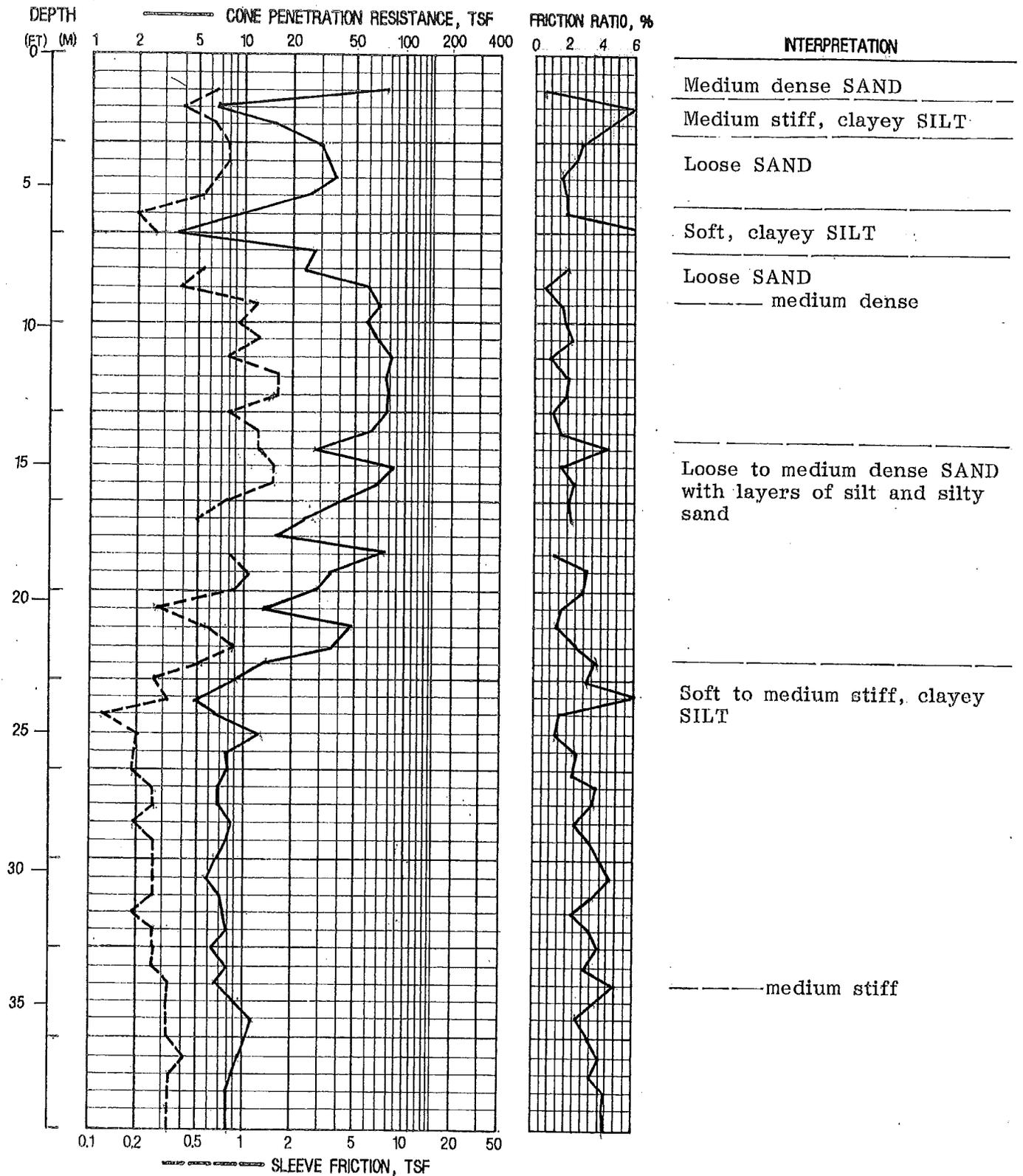
BORING B-2 (CONT.)



- I 2-IN.-OD SPLIT-SPOON SAMPLER
- II 3-IN.-OD THIN-WALLED SAMPLER
- G GRAB SAMPLE OF DRILL CUTTINGS
- NX CORE RUN
- SLOTTED PVC PIPE
- ▼ WATER LEVEL (DATE)
- ◆ TORVANE SHEAR STRENGTH, TSF
- UNDRAINED SHEAR STRENGTH, TSF
- LIQUID LIMIT
- MOISTURE CONTENT
- PLASTIC LIMIT



BORING B-2 (CONT.)

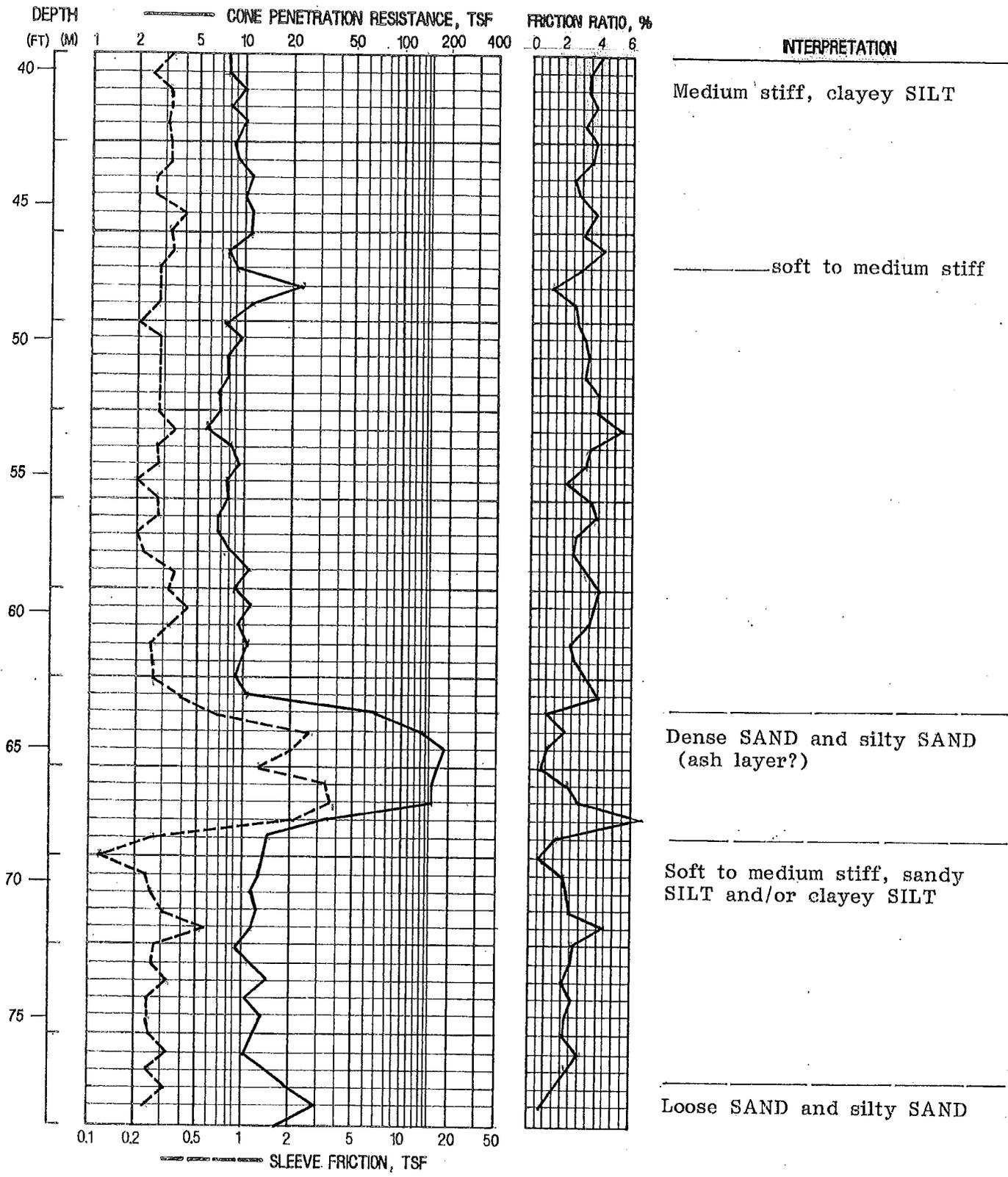


Friction Ratio is equal to sleeve friction divided by cone penetration, expressed as a percent

