Draft Geotechnical Report
USD Crude-by-Rail Terminal
Port of Grays Harbor
Hoquiam, Washington

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DRAFT GEOTECHNICAL REPORT
USD CRUDE-BY-RAIL TERMINAL
PORT OF GRAYS HARBOR
HOQUIAM, WASHINGTON

1.0 INTRODUCTION

This draft geotechnical report presents the results of our geotechnical explorations, laboratory testing performed, and preliminary geotechnical engineering analysis, for the proposed US Development (USD) Crude-by-Rail terminal project at the Port of Grays Harbor, Hoquiam, Washington. The proposed project is located about 1.5 miles west of downtown Hoquiam, near an existing wood pulp mill and export terminal (Terminal 3) on the north shore of Grays Harbor, as shown in Figure 1.

Included in this draft report are a site and project description, results of our geotechnical investigations and laboratory testing, a description of the subsurface soil and groundwater conditions, and preliminary results of geotechnical engineering analysis. Geotechnical recommendations for final design will be presented in our final geotechnical report after we receive comments and questions from the design team. Additionally, we will present geotechnical data and design recommendations for the proposed new dolphins to be installed at the end of the pier at Terminal 3 during the final design phase of the project.

The services described in this report were conducted in general accordance with the scope of services as outlined in our subcontract with HDR that was negotiated and signed in March 2013.

2.0 SITE AND PROJECT DESCRIPTION

USD intends to develop the site as an oil export terminal supplied by an existing rail network. The project site location is shown on the Vicinity Map, Figure 1. In general, the USD crude-by-rail terminal consists of constructing eight oil storage tanks, railroad embankments for a new spur line track and ladder siding, an operations and pump station, an aboveground delivery pipe from the tank site to the existing pier at Terminal 3, and a new dolphin mooring system at Terminal 3. The project site is located between Airport Way and State Route (SR) 109 immediately south of the Hoquiam High School. The approximately 2,100-foot-long by 1,700-foot-wide site was previously part of a log storage and handling area for a pulp mill. It is bordered on the west by the City of Hoquiam's wastewater treatment facilities and a wildlife viewing area. It is bordered on the east by an active pulp mill and export operation.

At this time we do not have specific information from the design team regarding the vertical and lateral loading on the foundations for the proposed oil tank area.

3.0 GEOTECHNICAL INVESTIGATIONS

To evaluate the subsurface soil and groundwater conditions, subsurface explorations were conducted at the corners of the proposed oil storage tank area, along the proposed aboveground pipe alignment and along the proposed new rail spur line and siding. The geotechnical investigations included a review of existing geotechnical data and a phased field exploration program.

3.1 Existing Geotechnical Data

Shannon & Wilson has completed three projects in the vicinity of the proposed terminal facility including: Geotechnical Report for the Wastewater Treatment Facility Biosolids Lagoon Project Hoquiam, Washington (2008), and Draft Geotechnical Report for Improvements to the Wastewater Treatment Plant Aberdeen, Washington (2001). Our scope of services includes review of geotechnical data to be provided by the former consultants for the Port of Grays Harbor for the Terminal 3 pier construction and recent dolphin modifications made in 2009. The Terminal 3 data has not yet been provided and is therefore not included in this draft report.

3.2 Explorations

A total of seven geotechnical borings were drilled to characterize the subsurface conditions at the tank site and pipe alignment. A total of 10 test pits were excavated along the new rail spur line and siding. The explorations were completed in series with the borings performed between April 2 and 11, 2013, and the test pits completed on April 15, 2013. The designation, type, drilling or excavation method, depth, and date for each of the explorations are summarized in Table 1. The approximate locations of the borings and the test pits are shown in Figure 2.

3.2.1 Geotechnical Borings

The seven borings completed in April 2013 were drilled by Gregory Drilling of Redmond, Washington, using a track-mounted drill rig. Drilling services were conducted under subcontract to Shannon & Wilson, Inc. A geologist from Shannon & Wilson was present throughout the field exploration periods to observe the drilling and sampling operations, retrieve representative soil samples for subsequent laboratory testing, and to prepare descriptive field logs. The samples were placed in airtight jars or sealed steel tubes and returned to our laboratory for testing.

The seven borings completed in April 2013 were drilled using hollow-stem auger (HSA) drilling and mud-rotary techniques. HSA drilling consisted of using continuous-flight augers to advance the boring and to remove soil from the borehole. Samples were obtained by removing the center bit and lowering a sampler through the auger. HSA methods were used in the upper 10 feet of borings in the Holocene fill. Mud-rotary borings are advanced by circulating drilling mud from a mud tank at the ground surface, down the drill rods, out through the drill bit, up the annulus between the drill rods and borehole, and back into the mud tank. The circulation of drilling mud removes the cuttings generated during the drilling process and carries them to the surface, where they are allowed to settle out in the mud tank. The drilling mud also helps keep the hole from caving or collapsing during sampling. Samples are obtained by removing the drill rods and drill bit from the borehole, removing the drill bit from the ends of the rods, attaching the sampler to the drill rods, and lowering the sampler to the bottom of the mud-filled open hole.

The boring logs for this project are presented in Appendix A. A boring log is a written record of the subsurface conditions encountered in the boring. It graphically shows the geologic units (layers) encountered in the boring and the Unified Soil Classification System (USCS) symbol of each geologic layer. It also includes the natural water content (where tested), penetration resistance, and various depths within the boring log where tests were performed. Other information shown on the boring logs includes ground surface elevation, types and depths of sampling, descriptions of obstructions and debris encountered in the borings, and observed drilling problems and soil behavior related to caving, raveling, and heave. A soil classification and log key for the boring logs is presented in Figure A-1 (Appendix A).

After completion of the drilling and sampling, observation wells were installed in B-3 and B-6 to measure groundwater levels. A driller licensed in Washington installed the wells. The installation details for the observation wells and the most recent groundwater level measurements are included on the boring logs in Appendix A.

An archeologist from Archaeological Macroflora Identification, Olympia, Washington, was on site during the drilling of select borings to screen the samples and cuttings for artifacts. To our knowledge, no cultural artifacts were encountered.

All cuttings and drilling mud from borings on the site were spread out next to the boring.

3.2.2 Test Pits

Shallow subsurface conditions along the proposed new rail spur line were evaluated using test pit excavations. The locations of the test pits are shown in Figure 2. The test pits were excavated using a steel-tracked excavator operated by Quigg Bros., Inc. subcontracted to

Shannon & Wilson, Inc. The test pits depths ranged from 4 to 9 feet and they were backfilled using the materials excavated. A Shannon & Wilson representative observed and logged the test pits, retrieved representative grab samples, and prepared descriptive test pit logs. The logs of the test pits are presented in Appendix A.

3.2.3 Soil Sampling

Soil samples from the borings were typically obtained in conjunction with the Standard Penetration Test (SPT) at the depths shown on the boring logs. SPTs were performed in general accordance with ASTM International (ASTM) Designation D 1586, Standard Method for Penetration Testing and Split-Barrel Sampling of Soils. SPTs were generally performed every 2.5 feet to a depth of 20 feet and then every 5 feet to the bottom of the borehole. The SPT consists of driving a 2-inch outside-diameter, split-spoon sampler a distance of 18 inches into the bottom of the borehole with a 140-pound hammer falling 30 inches. The number of blows required for the last 12 inches of penetration is termed the Standard Penetration Resistance (N-value). These values are plotted at the appropriate depths on the boring logs. Generally, whenever 50 or more blows were required to cause 6 inches or less of penetration, the test was terminated, and the number of blows and the corresponding penetration were recorded. The N-value is an empirical parameter that provides a means for evaluating the relative density, or compactness, of granular soils and the consistency, or stiffness, of cohesive soils.

At select locations, relatively undisturbed samples were obtained using 73-millimeter (mm) inside-diameter, hydraulically pushed, thin-walled tube samplers (Shelby tubes). These samples were collected in general accordance with ASTM D 1587-08, Standard Practice for Thin-Walled Tube Geotechnical Sampling of Soils. This sampling method employs a thin-walled, steel tube connected to a sampling head that is attached to the drill rods. The tube is pushed by the hydraulic rams of the drill rig into the soil below the bottom of the drill hole and then retracted to obtain a sample. This type of sampler is generally used in soft to stiff, fine-grained soils.

Grab samples were obtained from the test pits. The designations and depths of the grab samples are presented on the test pit logs in Appendix A.

4.0 LABORATORY TESTING

Laboratory tests were performed on selected soil samples retrieved from the borings and test pits. The laboratory testing program included a variety of tests to classify the soils and to provide data for engineering studies. Classification and index laboratory tests included visual classification

and tests to determine natural water content and the grain size distribution. The results from the laboratory tests are included in Appendix B.

4.1 Water Content Determinations

Water content was determined on selected samples in general accordance with ASTM D 2216, Test Method for Determination of Water (Moisture) Content of Soil and Rock. The water content is shown graphically on each boring log (Appendix A).

4.2 Grain Size and Hydrometer Analyses

The grain size distribution of selected samples was determined in general accordance with ASTM D 422, Standard Test Method for Particle-Size Analysis of Soils. Results of these analyses are presented as gradation curves in Appendix B. Each gradation sheet provides the USCS group symbol, the sample description, and water content. The samples with fewer than 50 percent fines were assigned USCS classifications in general accordance with ASTM D 2488, Standard Recommended Practice for Description of Soils (Visual-Manual Procedure).

4.3 Atterberg Limits

Soil plasticity was determined in general accordance with ASTM D 4318, Standard Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils by performing Atterberg limits tests on selected fine-grained samples. The Atterberg limits include Liquid Limit (LL), Plastic Limit (PL), and Plasticity Index (PI=LL-PL). The results are shown graphically on the boring logs in Appendix A and plotted on plasticity charts presented in Appendix B. The plasticity charts provide USCS group symbols, the sample descriptions, water content, and percent passing the No. 200 sieve (if a grain size analysis was performed).

4.4 One-dimensional Consolidation Tests on Relatively Undisturbed Tube Samples

After being carefully transported from the field, each relatively undisturbed tube sample was properly stored in the laboratory in an upright position, as it was taken in the field. The samples were pushed out of the tubes (in the same direction they entered the tube) onto a continuously supported tray. The samples were carefully logged, the soils classified, and water content tests performed. The classifications and water contents are shown on the boring logs in Appendix A. Representative samples were taken of clay soils for consolidation testing.

A one-dimensional consolidation test was performed on three relatively undisturbed cohesive soil samples in general accordance with ASTM Designation: D 2435, Standard Test Method for

One-Dimensional Consolidation Properties of Soils. The samples were incrementally loaded in a 63.5-mm-diameter fixed-ring consolidometer. Each load increment approximately doubled the previous load, to a maximum stress of 594 kilo Pascals (kPa). Drainage was allowed from both the top and bottom of the sample. The sample was inundated with distilled water after the first load increment, with additional loads applied immediately as necessary to prevent swelling. Thereafter, load increments were applied at the end of primary consolidation, or after each day during testing. Upon reaching 38 kPa, 148 kPa, and the maximum normal load (594 kPa), the sample was unloaded in decrements equal to about one-fourth of the previous load.

The consolidation test results are presented in Figures B-6a through B-6j. They include vertical deformation versus logarithmic time curves for each load increment and summary plots of percent settlement and void ratio versus logarithmic stress. Pertinent specimen data are also presented on the data sheets.

5.0 SUBSURFACE CONDITIONS

The geology and subsurface conditions along the project alignment were inferred from soil samples and information obtained from current borings, test pits, and observation wells; from data gathered from existing projects in the vicinity; and from the Earthquake-induced Landslide and Liquefaction Susceptibility and Initiation Potential Maps for Tsunami Inundation Zones in Aberdeen, Hoquiam, and Cosmopolis, Grays Harbor County, Washington (Slaughter and others, 2013). The following sections include a description of the general geology, geologic units, and the subsurface soil and groundwater conditions encountered in the project area.

5.1 General Geology

The project site is located in the Grays's Harbor basin, south of the Olympic Range and east of the convergence zone where the North American Tectonic Plate is subducting beneath the Juan De Fuca Plate. The Grays Harbor Basin is partially bounded on the west by two peninsulas including the 7.5-mile-long Ocean Shores peninsula to the north and the 5-mile-long Westport peninsula to the south. The main river flowing through the area is the Chehalis and flows from the east of the project location. The Chehalis River watershed is the second largest in Washington State, draining an approximately 2,600-square-mile basin (Ely and others, 2008) and located entirely in southwest Washington (Slaughter and others, 2013). The geology of the project area consists primarily of unconsolidated sediments in the low-lying area between the Chehalis River and the bedrock cored hills to the north. These sediments include elements of alluvium from the current Chehalis River, including tide flat, estuarial sediments, and flood plain sediments, and older Pleistocene aged alluvium. Soil at depths of 150 feet or more consist of outwash from alpine glaciations in the Olympic Mountains (Logan, 1987).

5.2 Geologic Units

Based on our review of geologic maps and geotechnical investigations, the project area is predominately underlain by Holocene fill, alluvium, estuarine deposits, and deep glacial outwash deposits. The soil units encountered in our geotechnical investigations, from youngest to oldest, are as follows:

- **Fill (Hf)** – Materials placed by humans, both engineered and nonengineered. Typically, very loose to dense, comprised of various materials including soil, quarry spalls, construction debris, cobbles, boulders, wood chips, and debris. The fill thicknesses identified in the borings ranged from 3 to 15 feet.
- **Estuarine Deposits (He)** – Estuary deposits of the current and ancestral Chehalis River. Clayey silt to silty clay with interbedded, silty sand. Local concentrations of organic-rich silt.
- **Alluvium (Ha)** – River or creek deposits, normally associated with historic streams, including overbank deposits. Typically, very loose to medium dense silts, sands and gravels; can include very soft to stiff clay, silt, peat, and wood debris. These deposits are present in the vicinity of Ennis Creek.
- **Advance Outwash (Qva)** – Glaciofluvial sediments deposited as the glacial ice advanced. Clean to silty sand, sandy gravel; dense to very dense.

5.3 Subsurface Conditions

Our understanding of the subsurface soil conditions along the alignment is based on our review of geologic maps, existing data, recent geotechnical investigations, and on our general understanding of the geologic history and stratigraphy of the region. Our interpretation of the subsurface soil and groundwater conditions along the project alignment is shown on the Generalized Subsurface Profile in Figure 3.

In general, the project site is underlain by a variety of normally consolidated deposits including very loose to very dense fill (Hf), estuarine deposits (He), and alluvial deposits (Ha). Underlying these recent deposits are dense to very dense advance outwash deposits (Qva). The subsurface conditions along the project alignment can be divided into three sections including from north to south at the proposed oil tank storage locations, from north to south along the current pulp mill access road, and from northwest to southeast along the proposed rail spur line and siding.