Test performed and interpreted by
Subterranean, Inc. of Sumner Washington



CPT PROBE P-1

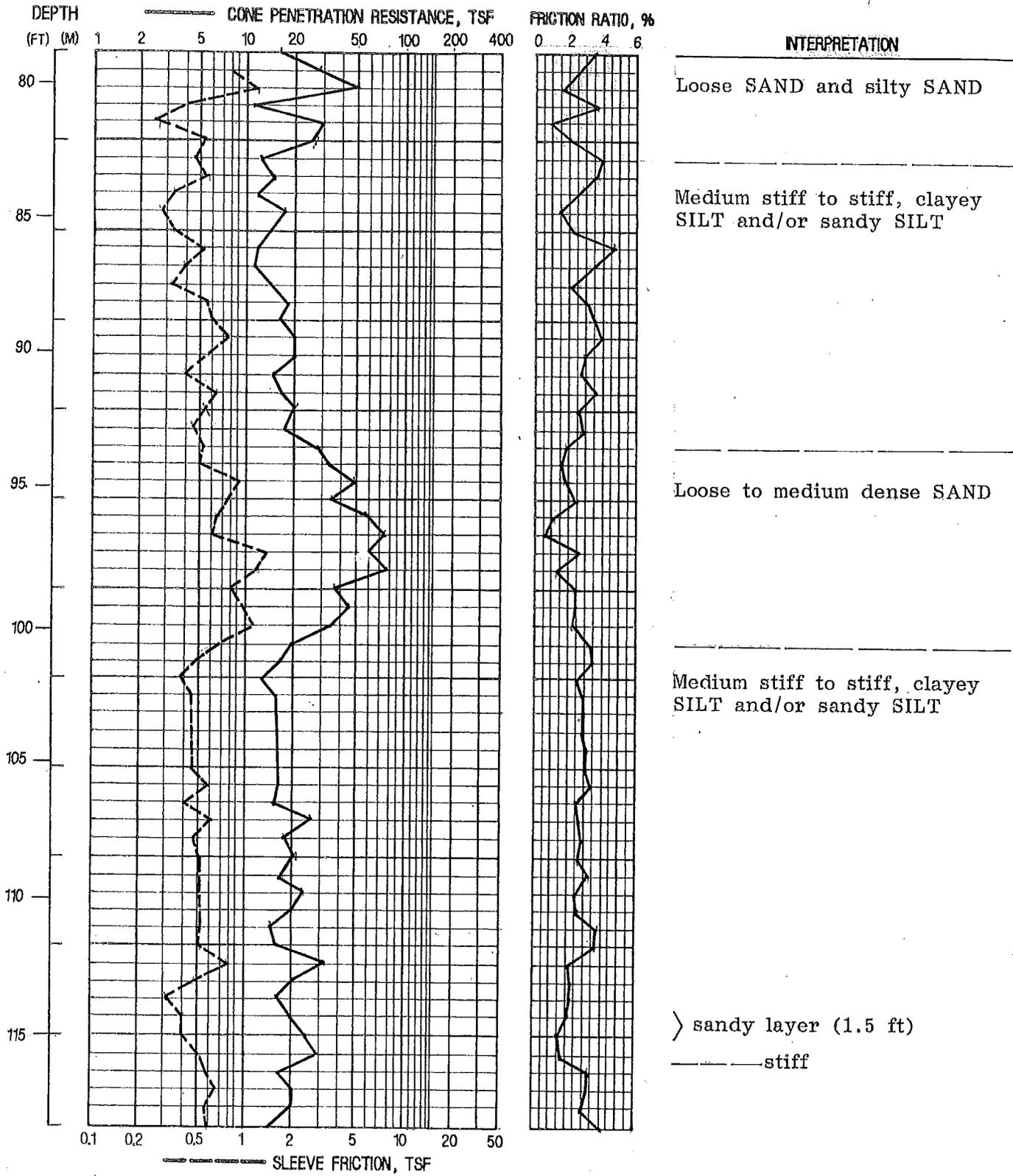


Friction Ratio is equal to sleeve friction divided by cone penetration, expressed as a percent

Test performed and interpreted by
Subterranean, Inc. of Sumner Washington



CPT PROBE P-1 (CONT.)

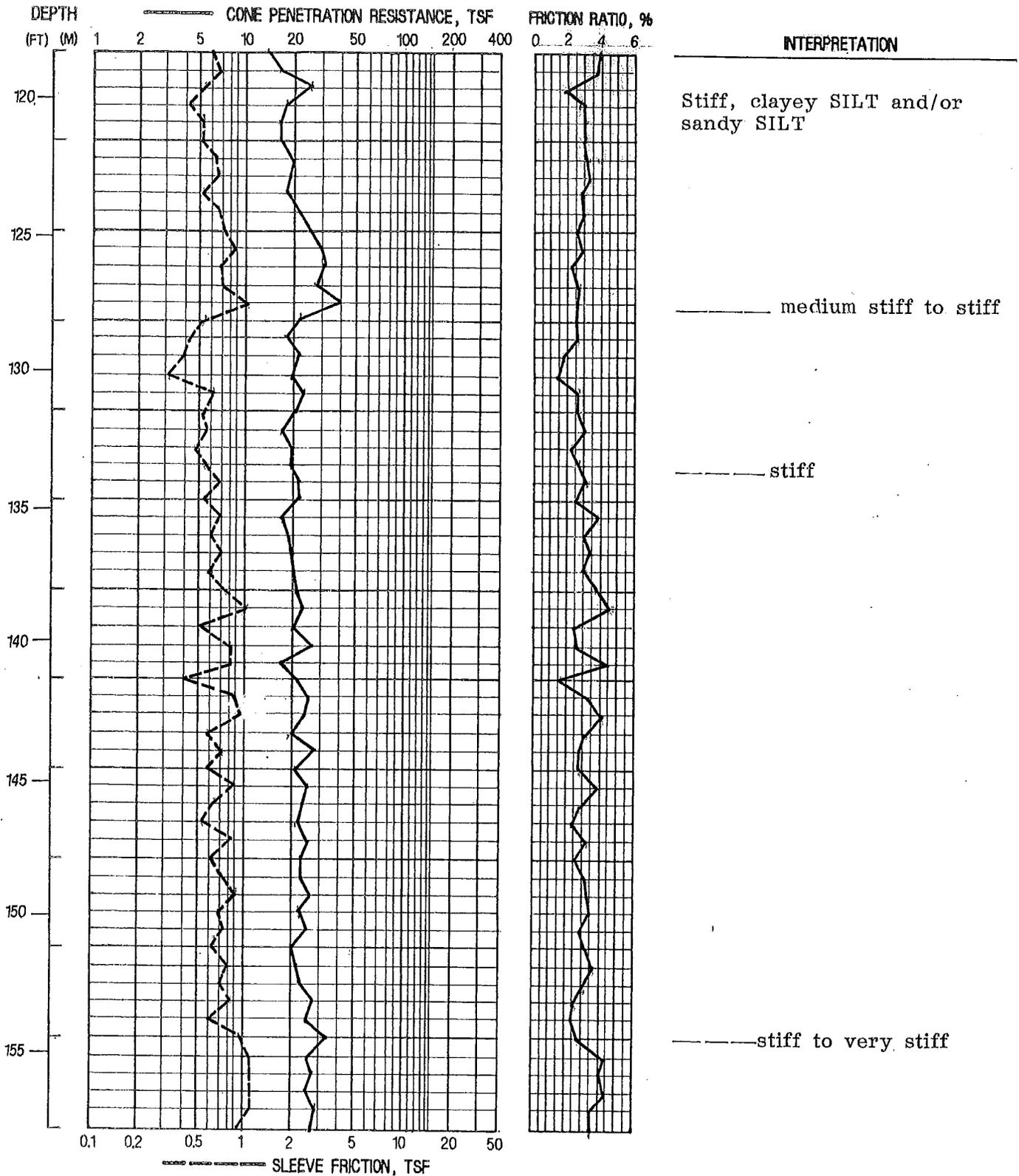


Friction Ratio is equal to sleeve friction divided by cone penetration, expressed as a percent

*Test performed and interpreted by
Subterranean, Inc. of Sumner Washington*



CPT PROBE P-1 (CONT.)

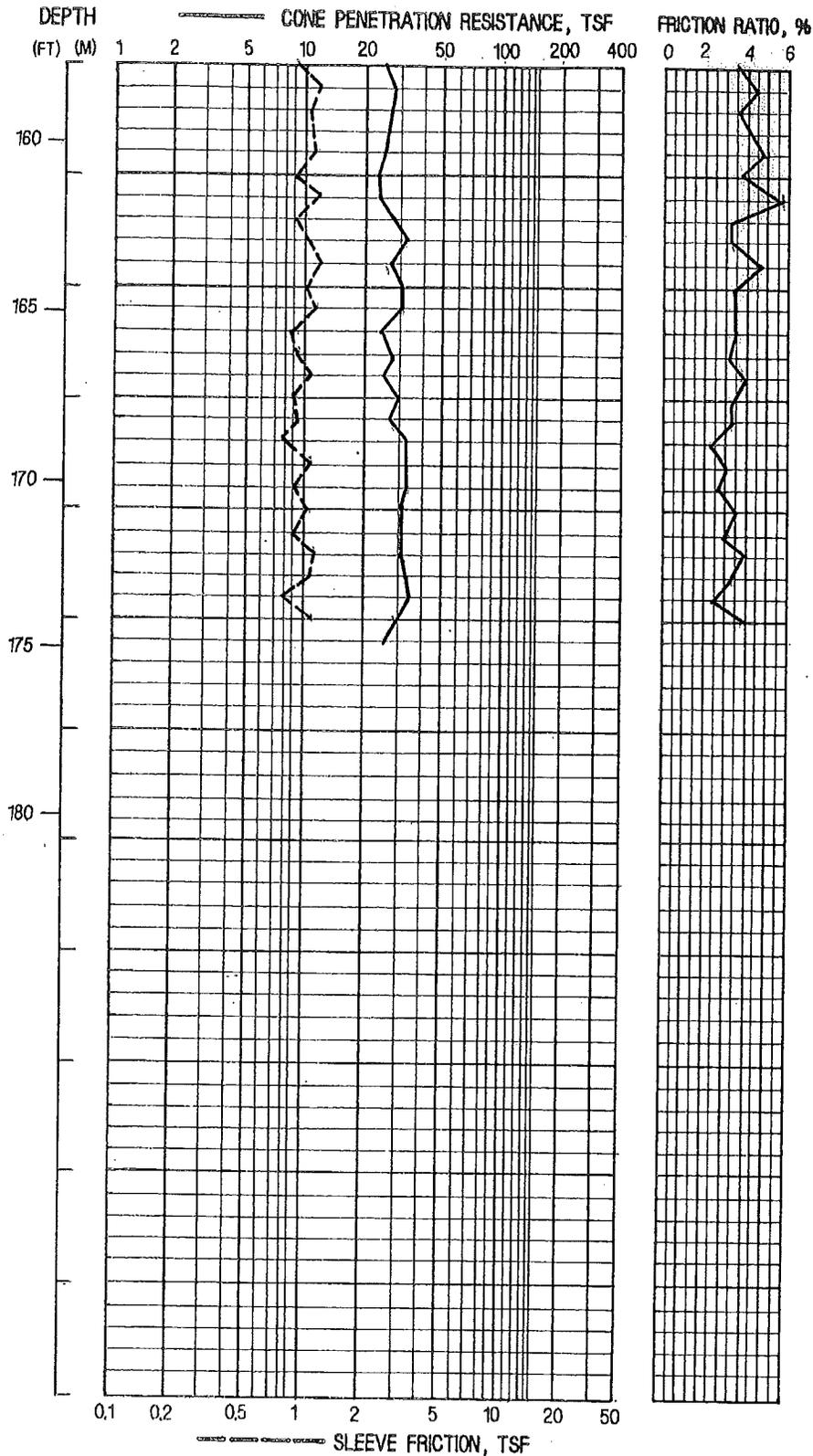


Friction Ratio is equal to sleeve friction divided by cone penetration, expressed as a percent

Test performed and interpreted by
Subterranean, Inc. of Sumner Washington



CPT PROBE P-1 (CONT.)



INTERPRETATION

Stiff to very stiff, clayey SILT and/or sandy SILT

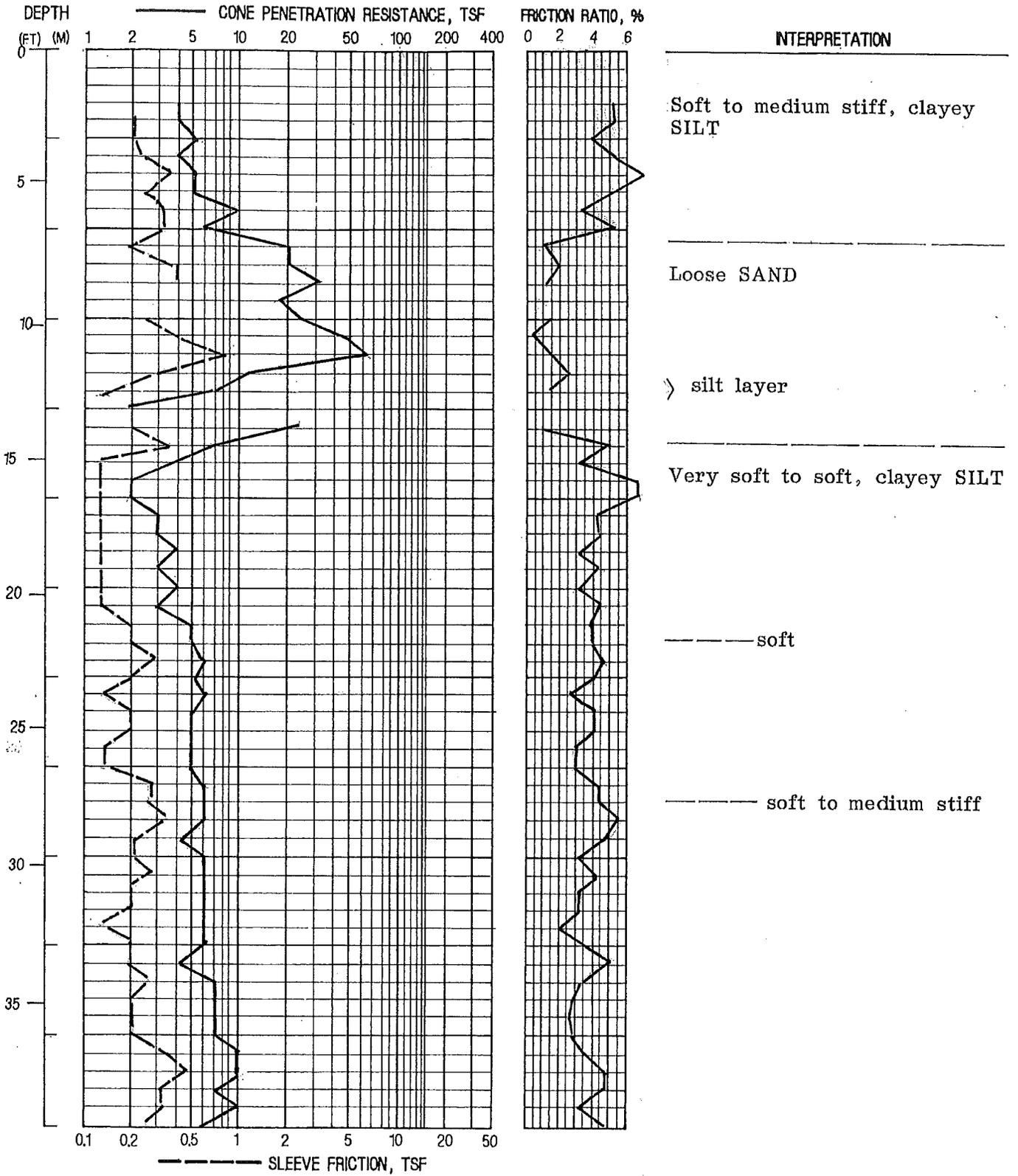
Bottom of probe 175 ft (5/18/93)