5.3.1 Oil Storage Tank Area Cross Section

The subsurface soil and groundwater conditions for the proposed oil tank storage area were inferred from borings B-1, B-2, B-3, B-4, and B-5. The subsurface soil and groundwater conditions along this section of the alignment are shown in Figure 3, Sheet 1 of 2.

From the north end of the subsurface profile to the southern end, soil conditions consist of about 5 to 10 feet of fill (Hf), overlying the first layer of estuarine deposits (He) (35 to 40 feet thick), 30 to 40 feet of alluvial sand (Ha), and a second layer of estuarine deposits (60 to 70 feet thick) (He). The second layer of estuarine deposits overlies a layer of advance glacial outwash consisting of sandy gravel and gravelly cobbles with boulders. The fill (Hf) contains loose to dense, sandy gravel, moist to wet, with scattered to abundant organics. The fill is separated from the underlying deposits by a layer of separation geotextile that is clearly visible in the test pit sections. The estuarine deposits (He) consists of very soft to loose, interbedded silty sand, clayey silt, silty clay, and organic silt. The alluvium (Ha) consists of medium dense to dense, silty sand. The advance glacial outwash (Qva) consists of dense to very dense, slightly silty, sandy gravel grading to gravelly cobbles with boulders.

Groundwater information obtained from the observation well installed in B-3 indicates a groundwater depth of about 17 feet below ground surface (bgs) (Elevation 10 feet). During drilling, observations made in borings B-1, B-2, B-4, and B-5, indicate groundwater depths of 5 to 7 feet bgs (Elevation +10 to +13 feet).

5.3.2 Elevated Pipe Alignment

The subsurface soil and groundwater conditions along the alignment of the elevated transport pipe were inferred from borings B-6 and B-7. Based on these borings, the soils along this alignment consist of 1 to 3 feet of fill (Hf) over 50 feet of estuarine deposits (He), and a layer of alluvial sand (Ha) from Elevation -35 feet to the bottom of the completed borings. The fill (Hf) consists of gravel and fill placed to construct the access road located immediately east of the boring locations. The estuarine deposits (He) consist of very soft to medium dense interbedded silty sand; slightly sandy, clayey silt; silty clay; and organic-rich silt. The alluvial deposits (Ha) consist of medium dense, silty sand with shell fragments.

Groundwater information, obtained from observations during drilling in boring B-7 and an observation well in B-7, indicates a groundwater depth of about 5 feet bgs (Elevation +17 feet).

5.3.3 Test Pits along Proposed Rail Spur Line and Siding

The subsurface soil and groundwater conditions along the proposed spur line and siding are divided into two different sections. The first section covers the first 600 feet from test pit TP-1 to TP-3. Based on the observed soils in the test pits, the soils along this section generally consist of 7 to 8 feet of fill (Hf) overlying estuarine deposits (He). The fill can be divided into three sub-sections including approximately 1 to 2 feet of yellow to orange clayey sand (weathered bedrock fill), possibly sourced from the construction of SR 109 to the north. The

second layer of fill contains wood waste and scattered gravel and cobbles with remnants of logs 4 to 6 feet in length. Typically, groundwater is located in this layer and the test pit excavations were flooded with groundwater flowing at 1 to 5 gallons per minute into the excavated area. This wood waste layer was interbedded with an approximately 1-foot-thick layer of compacted gravel and silt, possibly derived from a previous working surface of former mill operations. The third layer consisted of more wood waste and gravel with abundant wood chips. An estuarine deposit (He) containing very soft silty clay and clayey silt is located below the fill.

Test pits TP-4 through TP-10 typically contain 3 to 7 feet of Holocene fill (Hf) on top of either estuarine deposits (He) or alluvial sand (Ha). The fill typically contains dense to very dense imported gravel and quarry rock but locally around drainage swales the rock/gravel is thinner (TP-4) or absent (TP-8). The alluvium (Ha) consists of loose, slightly silty to silty sand with scattered to abundant organics. The estuarine deposits (He) consist of very soft silty clay and clayey silt and locally black organic silt with scattered wood chips and log fragments. A layer of woven geotextile was found in most of the test pits between TP-4 and TP-10 including (TP-4, TP-7, TP-9, and TP-10) at depths between 3 and 6 feet bgs.

6.0 ENGINEERING STUDIES AND RECOMMENDATIONS

6.1 General

Based on our current understanding of the proposed oil loading and storage facility and the results of our geotechnical studies, we have developed geotechnical recommendations for design and construction of the proposed facility. The following sections provide recommendations for seismic design considerations, deep foundation design, and other pertinent geotechnical design and construction issues.

6.2 Geotechnical Earthquake Engineering

6.2.1 Design Ground Motions

The project is located in a moderately active seismic region. While the region has historically experienced moderate to large earthquakes (i.e., April 13, 1949, magnitude 7.1 Olympia Earthquake; April 29, 1965, magnitude 6.5 Seattle-Tacoma Earthquake; and February 28, 2001, magnitude 6.8 Nisqually Earthquake), geologic evidence suggests that larger earthquakes have occurred in the prehistoric past and will occur in the future (e.g., magnitude 8.5 to 9.0 Cascadia Subduction Zone Interplate events, magnitude 7.5 Seattle Fault events).

We understand that the proposed structures will be designed in accordance with the International Building Code (IBC) 2012. For the IBC 2012, the seismological inputs are short-period spectral acceleration, S_S, and spectral acceleration at the 1-second period, S_1. The coefficients, S_S and S_1, are for a maximum considered earthquake, which corresponds to a ground motion with a 2 percent probability of exceedance in 50 years, or a 2,475-year return period (with a deterministic maximum cap in some regions). The coefficients are based on regional probabilistic ground motion studies completed in 2008 by the U.S. Geological Survey (USGS).

The spectral response acceleration values are scaled by site soil response factors to account for site amplification/damping effects. The site classification determines the site soil response factors. Our analysis of geologic conditions indicates that the proposed building site is underlain by soft silts and clays, therefore, the site can be classified as Site Class E. The seismological inputs are short-period spectral acceleration, S_S, and spectral acceleration at the 1-second period, S_1, taken from approved National Earthquake Hazards Reduction Program spectral response acceleration contour map for Class B sites (shown in Figure 1613 in the code). Sites classified as Class B are defined as firm rock having a shear-wave velocity between 2,500 and 5,000 feet per second in the top 100 feet. The seismological inputs are modified for Site Class E. The mapped S_S and S_1 values, site coefficients, and design values corresponding to Site Class E are presented in Table 2.

6.2.2 Earthquake-induced Geologic Hazards

Earthquake-induced geologic hazards that may affect a site include landsliding, fault rupture, settlement, and liquefaction and associated effects (such as loss of shear strength, bearing capacity failures, loss of lateral support, ground oscillation, and lateral spreading). Because of the flat site topography, the risk of landsliding at this site is low.

The potential for fault rupture is also low. The nearest mapped fault (USGS, 2006) is a northeast trace of the Saddle Hill Fault Zone, which consists of short, discontinuous traces that trend northeast-southwest for a distance of about 16 miles. The project site is located about 10 miles southeast of the most northeastern trace. Evidence of Holocene rupture (i.e., movement within the last 10,000 years) has not been reported along this trace of the Saddle Hill Fault Zone.

Liquefaction and related effects pose an earthquake-induced geologic hazard at the site. Shannon & Wilson calculated factors of safety (FSs) against liquefaction for boring SPT N-values. To calculate the FSs, we used design earthquake ground motions and empirical procedures established by the National Center for Earthquake Engineering Research that include

procedures developed by Youd and others (2001), Idriss and Boulanger (2004), and Cetin and others (2004). We computed the FSs using a 0.64g peak ground acceleration from a magnitude 9.0 source, located about 20 miles away (using the IBC 2012 design values).

Liquefaction potential of the subsurface soils was estimated using the SPT N-values for soil samples obtained in the borings and the measured or estimated fines content of those samples. Based on our calculations, cohesionless and marginally cohesive soil units at the site are potentially susceptible to liquefaction. Specifically, layers of silty sand and sandy silt (Alluvium deposits), located about 40 to 110 feet deep, are susceptible to liquefaction, while silty clay and clayey silt layers (Estuarine deposits) within the upper 30 feet are generally not susceptible to liquefaction. Thin sand layers within the upper 30 to 40 feet are either discontinuous laterally or display widely varying fines contents.

Shannon & Wilson estimated post-liquefaction settlement using the methods of Tokimatsu and Seed (1987) and Ishihara, Yoshimine, and Mitsutoshi (1992). We based our estimate of the FSs against liquefaction and soil relative density (using correlations with corrected SPT blow counts). Post-liquefaction settlement of up to 1 to 2 feet could occur at the site.

One of the major liquefaction-induced types of ground failure is lateral spreading of shoreline areas. Lateral spreading movement of gently sloping ground occurs as a result of pore-pressure buildup or liquefaction in the underlying soil deposit. A lateral spread often contains a liquefied layer overlain by a non-liquefied layer at the ground surface that rides along the top of the liquefied soil. The non-liquefied layer is present because either it lies above the groundwater table or it is not susceptible to liquefaction.

Based on the empirical procedure by Youd, Hansen, and Bartlett (2002), lateral spreading displacement of the shoreline at the subject site for the design level earthquake could be as much as 10 feet. The lateral spread displacements would generally be in a southerly direction (toward Grays Harbor). In our opinion, it is not likely that lateral spreading would progress northward such that the proposed tank farm would be impacted. Pile-supported structures along the shoreline could be affected by lateral spreading.

6.3 Evaluation of Shallow Foundations

We have considered the feasibility of using shallow ring foundations or mat foundations for support of the proposed oil storage tanks. The following sections present results of our

settlement analyses at the oil storage tank area and rail spur line and siding. We performed the settlement analyses using Settle3D (Rocscience, 2012).

6.3.1 Oil Storage Tank Area

We evaluated the suitability of shallow foundations in the oil storage tank area by assuming a bearing pressure of 4,000 pounds per square foot (psf) at each of the eight tanks. Under this loading condition, we estimate 3 to 10 feet of primary consolidation settlement to occur over the course of approximately 10 years. The magnitude and duration of settlement could be reduced by installing wick drains and preloading the site. However, the amount of soil required for preloading is not likely to be economical when compared to deep foundations. We do not recommend using shallow foundations for the oil storage tanks.

6.3.2 Rail Spur Line and Siding

We evaluated settlement for the rail spur line and siding. We assumed the embankment would be up to 100 feet wide at the base and have a combined rail and soil load of about 750 psf. Our analyses estimate 3 to 6 inches of settlement. We predict that about one-quarter of the settlement would occur during or immediately after construction and another one-quarter of the settlement would occur within about one year. We estimate that primary consolidation would conclude about five years after construction.

Based on our consolidation testing, the upper estuarine layer appears to be preconsolidated by about 1,000 psf. The magnitude of settlement would increase exponentially if the loads on the soil exceed this preconsolidation pressure. We recommend limiting the height of the embankment and/or load of parked rail cars such that the combined load does not exceed 1,000 psf.

6.4 Deep Foundations

Based on the time and magnitude of settlement associated with preloading the site for a mat foundation, we recommend that the oil storage tanks be supported on deep foundations. We recommend that closed-end, steel pipe piles be used as the deep foundation system for this project. The following sections present recommendations for closed-end steel pipe piles at the oil storage tank area and aboveground delivery pipe alignment. Pile installation recommendations are provided later in the Construction Considerations section of this report.

6.4.1 Pile Design Requirements

We evaluated two sizes of deep foundations for the tank area and delivery pipe alignment: driven 18- and 24-inch-diameter, closed-end, steel pipe piles. We understand that total allowable axial loads have not yet been determined at the tank area and delivery pipe alignment; however, we assume that pile capacities on the order of 500 to 1,500 kips may be required. The following recommendations are intended to provide the preliminary pile size and capacity information for the proposed loading conditions.

6.4.2 Axial Pile Capacities

Based on our review of recent and previous borings, we anticipate that the pile capacity will be derived by a combination of side and end resistance in the very dense, sandy gravel bearing layer. Our subsurface explorations indicate that the depth to the top of the dense to very dense layer is approximately 155 to 160 feet bgs at the proposed oil storage tank locations. In order to achieve full end bearing resistance, we recommend penetration depths of at least 5 feet into the sandy gravel layer. Adequate pile embedment is critical to pile performance and will require careful observation and control of pile installation.

Our analysis was performed using an in-house computer program that determines ultimate axial compressive capacity by summing ultimate skin friction and negative settlement-induced downdrag forces along the side of the pile and ultimate end bearing at its tip. For each pile size we considered static and post-seismic loading. For static loading, no downdrag forces were applied and we recommend an FS of 2.0 be applied to the ultimate total compressive capacity. After a seismic event (post-seismic loading), the induced excess porewater pressures in the liquefied layers will gradually reduce, causing consolidation settlement. Settlement in the liquefied layers may cause overlying layers to settle as well, resulting in downdrag forces (negative friction). We recommend applying an appropriate FS (typically 1.1 to 1.5) to ultimate total compressive capacity to obtain allowable total compressive capacity for seismic and (temporary) post-seismic loading conditions. Adding downdrag loads to the pile design capacity has the effect of temporarily reducing the FS.

The results of our analyses are presented in Table 3.

The recommended embedment into the dense to very dense layer is dependent on the pile diameter, hammer selection, and the required design load. We recommend that a test pile program be conducted during construction to confirm that the design pile capacity can be

achieved at the design pile tip elevation. Test pile program recommendations are provided later in this report.

Our analyses were performed for a single pile; no group effects were considered. We recommend that piles be spaced no closer than three pile diameters apart measured center to center. At this spacing, a group reduction factor is not warranted when estimating group axial capacity.

Assuming pile penetration into the very dense gravel, we estimate total pile settlements, including elastic strain, of a closed-end steel pipe pile would be on the order of ¼ to ½ inch, with differential settlements between piles of about half the total settlement. Due to the granular nature of the bearing soils, these settlements would be primarily elastic and would occur as the load is applied.

6.4.3 Uplift Resistance

We recommend an FS of 3.0 for the long-term loading and an FS of 1.5 for transient loading, such as wind and seismic loads, be applied to the ultimate uplift capacities. The uplift resistance is a summation of the ultimate skin friction along the side of the pile.

6.4.4 Lateral Resistance

Lateral loads acting on the oil storage tanks or delivery pipes from earthquake and wind, as well as other loadings, may be resisted by the lateral resistance provided by the deep foundations. The magnitude of lateral resistance developed by a deep foundation depends on the subsurface conditions encountered and the deep foundation type and size.

We have developed soil input parameters for use in the computer program LPILE Plus (Reese & Wang, 2006). We assume that the LPILE Plus analyses will be conducted by the structural design team members. These input parameters are summarized in Table 4.