Friction Ratio is equal to sleeve friction divided by cone penetration, expressed as a percent

*Test performed and interpreted by
Subterranean, Inc. of Sumner Washington*



CPT PROBE P-1 (CONT.)

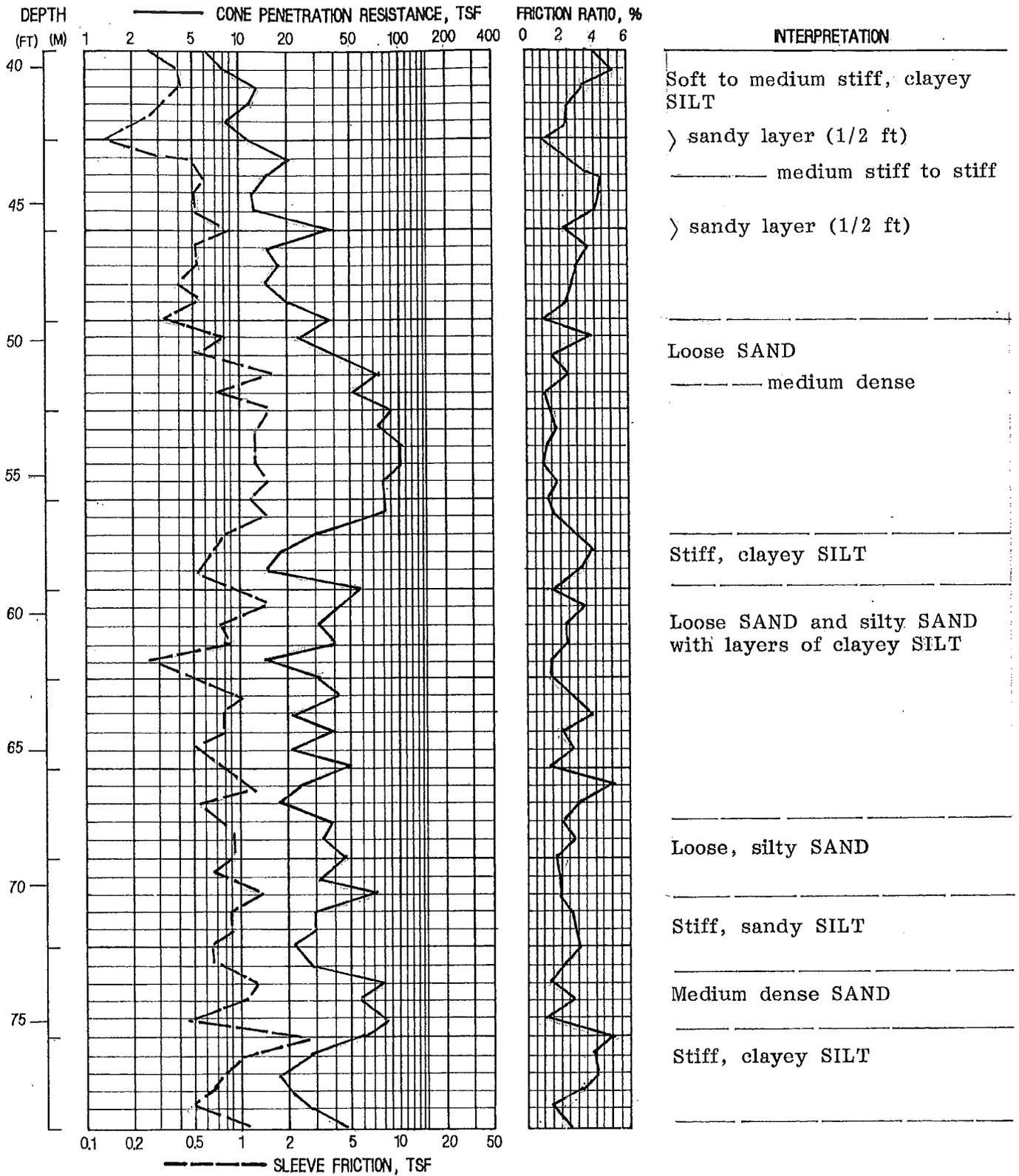


Friction Ratio is equal to sleeve friction divided by cone penetration, expressed as a percent

*Test performed and interpreted by
Subterranean, Inc. of Sumner Washington*



CPT PROBE P-2

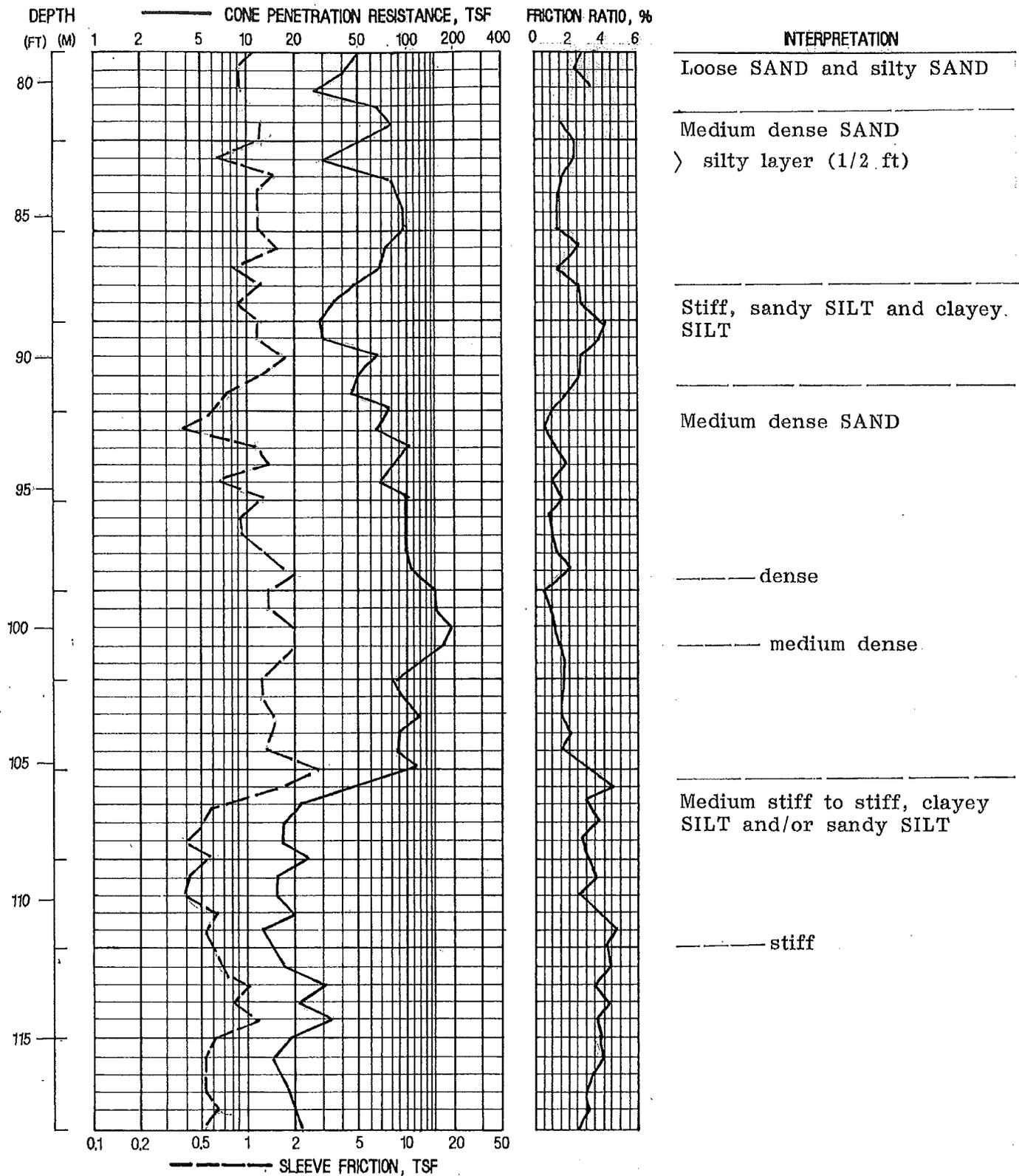


Friction Ratio is equal to sleeve friction divided by cone penetration, expressed as a percent

Test performed and interpreted by Subterranean, Inc. of Sumner Washington



CPT PROBE P-2 (CONT.)

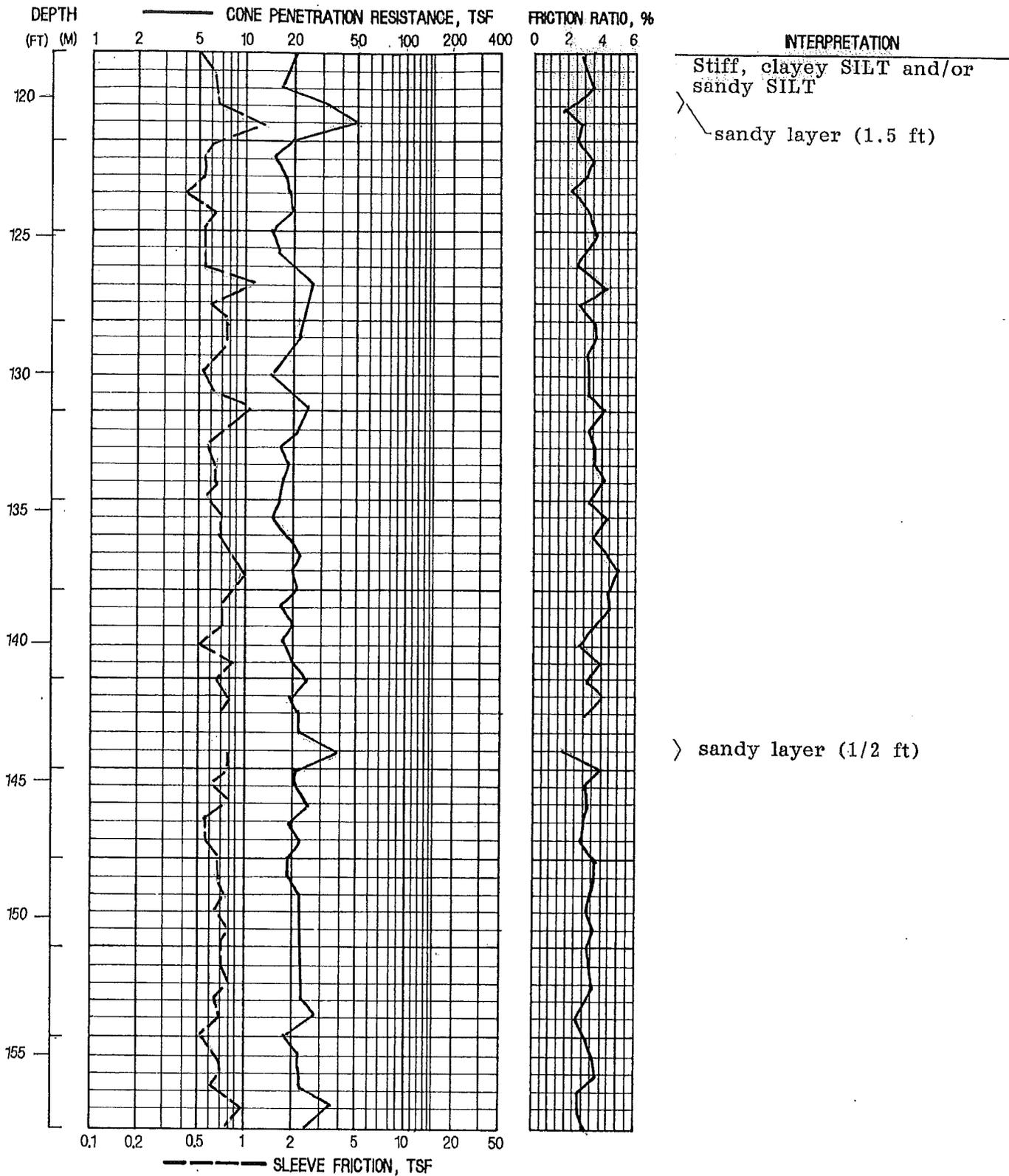


Friction Ratio is equal to sleeve friction divided by cone penetration, expressed as a percent

Test performed and interpreted by
Subterranean, Inc. of Sumner Washington



CPT PROBE P-2 (CONT.)