6.4.5 Pile-driving Criteria

Hammer stroke lengths and blow counts for specific axial capacities can be provided after the Contractor selects the final driving system. All pipe piles will be driven to an ultimate capacity, which is twice the design compression capacity. We recommend that the driving criteria for test and production piles be established based on a Wave Equation Analysis of Pile Driving (WEAP). This method includes an evaluation of driving stresses so that an appropriate

pile-driving hammer size can be selected to obtain the desired pile capacity without damage to the pile or hammer. This analysis also determines the ultimate pile capacity for a given driving resistance.

The Contractor should furnish the manufacturer's specifications and catalog for the proposed hammer at least seven days in advance of the scheduled pile driving in order for us to complete the WEAP studies. We recommend that a geotechnical engineer from our firm who is experienced in pile driving and familiar with the subsurface conditions at the site be retained on a full-time basis to evaluate pile-driving records so that timely decisions on acceptance can be made.

We recommend that all piles be driven to the estimated pile tip elevations or minimum penetration into gravel and to the required final driving resistance for the last foot as determined by WEAP. If the pile-driving resistances are less than the minimum values obtained from WEAP under continuous driving conditions as they approach the minimum penetration depths, the Contractor should continue driving the piles until they reach the required driving resistances. However, if the pile-driving resistances continue to be less than the minimum values obtained from WEAP, pile driving should stop when the piles are 6 inches above the final cutoff elevation. For these piles, driving should be discontinued for a minimum of 24 hours and then redriven for 6 inches or less of penetration. The acceptable redrive resistance should be greater than the specified minimum driving resistance. The redrive resistance should be based on Case Pile Wave Analysis Program (CAPWAP) results. If the redrive resistances do not meet the specified values, the Engineer will determine the acceptability of the piles and the subsequent procedures to be taken.

Should the required minimum driving resistance be achieved before piles reach the estimated tip penetrations, the piles should be driven to approximately 120 blows per foot. If the driving resistance exceeds 120 blows per foot at a penetration more than 10 feet above the design tip elevation, the Contractor should inform the Geotechnical Engineer, who will determine if the pile is acceptable as is or whether additional methods must be employed to meet the penetration requirement. In order to avoid over-stressing the pile section for refusal conditions, a higher-yield-strength steel (greater than 45 kips per square inch) may be necessary. WEAP can determine the pile stresses caused by pile driving at the higher driving resistance.

6.4.6 Test Pile Program

The recommendations for pile foundations and embedment are based on theoretical and empirical data, average subsurface conditions at the site, and our engineering judgment and

experience. In order to substantiate our preliminary estimates, we recommend that a test pile program be undertaken to determine lengths and driving resistance for production piles. Production pile order lengths should be determined after evaluation of the test pile driving records. In our opinion, a test pile program is very useful for refining soil structure interaction assumptions and often results in a cost-effective driving criteria that can reduce driving time.

The test pile program should consist of driving a minimum of 10 indicator piles at the site. An indicator pile should be located within the footprint of each tank (8) and in at least two locations along the above ground delivery pipe. We have assumed a depth to the high-capacity pile bearing layer of approximately 160 feet.

At the elevated pipeline bridge location, test piles could indicate shorter piles are appropriate. That will be evaluated during test pile driving. The piles should be driven to the ultimate capacity (two times the design capacity). Pile dynamic analyzer measurements should be performed for each indicator pile. A CAPWAP should be performed on each test pile. Based on our experience, dynamic pile tests are one of the most cost-effective methods for determining the total ultimate capacities and the distribution of skin friction and end bearing of the piles. Test piles may be used as production piles if they meet the specified installation procedures and requirements. The results of CAPWAP analyses sometimes indicate a time-dependant capacity increase (setup) that occurs in granular bearing soils. Therefore, we recommend that a CAPWAP restrike analysis be performed on at least three indicator piles after an appropriate setup time has passed, typically about three days.

6.5 Driven Pile Installation

6.5.1 Pile-driving Equipment

A diesel-powered hammer may be used for driving the proposed steel piles. All pile-driving equipment should be designed, constructed, and maintained in a manner suitable for the work to be accomplished for this project. If, in the opinion of the Owner, the driving equipment is inadequate or deficient, the Owner may direct that it be removed from the job site. All costs for re-mobilizing, removing, or replacing such equipment should be at the Contractor’s expense.

6.5.2 Pile-driving Conditions

Based on the conditions encountered in the borings, we anticipate that driven pile installation for the proposed oil storage tanks and delivery pipes would encounter easy driving conditions in the upper 50 feet, moderate driving conditions from about 50 to 80 feet, easy driving conditions from 80 to 155 feet, and hard driving conditions below 155 feet in the very

dense gravel. The loose/soft soil and soft silt/clay layers could cause misfire of diesel hammers because of the minimal driving resistance. We recommend piles be driven with a hammer that allows variable energy settings. Alternatively, the piles may be predriven through the soft soils using a smaller diesel hammer or a vibratory hammer. A larger hammer could then be used to drive the piles into the dense, bearing layer.

6.5.3 Monitoring Pile Driving

An experienced and qualified geotechnical engineer from our firm familiar with the subsurface conditions at the site should observe and evaluate all pile driving by making a continuous driving record of each pile. For this purpose, the Contractor should be required to mark the pile in 1-foot increments. During restrike, additional 1-inch increments between the 1-foot marks would be required.

The pile-driving record will include hammer stroke (diesel hammers), blows per foot, time, date, reasons for delays, and other pertinent information. In addition, the record will include tip elevation, specified criteria, and the initials of inspectors making final acceptance of the pile. The pile-driving records should be reviewed on a daily basis.

It is often difficult to visually estimate the energy delivered by diesel hammers. During construction, we recommend that the Saximeter, developed by Pile Dynamic, Inc., be used to record blow counts and stroke length that will correlate to driving energy. Use of the Saximeter during pile driving allows for verification that the pile is being driven with the hammer energy needed to develop the required axial capacity.

6.5.4 Pile-driving Vibrations, Movement Monitoring, and Noise Levels

In general, the potential exists for damage to existing nearby structures and buried utilities because of vibrations caused by pile-driving operations. Because the project site consists of a large, open field with no significant structures nearby, we do not expect that pile driving vibrations or noise will be significant issues for this project.

7.0 CONSTRUCTION CONSIDERATIONS

7.1 Fill Material, Placement, and Compaction

All fill material placed beneath structures, such as non-structural floor slabs, pavements, sidewalks, and around pile caps and grade beams or other areas where settlements are to be reduced, should consist of structural fill. The granular fill material that covers much of the oil tank site generally consists of sandy gravel and may be suitable for reuse as structural fill during

dry weather, provided it is stockpiled and maintained at its natural moisture content. This will require protection from rainfall. Native surface soils and fills encountered in the test pits and borings B-6 and B-7 are moisture sensitive and may be difficult to place and compact during wet weather, rendering them unacceptable for use as structural fill.

Imported structural fill for backfilling grade beam or pile cap overexcavations and beneath slab-on-grade floors that are not settlement sensitive should meet the gradation requirements of Section 9-03.14(1), Gravel Borrow, of the 2010 Washington State Department of Transportation specifications and the moisture content of the soils should be near optimum for proper compaction. If fill is to be placed during periods of wet weather or under wet conditions, it should have the added requirement that the percentage of fines (materials passing the No. 200 sieve based on wet-sieving the minus ¾-inch fraction) be limited to 5 percent or less. Any fines should be non-plastic.

Prior to placement of structural fill, any ponding water should be drained from the area. Structural fill should be placed in uniform lifts and compacted to a dense and unyielding condition, and to at least 95 percent of the Modified Proctor maximum dry density (ASTM D: 1557). All subgrades to receive structural fill should first be compacted to a dense, unyielding condition under the observation of a geotechnical engineer (or representative). In general, the thickness of soil layers before compaction should not exceed 10 inches for heavy equipment compactors and 6 inches for hand-operated mechanical compactors. The most appropriate lift thickness should be determined in the field using the Contractor's selected equipment and fill and verified by the geotechnical engineer (or representative) with in situ soil density testing. Structural fill operations should be observed and evaluated by an experienced geotechnical engineer or representative. Some subgrade areas could require ground improvement with geogrids and ballast for construction equipment and roadway support.

7.2 Excavation and Dewatering

Temporary excavation slopes could be used where planned excavation limits would not undermine existing structures, interfere with other construction, or extend beyond construction limits. Where there is not enough area for sloped excavations, temporary shoring should be provided.

Consistent with conventional construction practice, temporary excavation slopes should be made the responsibility of the Contractor. The Contractor is continually at the site and is able to observe the nature and conditions of the subsurface materials encountered, including groundwater, and has responsibility for the methods, sequence, and schedule of construction. If instability is detected, slopes should be flattened or shored. Regardless of the construction

method used, all excavation work should be accomplished in compliance with applicable local, state, and federal safety codes.

We anticipate that excavations could be accomplished with conventional excavating equipment, such as a dozer, front-end loader, or backhoe.

Except as otherwise designed and/or specifically covered in the contract, the Contractor should be made responsible for control of all surface and groundwater encountered during construction. In this regard, sloping, slope protection, ditching, sumps, trench drains, dewatering, and other measures should be employed as necessary to permit proper completion of work. Groundwater was previously observed in subsurface explorations performed by Shannon & Wilson at depths of 3 to 5 feet bgs, and may fluctuate because of weather or seasonal variations; therefore, the Contractor should anticipate the need to provide local dewatering measures (i.e., trenches and sump pumps) to control groundwater during excavation, if necessary.

7.3 Construction Erosion Control

The Contractor should employ proper erosion control measures during construction, especially if construction takes place during wet weather. Covering work areas, soil stockpiles, or slopes with plastic, sandbags, sumps, and other measures should be employed as necessary to permit proper completion of the work. Bales of straw, geotextile silt fences, rock-stabilized entrance, street sweeper, and drain inlet sediment screens/collection systems should be appropriately located to control soil movement and erosion.

7.4 Wet Weather Construction Considerations

In the project area, wet weather generally begins about mid-October and continues through about May, although rainy periods could occur at any time of year. In addition, during wet weather months, the groundwater levels could increase, resulting in seepage into site excavations. It would be advisable to schedule earthwork during the dry weather months of June through September. Performing earthwork during dry weather would reduce these problems and costs associated with rainwater, trafficability, and handling of wet soil. However, should wet weather/wet condition earthwork be unavoidable, the following requirements are recommended:

- The ground surface in and surrounding the construction area should be sloped to promote runoff of precipitation away from work areas and to prevent ponding of water.
- Work areas or slopes should be covered with plastic and appropriate temporary sediment and erosion control measures should be applied.

- The use of sloping, ditching, sumps, dewatering, and other measures should be employed as necessary to permit proper completion of the work.
- Earthwork should be accomplished in small sections to minimize exposure to wet conditions. That is, each section should be small enough so that the removal of unsuitable soils and placement and compaction of clean structural fill could be accomplished on the same day.
- The size of construction equipment may have to be limited to prevent soil disturbance. It may be necessary to excavate soils with a backhoe, or equivalent, and locate them so that equipment does not pass over the excavated area. Thus, subgrade disturbance caused by equipment traffic would be minimized.
- Soil should not be left uncompacted and exposed to moisture. A smooth-drum vibratory roller, or equivalent, should roll the surface to seal out as much water as possible.
- In-place soil or fill soil that becomes wet and unstable and/or too wet to suitably compact should be removed and replaced with clean, granular soil (see gradation requirements in Section 6.1).
- Excavation and placement of structural fill material should be observed on a full-time basis by a geotechnical engineer (or representative) experienced in wet weather/wet condition earthwork to determine that all work is being accomplished in accordance with the project specifications and our recommendations.
- Grading and earthwork should not be accomplished during periods of heavy, continuous rainfall.

We recommend that the above requirements for wet weather/wet condition earthwork be incorporated into the contract specifications.

8.0 ADDITIONAL SERVICES

8.1 Review of Plans and Specifications

We recommend that Shannon & Wilson be retained to review those portions of the plans and specifications that pertain to earthwork and foundation construction prior to completion of the 90 percent drawings, to determine that they are in accordance with recommendations presented in this report.

8.2 Construction Observation

We recommend that Shannon & Wilson be retained to observe the geotechnical aspects of construction, particularly the foundation installation and fill placement and compaction. This observation would allow us to verify the subsurface conditions as they are exposed during construction and to determine that work is accomplished in accordance with our

recommendations. If conditions encountered during construction differ from those anticipated, we can provide recommendations for the conditions actually encountered.

9.0 LIMITATIONS

This draft Geotechnical Report presents the data from field explorations, and field and laboratory testing of subsurface conditions at the specific locations and depths indicated, using the means and methods described in this report. No other representation is made. Subsurface conditions that are interpreted from the data included in this report may not be construed as a guarantee or warranty of such interpreted conditions.

Natural processes or human activity may alter subsurface conditions. Because a geotechnical report is based on conditions that existed at the time of subsurface explorations, construction decisions should not be based on a report whose adequacy may have been affected by time, unless verified. Unanticipated soil conditions are commonly encountered and cannot fully be determined by merely taking soil samples from borings.

SHANNON & WILSON, INC.

Martin Page, P.E., L.E.G.
Senior Associate
Geotechnical Engineer, LEED AP, DBIA™

MWP:TMG/mwp

REFRENCES

ASTM International, 2007, Standard test method for particle-size analysis of soils, D422-63 (2007): West Conshohocken, Pa., ASTM International, Annual book of standards, v. 04.08, soil and rock (I): D420 - D5876, 8 p., available: www.astm.org.

ASTM International, 2009, Standard practice for description and identification of soils (visual-manual procedure), D2488-09a: West Conshohocken, Pa., ASTM International, Annual book of standards, v. 04.08, soil and rock (I): D420 - D5876, 11 p., available: www.astm.org.

ASTM International, 2010a, Standard test methods for laboratory determination of water (moisture) content of soil and rock by mass, D2216-10: West Conshohocken, Pa., ASTM International, Annual book of standards, v. 04.08, soil and rock (I): D420 - D5876, 7 p., available: www.astm.org.

ASTM International, 2010b, Standard test methods for liquid limit, plastic limit, and plasticity index of soils, D4318-10: West Conshohocken, Pa., ASTM International, Annual book of standards, v. 04.08, soil and rock (I): D420 - D5876, 16 p., available: www.astm.org.

ASTM International, 2011a, Standard test method for standard penetration test (STP) and split-barrel sampling of soils , D1586-11: West Conshohocken, Pa., ASTM International, Annual book of standards, v. 04.08, soil and rock (I): D420 - D5876, 9 p., available: www.astm.org.

ASTM International, 2011b, Standard test methods for one-dimensional consolidation properties of soils using incremental loading, D2435-11: West Conshohocken, Pa., ASTM International, Annual book of standards, v. 04.08, soil and rock (I): D420 - D5876, 15 p., available: www.astm.org.

ASTM International, 2012a, Standard practice for thin-walled tube sampling of soils for geotechnical purposes, D1587-08(2012)e1: West Conshohocken, Pa., ASTM International, Annual book of standards, v. 04.08, soil and rock (I): D420 - D5876, 4 p., available: www.astm.org.

ASTM International, 2012b, Standard test methods for laboratory compaction characteristics of soil using modified effort (56,000 ft-lbf/ft3 (2,700 kN-m/m3)), D1557-12: West Conshohocken, Pa., ASTM International, Annual book of standards, v. 04.08, soil and rock (I): D420 - D5876, 14 p., available: www.astm.org.

Ely, D. Matthew; Frasl, Kenneth E.; Marshall, Cameron A.; Reed, Fred, **2008**, Seepage investigation for selected river reaches in the Chehalis River basin, Washington: U.S. Geological Survey Scientific Investigations Report 2008-5180, 12 p., available: http://pubs.usgs.gov/sir/2008/5180/.

Ishihara, Kenji and Yoshimine, Mitsutoshi, 1992, Evaluation of settlements in sand deposits following liquefaction during earthquakes: Soils and Foundations, v. 32, no. 1, p. 173-188.

Logan, R. L., 1987, Geologic map of the Chehalis River and Westport quadrangles, Washington: Olympia, Wash., Washington State Division of Geology and Earth Resources, Open File Report 87-8, 17 p., 1 plate, scale 1:100,000.

Reese, L. C.; Wang, S. T.; Isenhower, W. M.; and Arrellaga, J. A., 2010, LPILE Plus v. 6.0: Austin, Texas, Ensoft, Inc.

Rocscience, Inc., [n.d.], Settle 3D, v. 1.0: Toronto, Ontario, Rocscience, Inc.

Schasse, H. W., Wegmann, K. W., and Polenz, Michael, 2004, Geologic map of the Port Angeles and Ediz Hook 7.5-minute quadrangles, Clallam County, Washington: Washington State Department of Geology and Earth Resources, Open-File Report 2004-13, scale 1:24,000.

Shannon & Wilson, Inc., 2008, Geotechnical report wastewater treatment facility biosolids lagoon project, Hoquiam, Washington: Report prepared by Shannon & Wilson, Inc., Seattle, Wash., 21-1-21000-001, for HDR Engineering, Inc., December.

Slaughter, S. L.; Walsh, T. J.; Ypma, Anton; and others, 2013, Earthquake-induced landslide and liquefaction susceptibility and initiation potential maps for tsunami inundation zones in Aberdeen, Hoquiam, and Cosmopolis, Grays Harbor County, Washington, for a M9+ Cascadia subduction zone event: Washington State Department of Geology & Earth Resources, Report of Investigations 36, 39 p., 2 plates, scale 1:18,000.

TABLE 1
SUMMARY OF EXPLORATIONS

Designation	Type of Exploration	Drilling or Excavation Method	Depth (feet)	Date Completed
B-1	Boring	HSA/ Mud Rotary	181.0	4/03/2013
B-2	Boring	HAS/ Mud Rotary	170.0	4/05/2013
B-3	Boring	HSA/ Mud Rotary	66.5	4/05/2013
B-4	Boring	HSA/ Mud Rotary	170.0	4/09/2013
B-5	Boring	HSA/ Mud Rotary	176.0	4/11/2013
B-6	Boring	HSA/ Mud Rotary	61.5	4/11/2013
B-7	Boring	HSA/ Mud Rotary	61.5	4/11/2013
TP-1	Test Pit	Excavator	7.6	4/15/2013
TP-2	Test Pit	Excavator	8.0	4/15/2013
TP-3	Test Pit	Excavator	8.6	4/15/2013
TP-4	Test Pit	Excavator	9.2	4/15/2013
TP-5	Test Pit	Excavator	4.2	4/15/2013
TP-6	Test Pit	Excavator	6.4	4/15/2013
TP-7	Test Pit	Excavator	7.0	4/15/2013
TP-8	Test Pit	Excavator	5.8	4/15/2013
TP-9	Test Pit	Excavator	7.0	4/15/2013
TP-10	Test Pit	Excavator	7.8	4/15/2013

Note:
HSA = hollow-stem auger

TABLE 2
INTERNATIONAL BUILDING CODE 2012
GROUND MOTION PARAMETERS

S_S (g's)	S_1 (g's)	Site Class	S_{MS} (g's)	S_{M1} (g's)	S_{DS} (g's)	S_{D1} (g's)
1.465	0.711	E	1.319	1.707	0.879	1.138

TABLE 3
ESTIMATED AXIAL PILE CAPACITY SUMMARY

Pile Diameter (feet)	Wall Thickness (inch)	End Condition -	Ultimate Side Resistance (kips)	Ultimate End Bearing (kips)	Ultimate Capacity		Post-seismic Downdrag Load (kips)	Recommended Hammer Type -
					Compression (kips)	Uplilft (kips)		
18	⅜	Closed	400	400	800	400	100	D46
24	⅜	Closed	500	700	1,200	500	130	D62

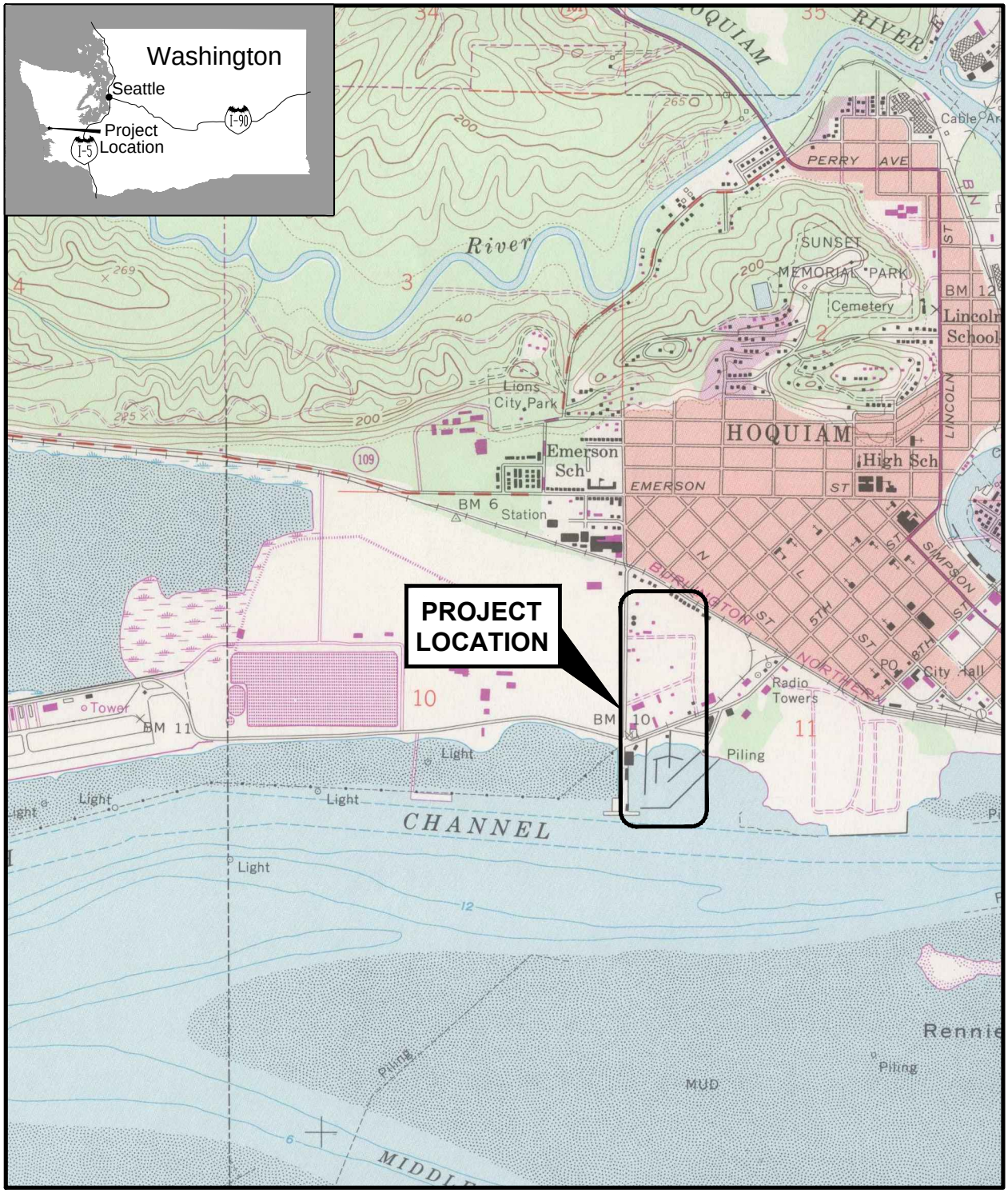
Notes:
1. Ultimate values assume at least 5 feet of penetration into gravel.
2. Assumes pile dynamic analyses will be performed during or before production piles.

TABLE 4
RECOMMENDED GEOTECHNICAL PARAMETERS FOR LATERAL RESISTANCE ANALYSIS

Location	Top Depth (feet)	Bottom Depth (feet)	Groundwater Depth (feet)	Representative Blow Count (blows/foot)	Geologic Unit	LPILE Soil Model	Total Unit Weight, γ (pcf)	Effective Unit Weight, γ' (pcf)	Representative Cohesion, c (psf)		Friction Angle, φ (degrees)		Modulus of Subgrade Reaction, k (pci)		Strain at 50% Max Stress, ε_{50} for Clay Model
									Static	Post-seismic	Static	Post-seismic	Static	Post-seismic	
Oil Storage Tank Area	0	5	5	60	Hf	Reese Sand	125	125	-	-	36	36	225	225	-
	5	50		1	He-1	Soft Clay	100	38	200	150	-	-	-	-	0.02
	50	90		20	Ha	Reese Sand	115	53	-	-	32	10	60	20	-
	90	155		1	He-2	Soft Clay	105	43	200	150	-	-	-	-	0.02
	155	180		90	Qva	Reese Sand	130	68	-	-	42	42	125	125	-

Notes:
1. The lateral parameters do not consider group effects. P-multipliers should be applied for groups of piles spaced closer than 5 diameters, center-to-center.
2. The lateral parameters shown are for a horizontal ground surface. Sloping ground surface modifications should be included in accordance with Ensoft, Inc.'s recommendations for the LPILE Plus program.
3. Based on nearby borings B-1 through B-5.

pcf = pounds per cubic foot
pci = pounds per cubic inch
psf = pounds per square foot



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Scale in Miles

NOTE

Map adapted from 1:24,000 USGS topographic map of Hoquiam, WA quadrangle, dated 1957, photorevised 1983.

USD Crude-By-Rail Terminal
Port of Grays Harbor
Hoquiam, Washington

VICINITY MAP

August 2013 21-1-21839-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 1

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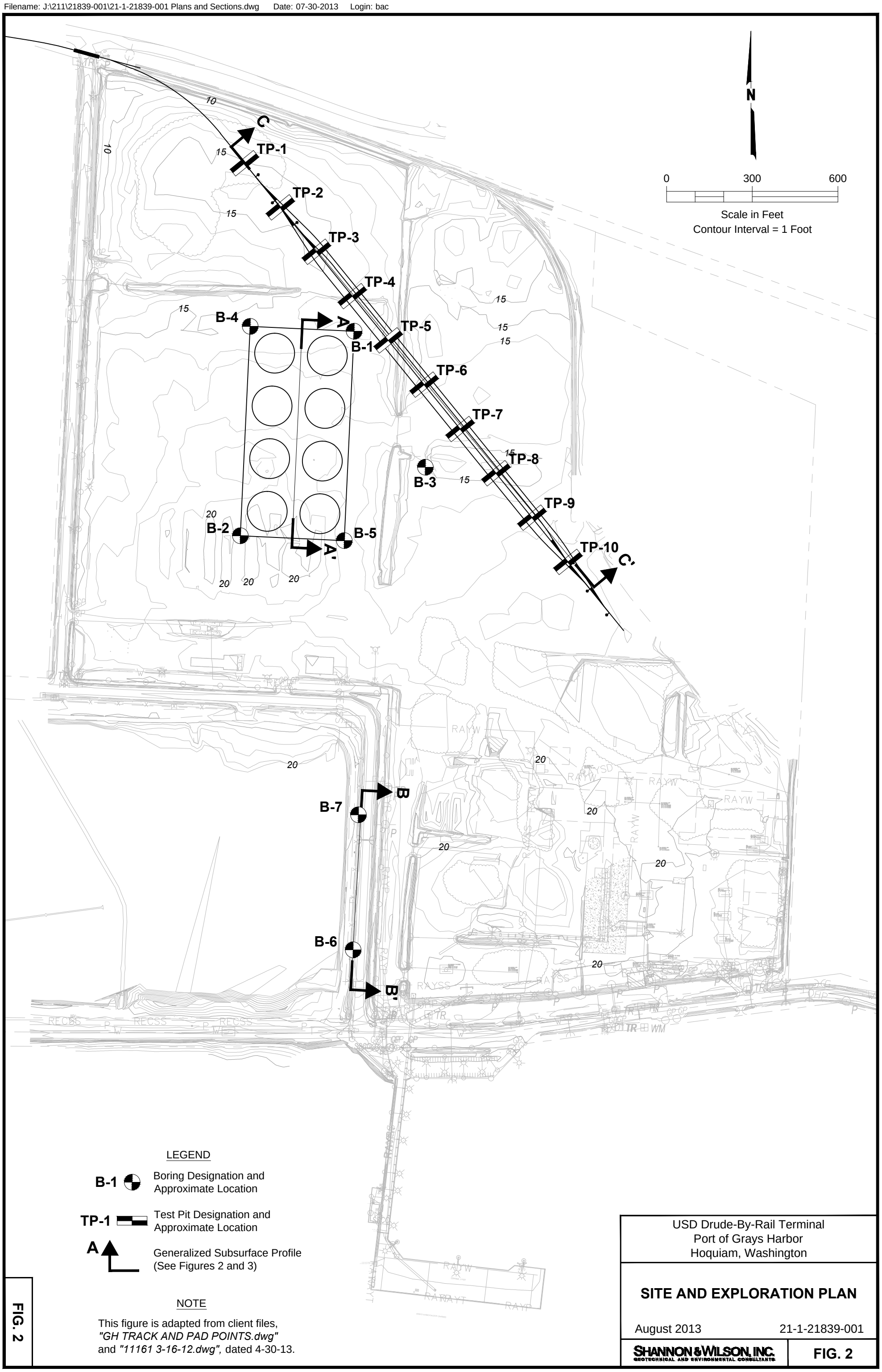
Filename: J:\211\21839-001\21-1-21839-001 Plans and Sections.dwg Date: 07-30-2013 Login: bac