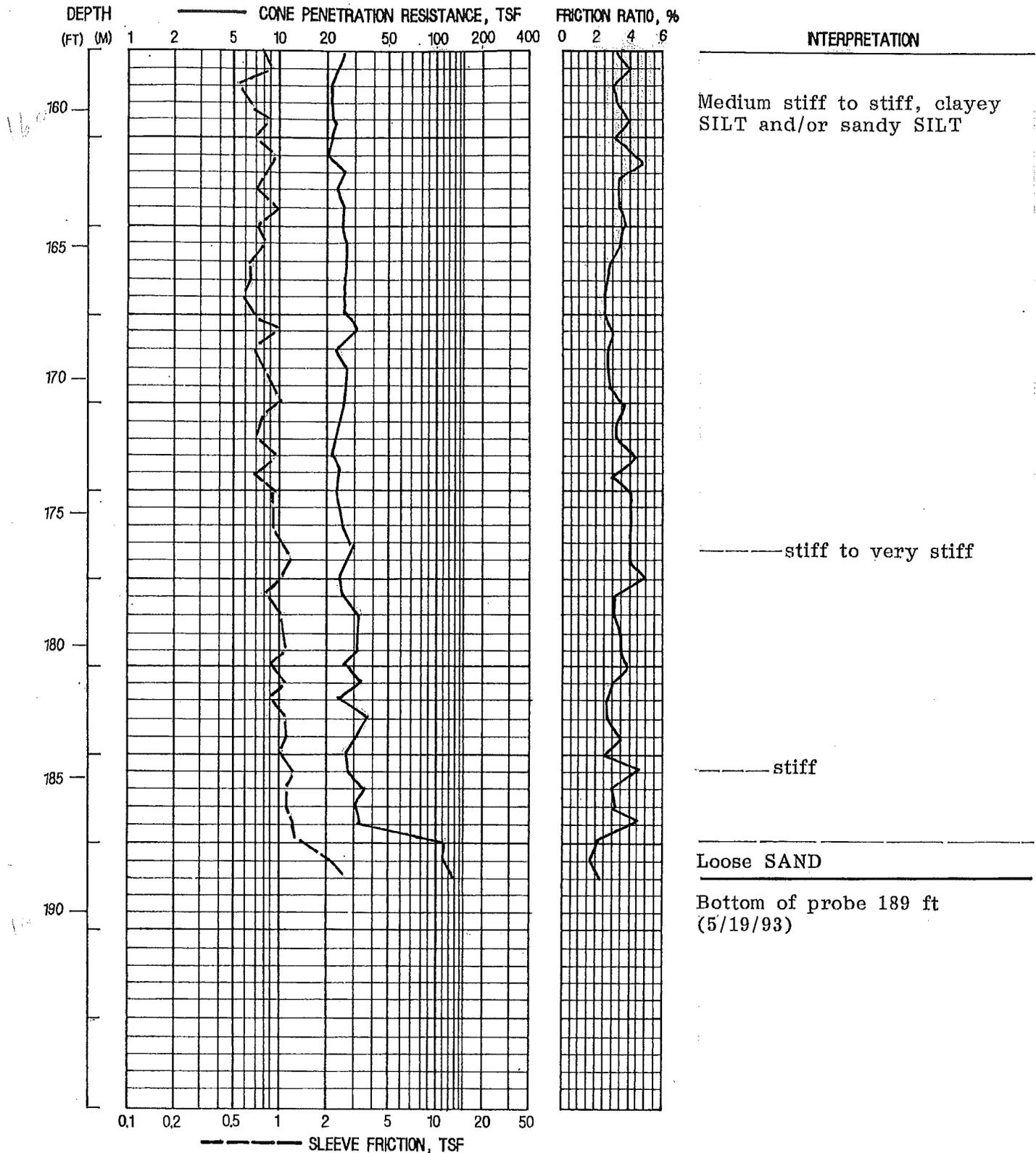


Friction Ratio is equal to sleeve friction divided by cone penetration, expressed as a percent

*Test performed and interpreted by
Subterranean, Inc. of Sumner Washington*



CPT PROBE P-2 (CONT.)



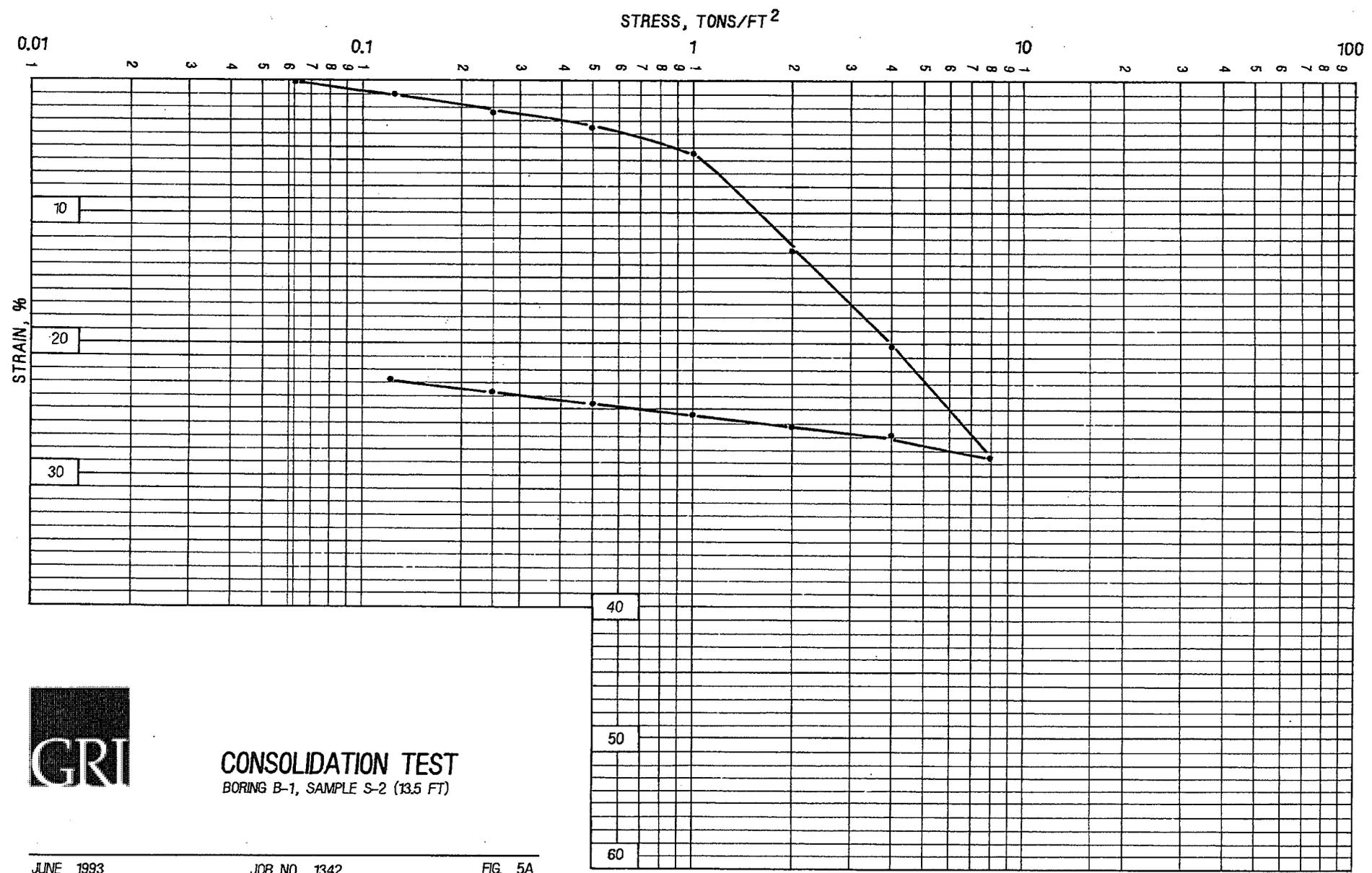
Friction Ratio is equal to sleeve friction divided by cone penetration, expressed as a percent

Test performed and interpreted by Subterranean, Inc. of Sumner Washington



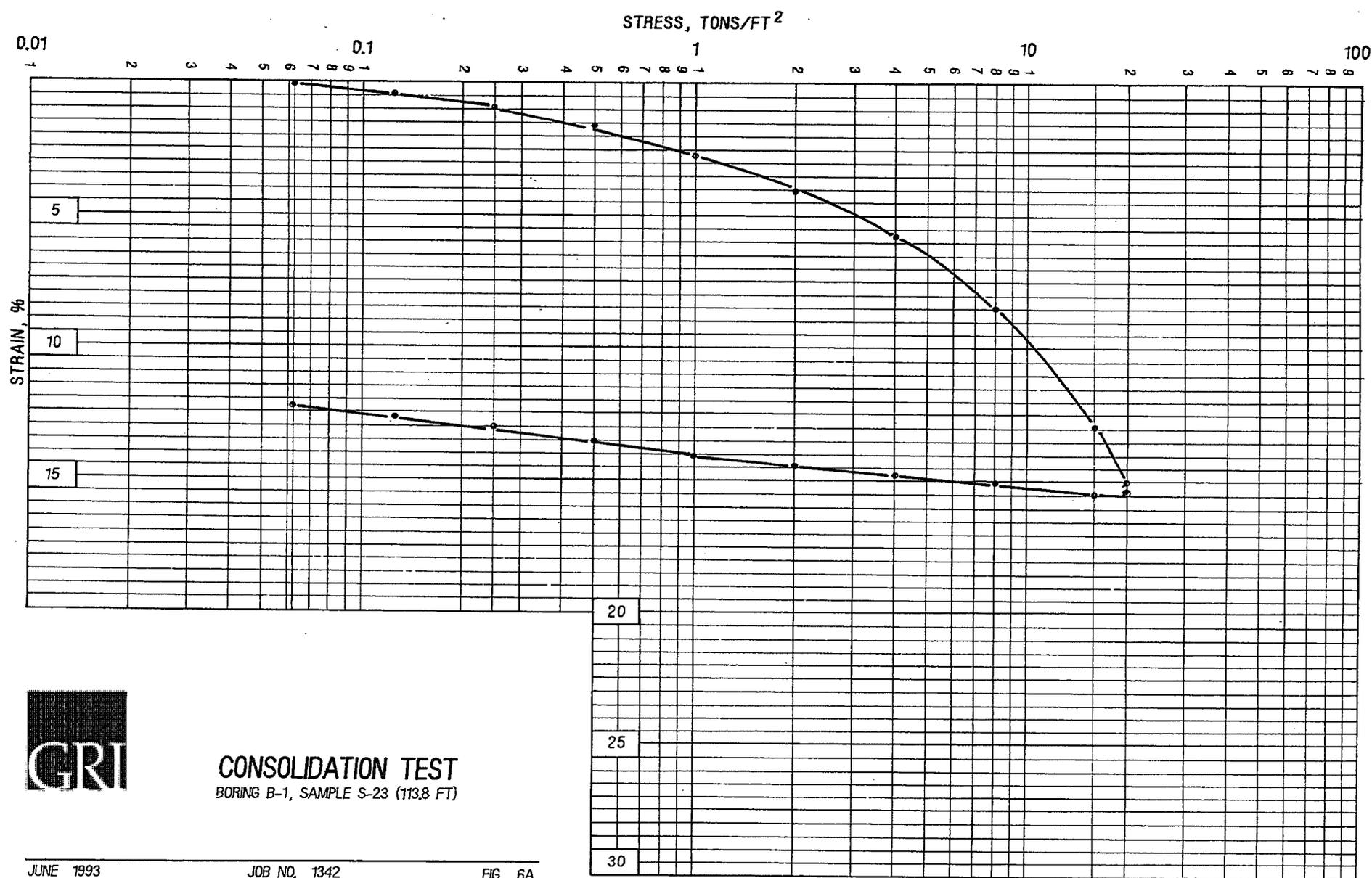
CPT PROBE P-2 (CONT.)

BORING	SAMPLE	DEPTH, FT	MOISTURE CONTENT, %		DRY UNIT WEIGHT, PCF		SOIL DESCRIPTION
			(INITIAL)	(FINAL)	(INITIAL)	(FINAL)	
B-1	S-3	13.5	86	61	50	66	SOFT, GRAY SILT; TRACE TO SOME CLAY AND ORGANICS



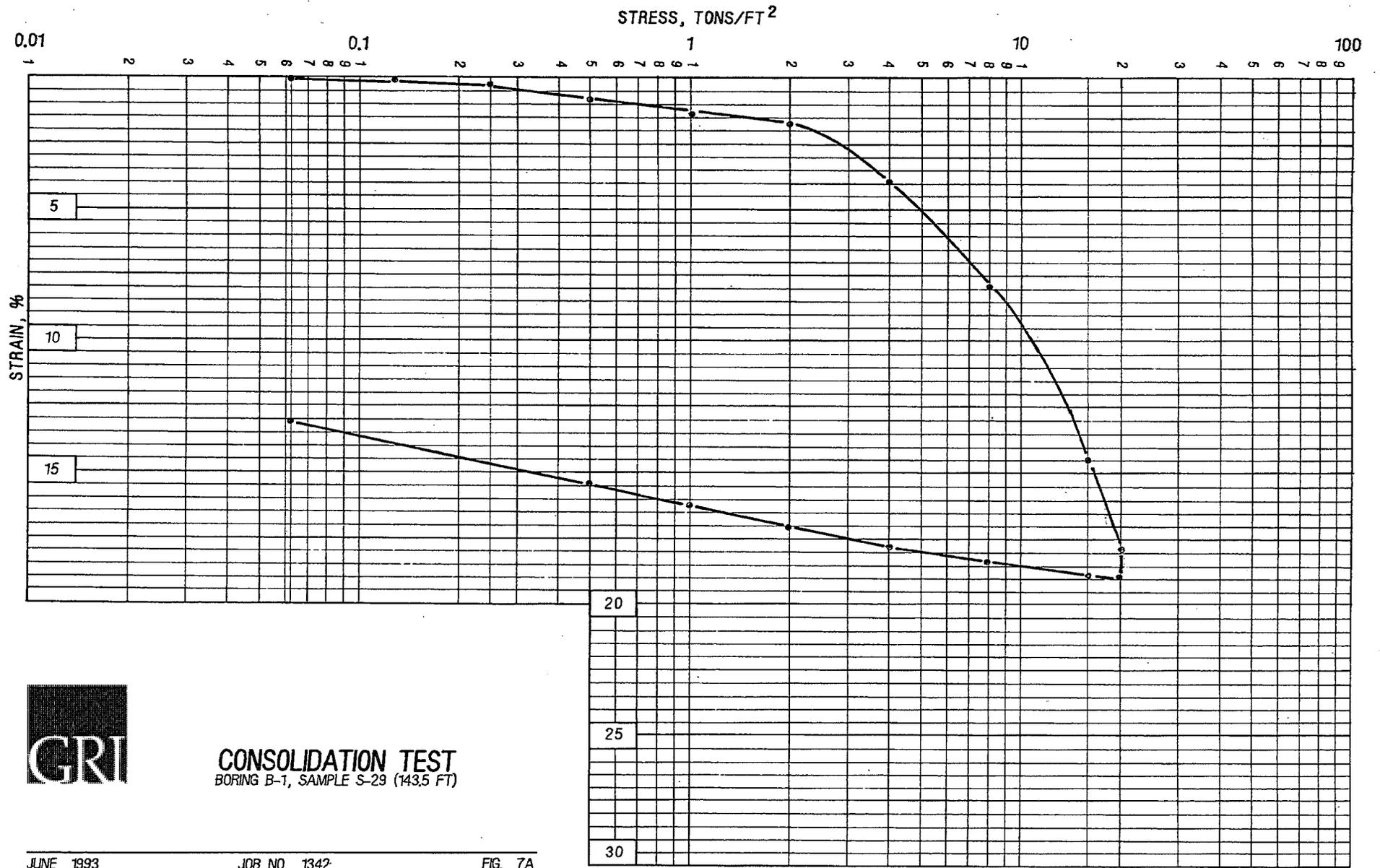
CONSOLIDATION TEST
BORING B-1, SAMPLE S-2 (13.5 FT)

BORING	SAMPLE	DEPTH, FT	MOISTURE CONTENT, %		DRY UNIT WEIGHT, PCF		SOIL DESCRIPTION
			(INITIAL)	(FINAL)	(INITIAL)	(FINAL)	
B-1	S-23	113.8	38	32	82	93	STIFF, GRAY SILT; TRACE FINE SAND