USD Drude-By-Rail Terminal
Port of Grays Harbor
Hoquiam, Washington

SITE AND EXPLORATION PLAN

August 2013 21-1-21839-001

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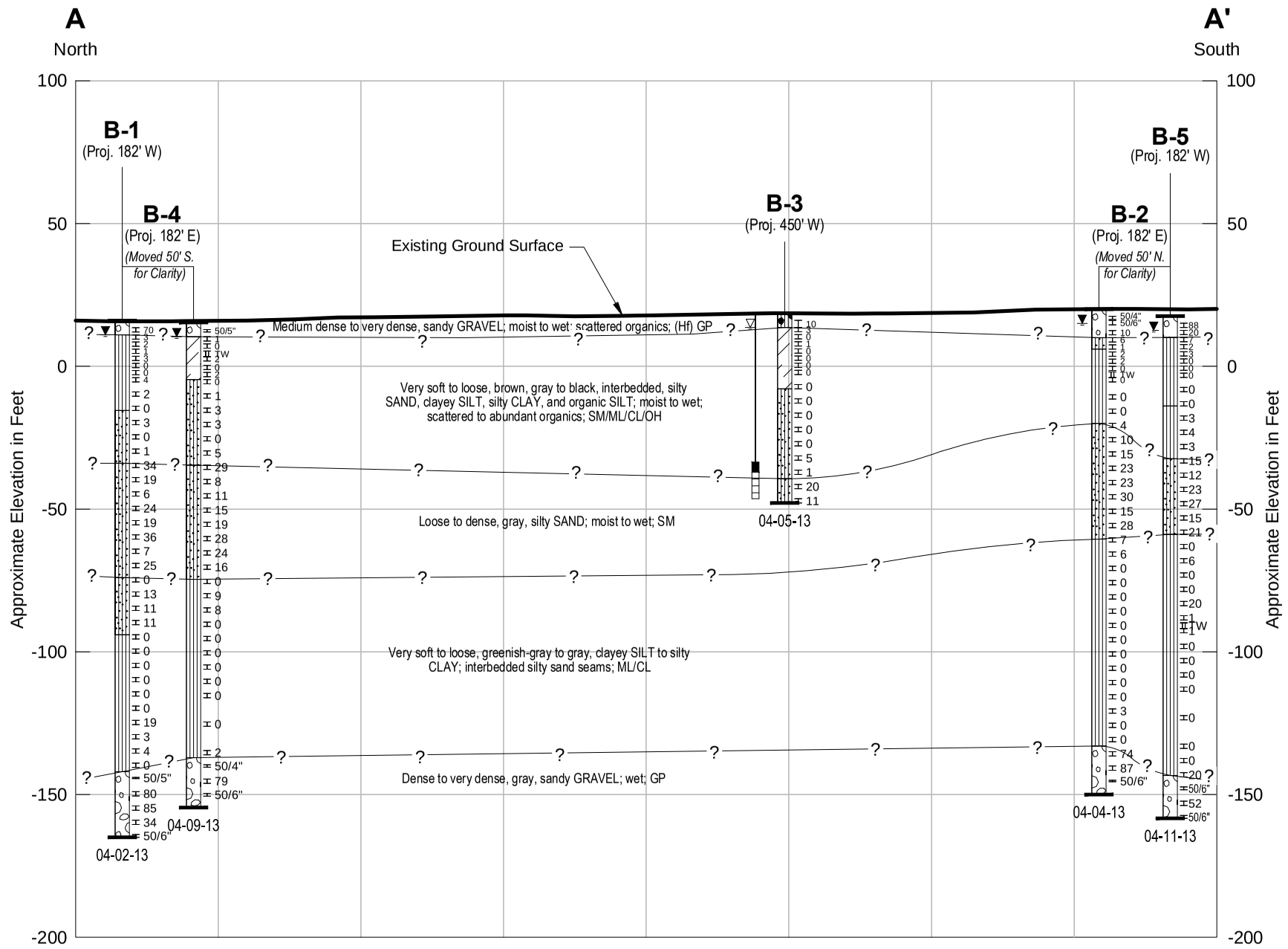
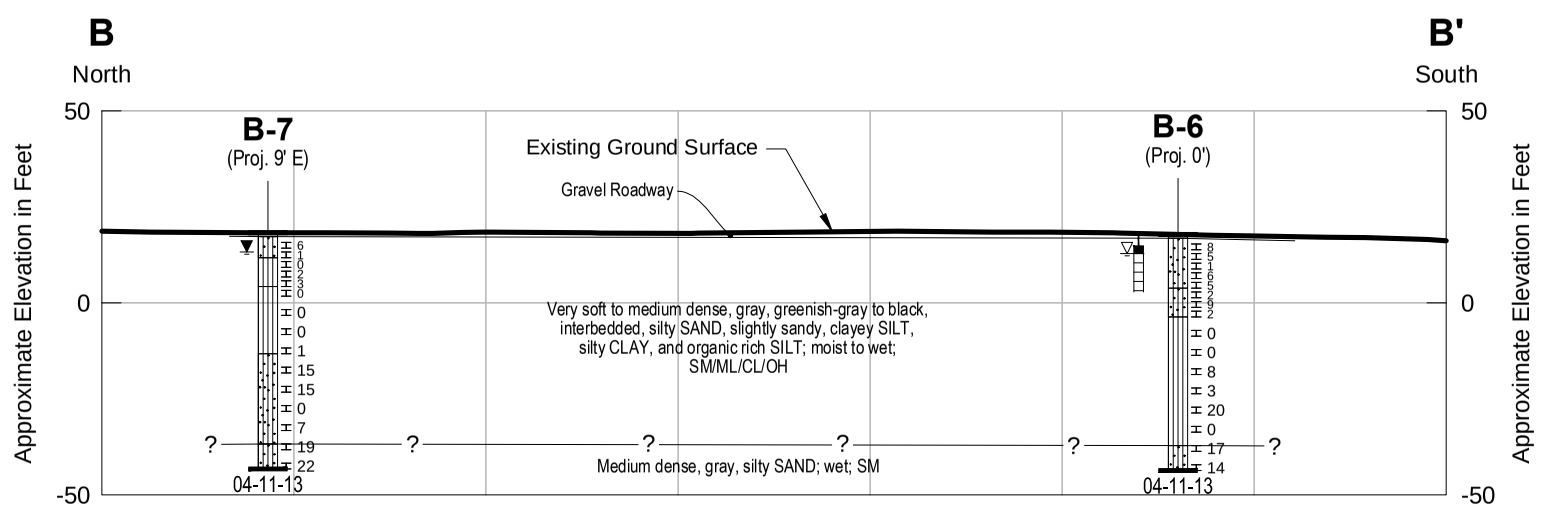
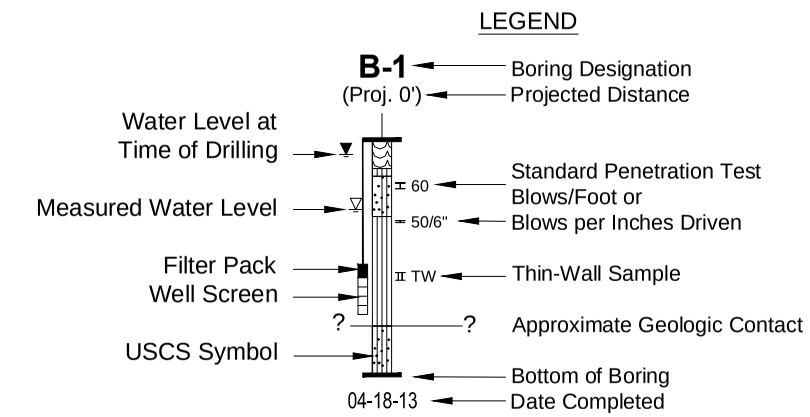
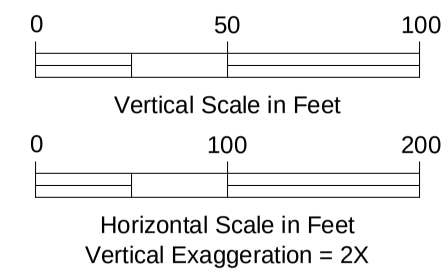
FIG. 2

FIG. 2









NOTES

1. Ground surfaces are adapted from client file, *"11161 3-16-12.dwg",* dated 4-30-2013.

2. These subsurface profiles are generalized from materials observed in soil borings. Variations may exist between profiles and actual conditions.

USD Crude-By-Rail Terminal
Port of Grays Harbor
Hoquiam, Washington

GENERALIZED SUBSURFACE PROFILES A-A' AND B-B'

August 2013 | 21-1-21839-001

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FIG. 3

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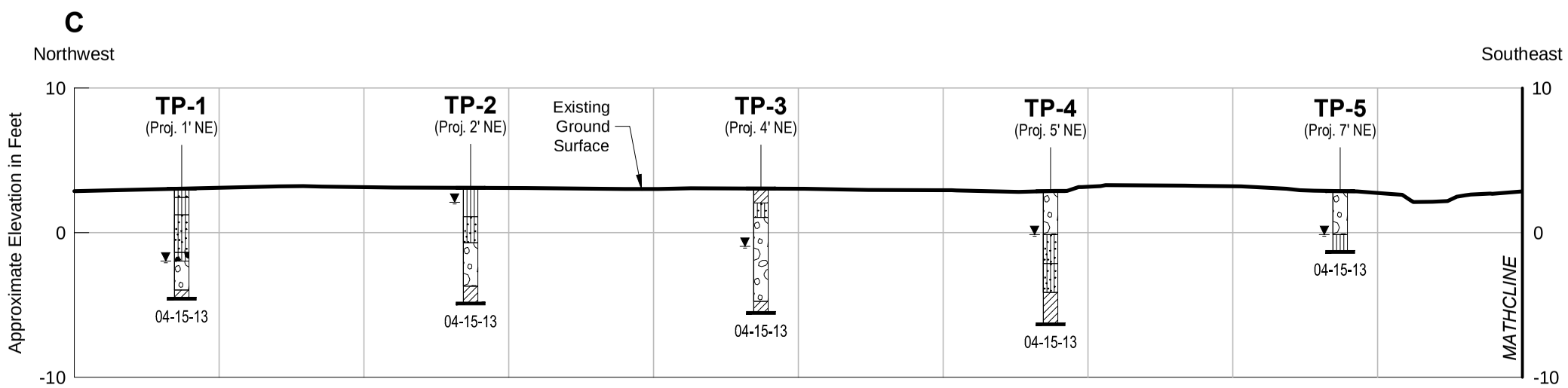

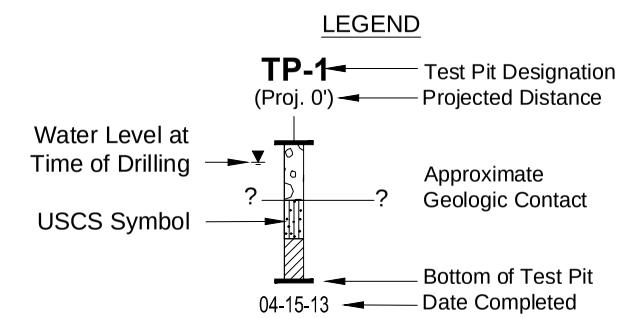

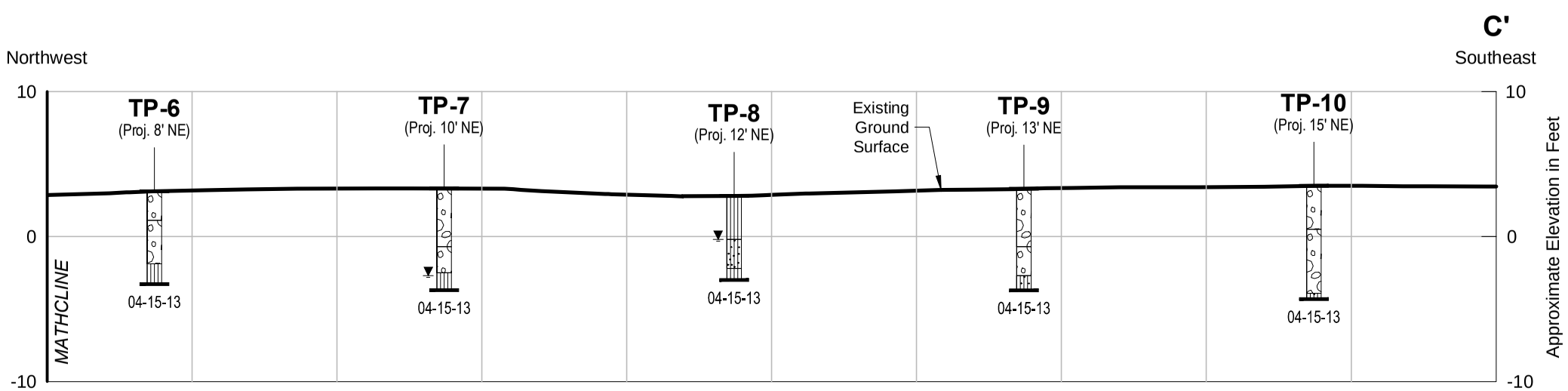

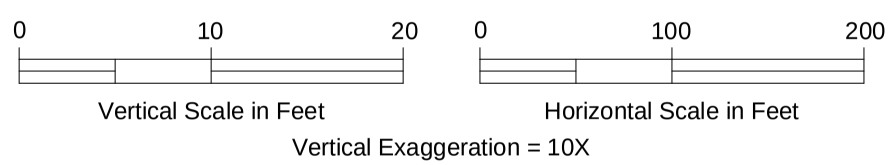

NOTES

1. Ground surface is adapted from client file, *"11161 3-16-12.dwg"*, dated 4-30-2013.
2. This subsurface profile is generalized from materials observed in soil borings. Variations may exist between profile and actual conditions.

USD Crude-By-Rail Terminal
Port of Grays Harbor
Hoquiam, Washington

GENERALIZED SUBSURFACE PROFILE C-C'

August 2013 | 21-1-21839-001

SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

FIG. 4

Filename: J:\211\21839-001\21-1-21839-001 Plans and Sections.dwg Date: 07-30-2013 Login: bac

APPENDIX A

BORING AND TEST PIT DATA

APPENDIX A

BORING AND TEST PIT DATA

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Shannon & Wilson, Inc. (S&W), uses a soil classification system modified from the Unified Soil Classification System (USCS). Elements of the USCS and other definitions are provided on this and the following page. Soil descriptions are based on visual-manual procedures (ASTM D2488-93) unless otherwise noted.

S&W CLASSIFICATION OF SOIL CONSTITUENTS

- MAJOR constituents compose more than 50 percent, by weight, of the soil. Major consituents are capitalized (i.e., SAND).
- Minor constituents compose 12 to 50 percent of the soil and precede the major constituents (i.e., silty SAND). Minor constituents preceded by "slightly" compose 5 to 12 percent of the soil (i.e., slightly silty SAND).
- Trace constituents compose 0 to 5 percent of the soil (i.e., slightly silty SAND, trace of gravel).

MOISTURE CONTENT DEFINITIONS

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

ABBREVIATIONS

ATD	At Time of Drilling
Elev.	Elevation
ft	feet
FeO	Iron Oxide
MgO	Magnesium Oxide
HSA	Hollow Stem Auger
ID	Inside diameter
in	inches
lbs	pounds
Mon.	Monument cover
N	Blows for last two 6-inch increments
NA	Not applicable or not available
NAD	North American Datum (year)
NAVD	North American Vertical Datum (year)
NGVD	National Geodetic Vertical Datum (year)
NP	Non plastic
OD	Outside diameter
OVA	Organic vapor analyzer
PID	Photo-ionization detector
ppm	parts per million
PVC	Polyvinyl Chloride
SS	Split spoon sampler
SPT	Standard penetration test
USC	Unified soil classification
WOH	Weight of hammer
WOR	Weight of drill rods

GRAIN SIZE DEFINITION

DESCRIPTION	SIEVE NUMBER AND/OR SIZE
FINES	< #200 (0.08 mm)
SAND*	
- Fine	#200 to #40 (0.08 to 0.4 mm)
- Medium	#40 to #10 (0.4 to 2 mm)
- Coarse	#10 to #4 (2 to 5 mm)
GRAVEL*	
- Fine	#4 to 3/4 inch (5 to 19 mm)
- Coarse	3/4 to 3 inches (19 to 76 mm)
COBBLES	3 to 12 inches (76 to 305 mm)
BOULDERS	> 12 inches (305 mm)

* Unless otherwise noted, sand and gravel, when present, range from fine to coarse in grain size.

RELATIVE DENSITY / CONSISTENCY

COARSE-GRAINED SOILS		FINE-GRAINED SOILS	
N, SPT, BLOWS/FT.	RELATIVE DENSITY	N, SPT, BLOWS/FT.	RELATIVE CONSISTENCY
0 - 4	Very loose	Under 2	Very soft
4 - 10	Loose	2 - 4	Soft
10 - 30	Medium dense	4 - 8	Medium stiff
30 - 50	Dense	8 - 15	Stiff
Over 50	Very dense	15 - 30	Very stiff
		Over 30	Hard

WELL AND OTHER SYMBOLS

Bent. Cement Grout	Surface Cement Seal
Bentonite Grout	Asphalt or Cap
Bentonite Chips	Slough
Silica Sand	Bedrock
PVC Screen	
Vibrating Wire	

USD Crude-by-Rail Project
Port of Grays Harbor
Hoquiam, Washington

SOIL CLASSIFICATION AND LOG KEY

September 2013 21-1-21839-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-1
Sheet 1 of 2

BORING_CLASS1 21-21839.GPJ SWNEW.GDT 3/24/14

UNIFIED SOIL CLASSIFICATION SYSTEM (USCS)
(From USACE Tech Memo 3-357)

MAJOR DIVISIONS			GROUP/GRAPHIC SYMBOL		TYPICAL DESCRIPTION
COARSE-GRAINED SOILS *(more than 50% retained on No. 200 sieve)*	Gravels *(more than 50% of coarse fraction retained on No. 4 sieve)*	Clean Gravels *(less than 5% fines)*	GW		Well-graded gravels, gravels, gravel/sand mixtures, little or no fines.
			GP		Poorly graded gravels, gravel-sand mixtures, little or no fines
		Gravels with Fines *(more than 12% fines)*	GM		Silty gravels, gravel-sand-silt mixtures
			GC		Clayey gravels, gravel-sand-clay mixtures
	Sands *(50% or more of coarse fraction passes the No. 4 sieve)*	Clean Sands *(less than 5% fines)*	SW		Well-graded sands, gravelly sands, little or no fines
			SP		Poorly graded sand, gravelly sands, little or no fines
		Sands with Fines *(more than 12% fines)*	SM		Silty sands, sand-silt mixtures
			SC		Clayey sands, sand-clay mixtures
FINE-GRAINED SOILS *(50% or more passes the No. 200 sieve)*	Silts and Clays *(liquid limit less than 50)*	Inorganic	ML		Inorganic silts of low to medium plasticity, rock flour, sandy silts, gravelly silts, or clayey silts with slight plasticity
			CL		Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
		Organic	OL		Organic silts and organic silty clays of low plasticity
	Silts and Clays *(liquid limit 50 or more)*	Inorganic	MH		Inorganic silts, micaceous or diatomaceous fine sands or silty soils, elastic silt
			CH		Inorganic clays of medium to high plasticity, sandy fat clay, or gravelly fat clay
		Organic	OH		Organic clays of medium to high plasticity, organic silts
HIGHLY-ORGANIC SOILS	Primarily organic matter, dark in color, and organic odor		PT		Peat, humus, swamp soils with high organic content (see ASTM D 4427)

NOTE: No. 4 size = 5 mm; No. 200 size = 0.075 mm

NOTES

1. Dual symbols *(symbols separated by a hyphen, i.e., SP-SM, slightly silty fine SAND)* are used for soils with between 5% and 12% fines or when the liquid limit and plasticity index values plot in the CL-ML area of the plasticity chart.

2. Borderline symbols *(symbols separated by a slash, i.e., CL/ML, silty CLAY/clayey SILT; GW/SW, sandy GRAVEL/gravelly SAND)* indicate that the soil may fall into one of two possible basic groups.

USD Crude-by-Rail Project
Port of Grays Harbor
Hoquiam, Washington

SOIL CLASSIFICATION AND LOG KEY

September 2013 | 21-1-21839-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-1
Sheet 2 of 2

BORING_CLASS2 21-21839.GPJ SWNEW.GDT 3/24/14

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

LOG OF TEST PIT TP-1 (HDR #5)

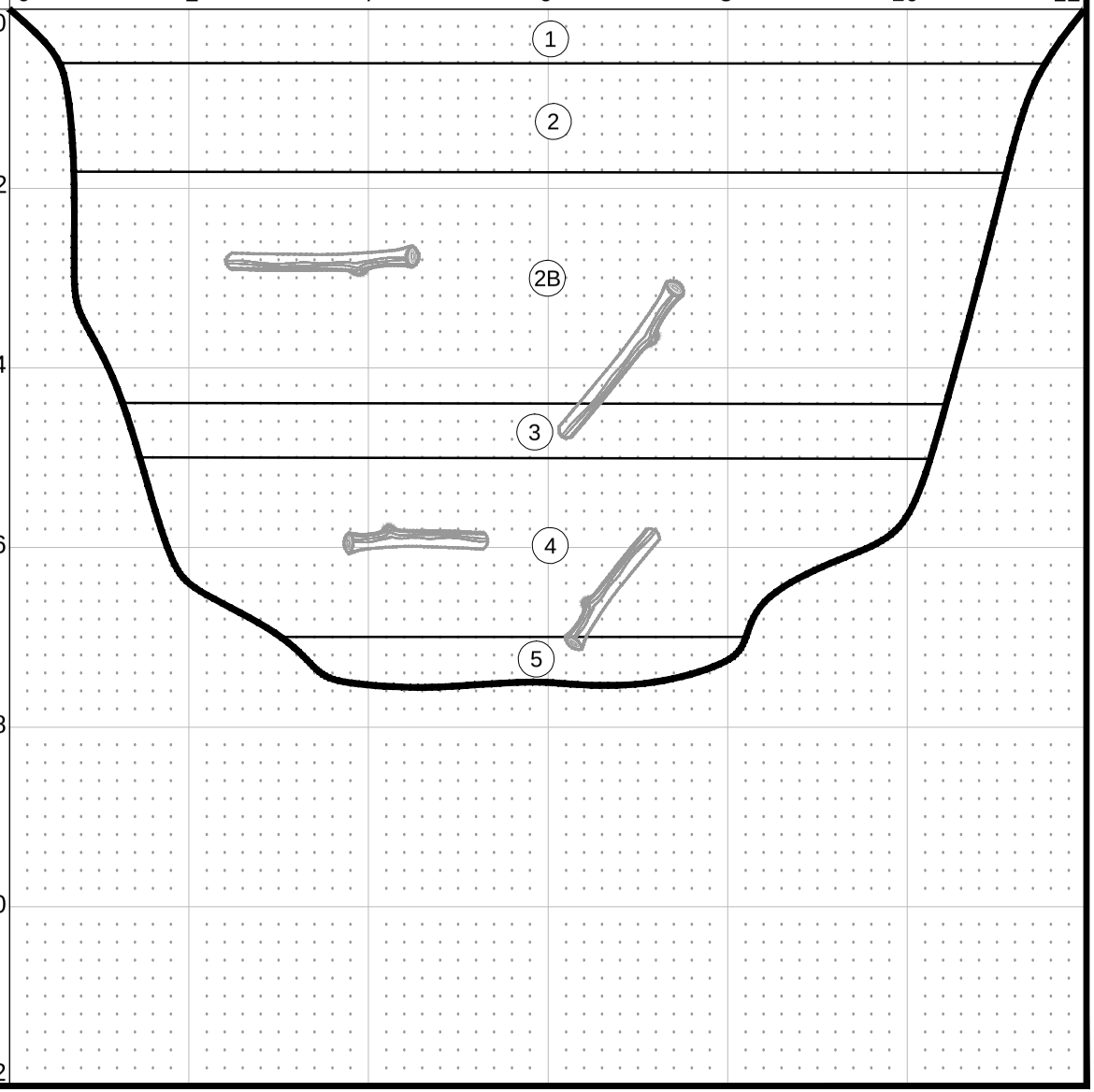
JOB NO: 21-1-21839-001 DATE: 4-15-13 LOCATION: See Site and Exploration Plan

PROJECT: Port of Grays Harbor Terminal

SOIL DESCRIPTION	Ground Water	% Water Content	Samples	Depth, Ft.
(1) Very loose, reddish-brown, gravelly, silty SAND; moist ; abundant root fragments; (Hf) SM.			S-1	0
(2) Very soft, reddish-brown, slightly sandy, clayey SILT; moist; (Hf) ML.			S-2	2
(3) Very loose, brown, slightly clayey, silty SAND; moist; abundant wood chips / organics; (Hf).			S-3	
(4) Medium dense, gray, sandy, silty GRAVEL; wet; gravel is quarry spalls; (Hf) GM.	▽		S-4	4
(5) Very loose, brown, sandy GRAVEL; wet; gravel is quarry spalls; abundant timber and wood chips; GP.			S-5	6
(6) Very soft, light brown, silty CLAY; moist; wood fragments; (Holocene Tidal Deposits).			S-6	8
				10
				12

Sketch of North Pit Side

Surface Elevation: Approx. ___ Ft.



NOTES

1. Abundant 4'- to 6'-long timber and wood chips from 2.5 to 4.4 and 5 to 7 feet.
2. Groundwater filled excavation up to 5 feet below ground surface.

FIG. A-2

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

LOG OF TEST PIT TP-2 (HDR #6)

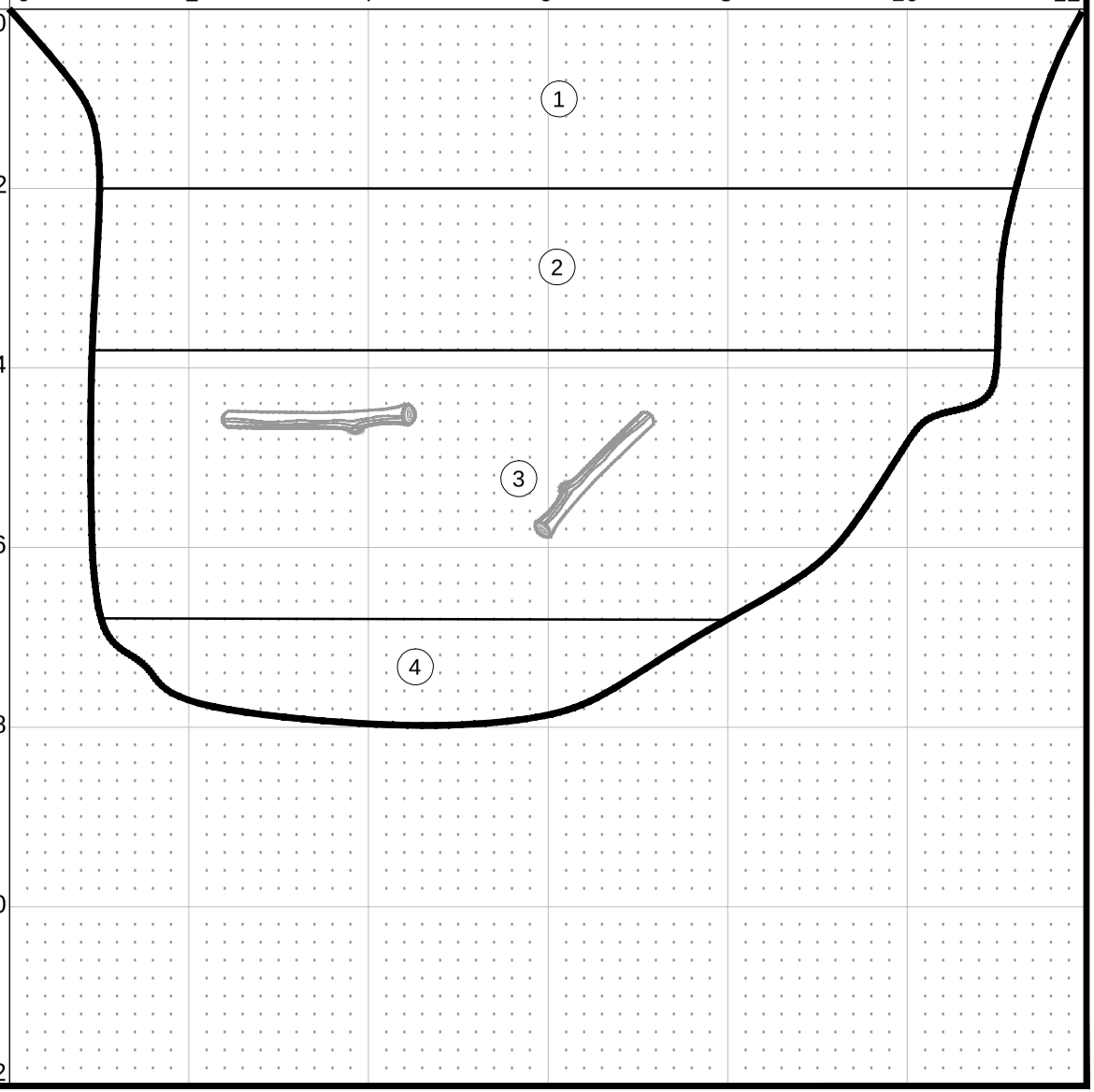
JOB NO: 21-1-21839-001 DATE: 4-15-13 LOCATION: See Site and Exploration Plan

PROJECT: Port of Grays Harbor Terminal

SOIL DESCRIPTION	Ground Water	% Water Content	Samples	Depth, Ft.
(1) Very soft, reddish-brown, slightly sandy, gravelly, clayey SILT; (Hf) ML.			S-1	0
(2) Medium dense, gray, silty, gravelly SAND; moist; gravel is quarry spalls; (Hf) SM.			S-2	2
(3) Very loose, brown, wood waste with GRAVEL; wet; scattered logs and wood chips; (Hf) GP.	▽		S-3	4
(4) Very soft, light brown, slightly sandy, silty CLAY; moist; CL.			S-4	6
				8
				10
				12

Sketch of North Pit Side

Surface Elevation: Approx. ___ Ft.