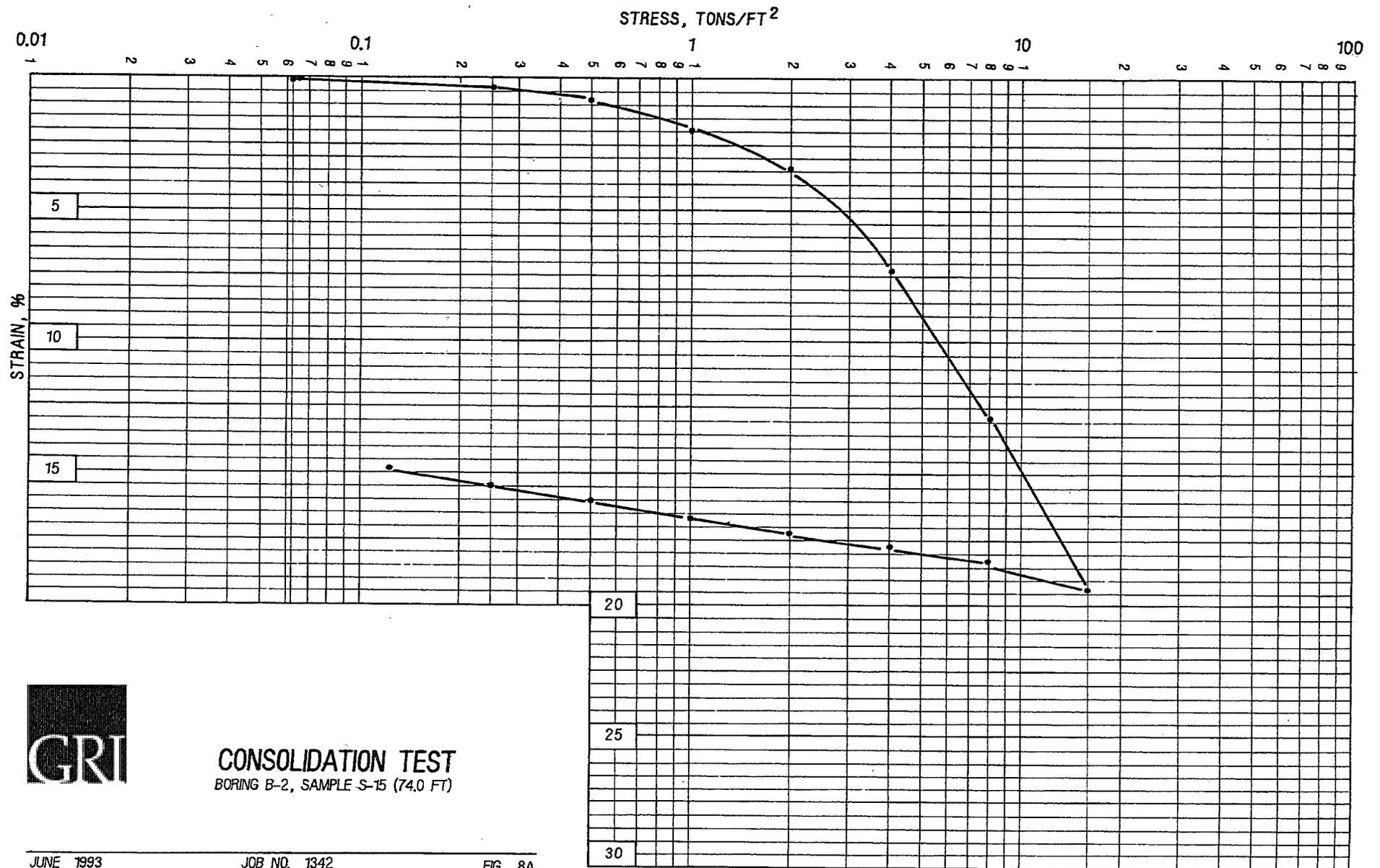
CONSOLIDATION TEST
BORING B-1, SAMPLE S-23 (113.8 FT)

BORING	SAMPLE	DEPTH, FT	MOISTURE CONTENT, %		DRY UNIT WEIGHT, PCF		SOIL DESCRIPTION
			(INITIAL)	(FINAL)	(INITIAL)	(FINAL)	
B-1	S-29	143.5	50	42	69	79	STIFF, GRAY SILT



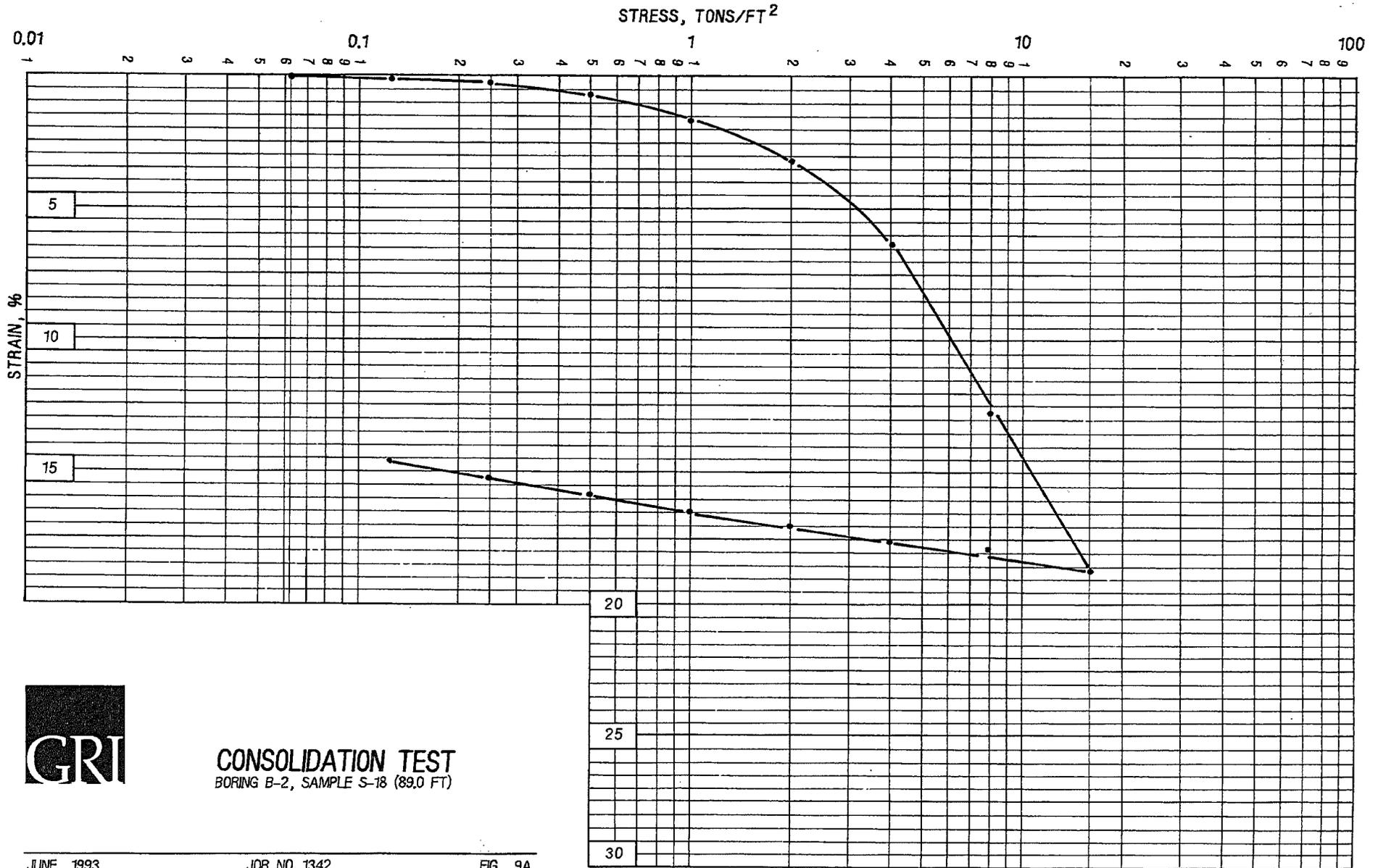
CONSOLIDATION TEST
BORING B-1, SAMPLE S-29 (143.5 FT)

BORING	SAMPLE	DEPTH, FT	MOISTURE CONTENT, %		DRY UNIT WEIGHT, PCF		SOIL DESCRIPTION
			(INITIAL)	(FINAL)	(INITIAL)	(FINAL)	
B-2	S-15	74.0	47	38	73	85	MEDIUM STIFF, GRAY SILT; SOME FINE SAND, TRACE ORGANICS



CONSOLIDATION TEST
BORING B-2, SAMPLE S-15 (74.0 FT)

BORING	SAMPLE	DEPTH, FT	MOISTURE CONTENT, %		DRY UNIT WEIGHT, PCF		SOIL DESCRIPTION
			(INITIAL)	(FINAL)	(INITIAL)	(FINAL)	
B-2	S-18	89.0	47	37	74	86	MEDIUM STIFF TO STIFF, GRAY SILT; TRACE TO SOME FINE SAND, TRACE ORGANICS



CONSOLIDATION TEST
BORING B-2, SAMPLE S-18 (89.0 FT)