NOTES

1. Wood waste layer 3.8 to 6.8 feet contains scattered 2'- to 6'-long logs.
2. Groundwater filled excavation to 4 feet below ground surface.

FIG. A-3

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

LOG OF TEST PIT TP-3 (HDR #7)

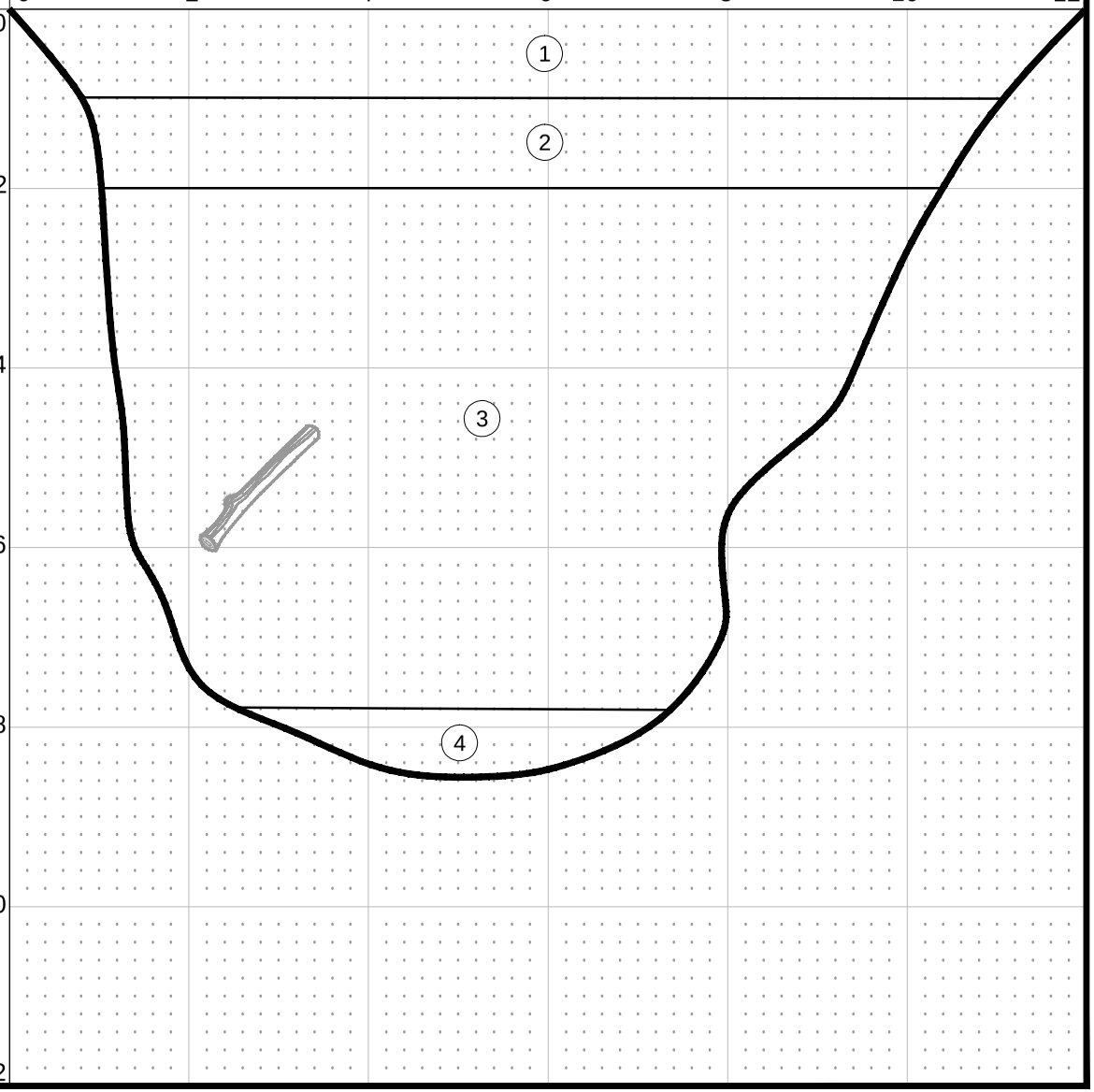
JOB NO: 21-1-21839-001 DATE: 4-15-13 LOCATION: See Site and Exploration Plan

PROJECT: Port of Grays Harbor Terminal

SOIL DESCRIPTION	Ground Water	% Water Content	Samples	Depth, Ft.
(1) Very soft, reddish-brown, slightly sandy, gravelly, silty CLAY; moist; (Hf) CL.			S-1	0
(2) Medium dense, gray, silty, gravelly SAND/sandy GRAVEL; moist; gravel is quarry spalls; (Hf) SM/GM.			S-2	2
(3) Very loose, brown, wood waste with sandy GRAVEL; wet; scattered 2'- to 4'-long logs; (Hf) GP.			S-3	4
(4) Very soft, light brown, slightly sandy, silty CLAY; moist; CL.	▽			6
			S-4	8
				10
				12

Sketch of West Pit Side

Surface Elevation: Approx. ___ Ft.



NOTES

1. Wood waste layer contains scattered 2'- to 4'-long logs, less frequent.
2. Groundwater filled excavation to 6.0 feet below ground surface.

FIG. A-4

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

LOG OF TEST PIT TP-4 (HDR #8)

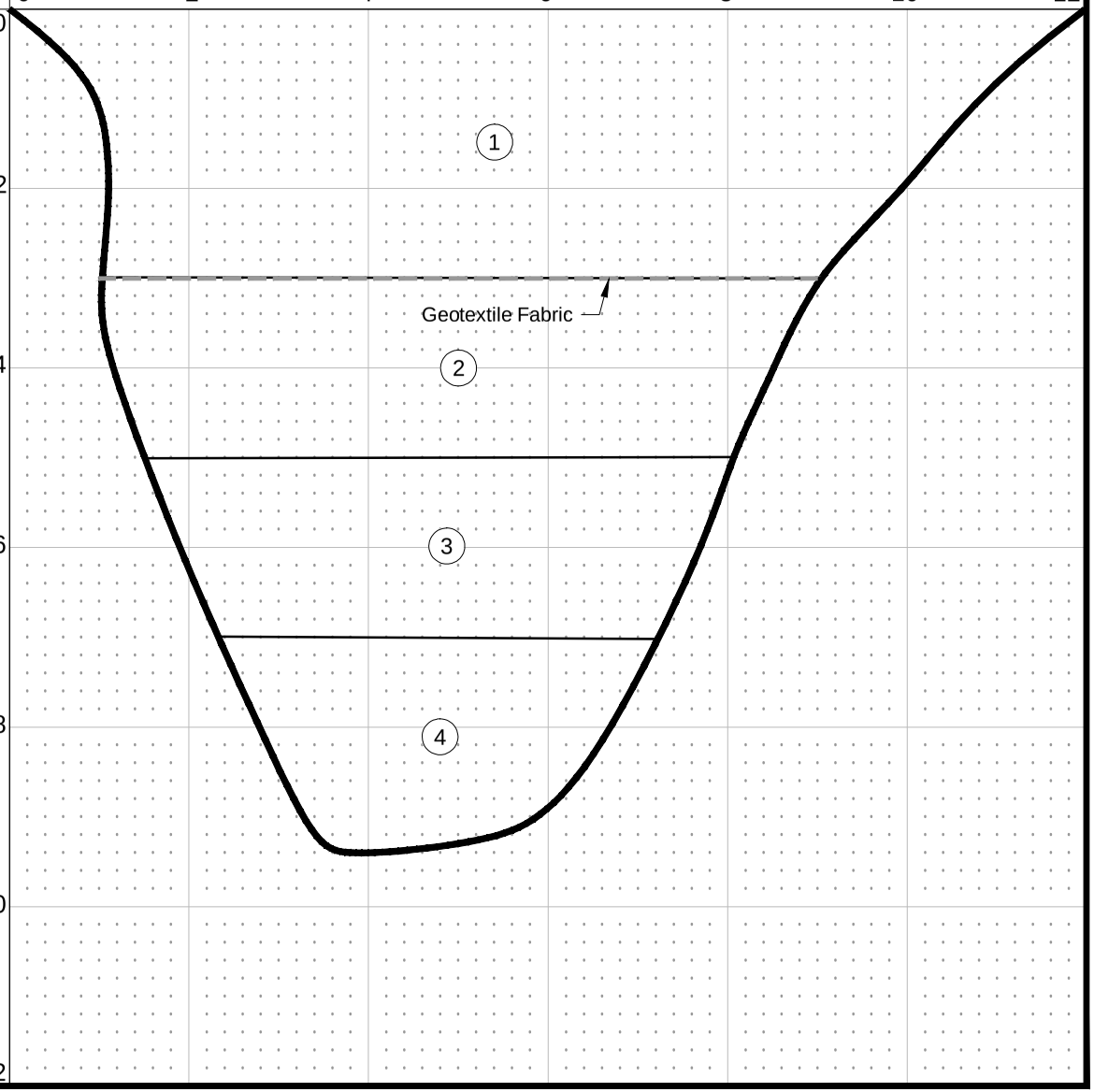
JOB NO: 21-1-21839-001 DATE: 4-15-13 LOCATION: See Site and Exploration Plan

PROJECT: Port of Grays Harbor Terminal

SOIL DESCRIPTION	Ground Water	% Water Content	Samples	Depth, Ft.
(1) Medium dense to dense, brown, sandy GRAVEL; moist; abundant root fragments, gravel is road gravel; (Hf) GP.			S-1	0
(2) Loose, reddish-brown, silty SAND; abundant iron-oxide staining; (Hf) SM.	▽			2
(3) Loose, gray, slightly silty, gravelly SAND; moist to wet; (Hf) SM.			S-2	
(4) Very soft, gray to black, slightly sandy, silty CLAY; moist; (Holocene Tidal Deposits) CL.				4
			S-3	6
			S-4	8
				10
				12

Sketch of West Pit Side

Surface Elevation: Approx. ___ Ft.



NOTES

1. Geotextile fabric at 3.0 feet below ground surface.
2. Groundwater flowed into excavation less than 1 gallon per minute.

FIG. A-5

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

LOG OF TEST PIT TP-5 (HDR #9)

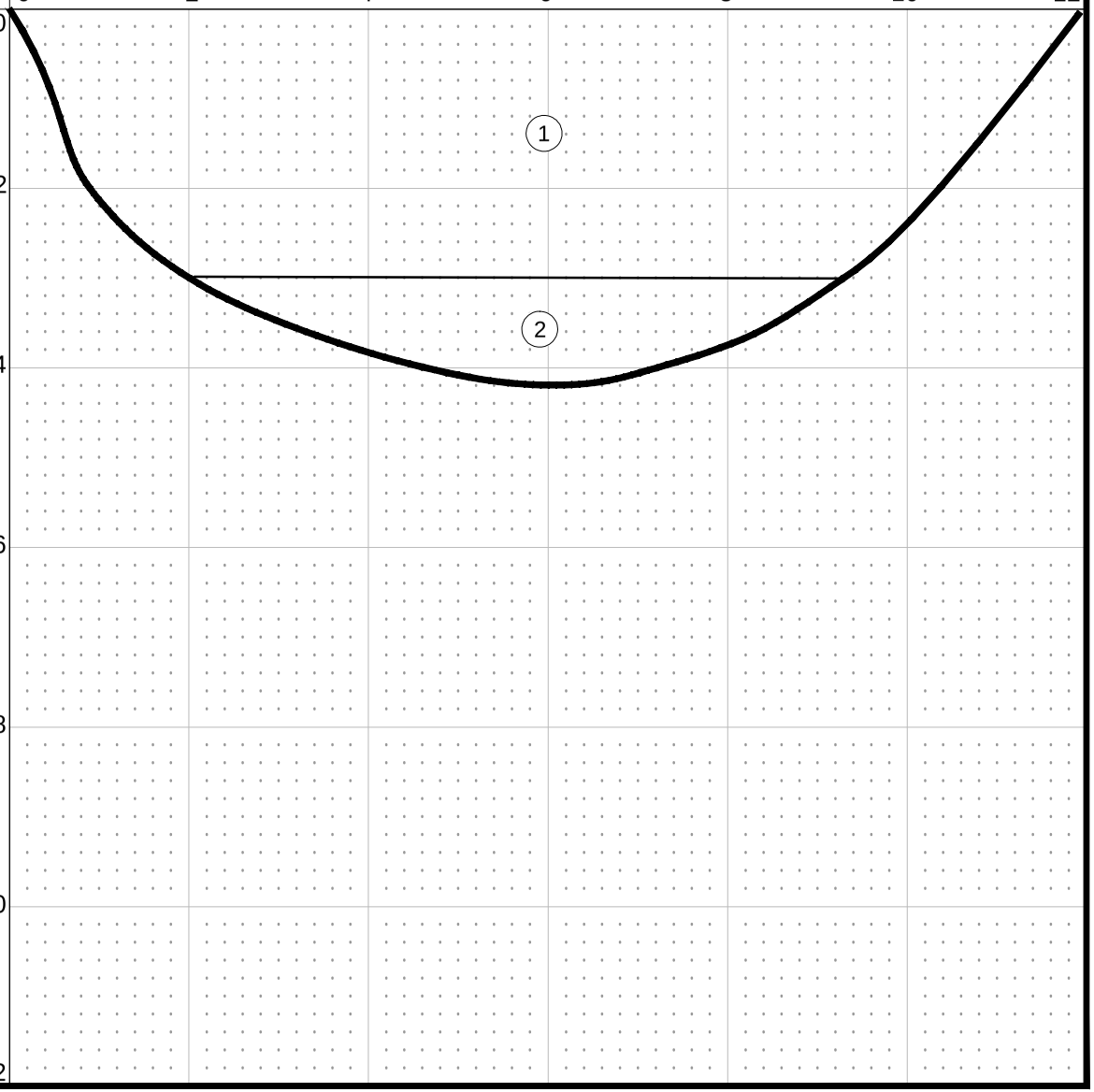
JOB NO: 21-1-21839-001 DATE: 4-15-13 LOCATION: See Site and Exploration Plan

PROJECT: Port of Grays Harbor Terminal

SOIL DESCRIPTION	Ground Water	% Water Content	Samples	Depth, Ft.
(1) Dense, brown, sandy GRAVEL; moist; abundant root fragments, gravel is quarry spalls; (Hf) GP.			S-1	0
(2) Very soft, light gray, slightly sandy, clayey SILT; moist; scattered wood fragments; (Holocene Tidal Deposits) ML.	▽		S-2	2
				4
				6
				8
				10
				12

Sketch of South Pit Side

Surface Elevation: Approx. ___ Ft.



NOTE

Groundwater filled excavation to 3.0 feet below ground surface.

FIG. A-6

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

LOG OF TEST PIT TP-6 (HDR #10)

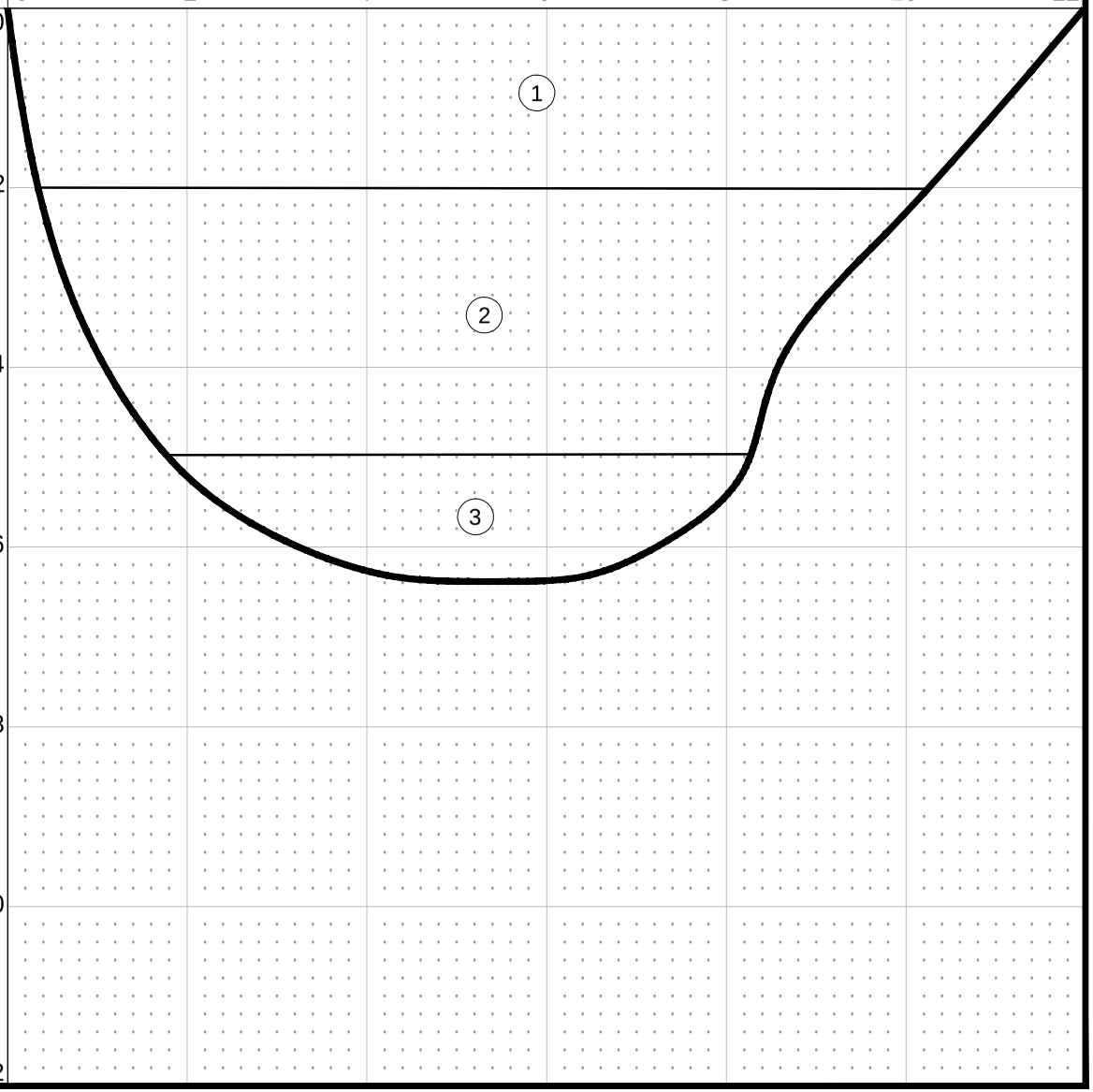
JOB NO: 21-1-21839-001 DATE: 4-15-13 LOCATION: See Site and Exploration Plan

PROJECT: Port of Grays Harbor Terminal

SOIL DESCRIPTION	Ground Water	% Water Content	Samples	Depth, Ft.
(1) Dense, gray, sandy GRAVEL; moist; gravel is crushed rock; (Hf) GP.			S-1	0
(2) Very dense, gray, quarry rock (0.3' to 0.8' diameter) and GRAVEL; (Hf).				2
(3) Very soft, gray to dark gray, slightly sandy, clayey SILT; moist; ML.	▽		S-2	4
			S-3	6
				8
				10
				12

Sketch of South Pit Side

Surface Elevation: Approx. ___ Ft.



NOTE

Groundwater filled excavation to 5.0 feet below ground surface.

FIG. A-7

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

LOG OF TEST PIT TP-7 (HDR #11)

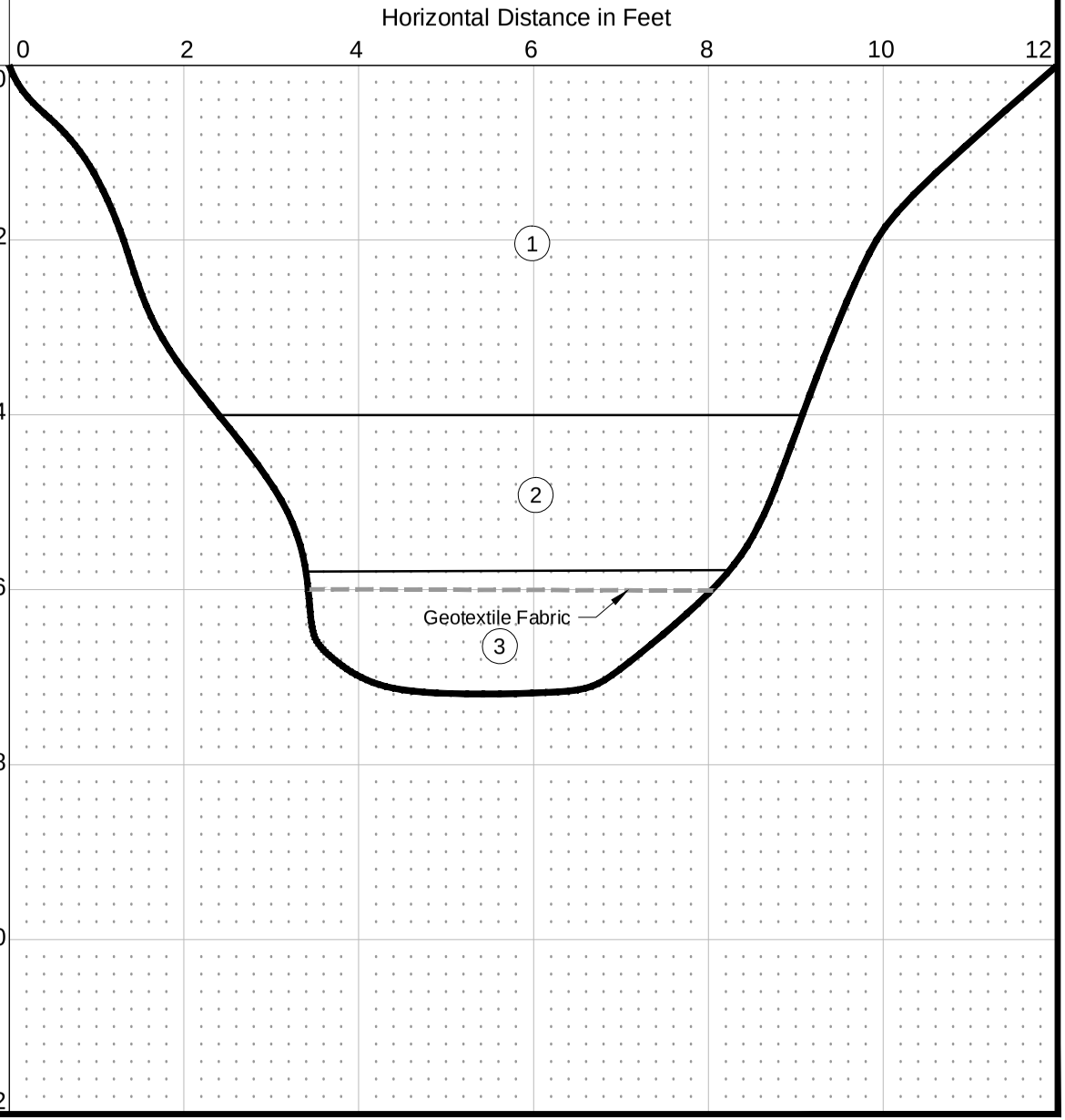
JOB NO: 21-1-21839-001 DATE: 4-15-13 LOCATION: See Site and Exploration Plan

PROJECT: Port of Grays Harbor Terminal

SOIL DESCRIPTION	Ground Water	% Water Content	Samples	Depth, Ft.
(1) Very dense, gray, sandy GRAVEL; moist; abundant root fragments and wood chips; gravel is imported crushed rock; (Hf) GP.			S-1	0
(2) Very dense, gray, crushed quarry rock (0.2' to 0.5' diameter); (Hf).				2
(3) Very soft, light gray, slightly sandy, clayey SILT; moist; (Holocene Tidal Deposits) ML.			S-2	4
	▽		S-3	6
				8
				10
				12

Sketch of Northwest Pit Side

Surface Elevation: Approx. ___ Ft.



NOTES

1. Geotextile fabric at 6.0 feet.
2. Groundwater filled excavation to 6.0 feet below ground surface.

FIG. A-8

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

LOG OF TEST PIT TP-8 (HDR #12)

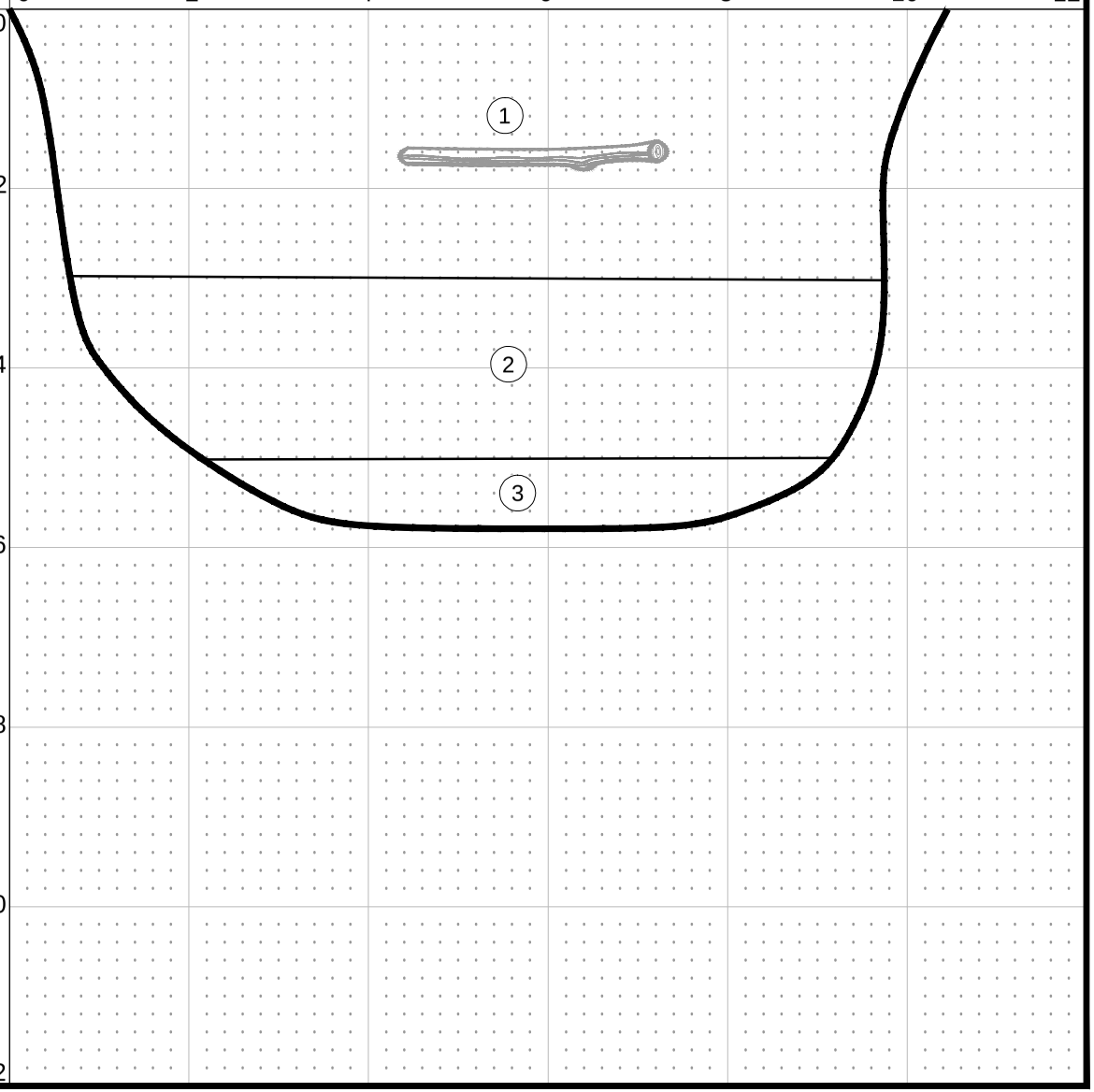
JOB NO: 21-1-21839-001 DATE: 4-15-13 LOCATION: See Site and Exploration Plan

PROJECT: Port of Grays Harbor Terminal

SOIL DESCRIPTION	Ground Water	% Water Content	Samples	Depth, Ft.
(1) Very loose, brown wood chips and wood fragments; scattered 4' to 6' logs.			S-1	0
(2) Loose, gray, silty SAND; moist; to wet; SM.				2
(3) Very soft, gray, sandy, clayey SILT; moist; scattered root fragments; ML.	▽		S-2	4
			S-3	6
				8
				10
				12

Sketch of West Pit Side

Surface Elevation: Approx. ___ Ft.



NOTE

Groundwater flow less than 1.0 gallon per minute into excavation.

FIG. A-9

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

LOG OF TEST PIT TP-9 (HDR #13)

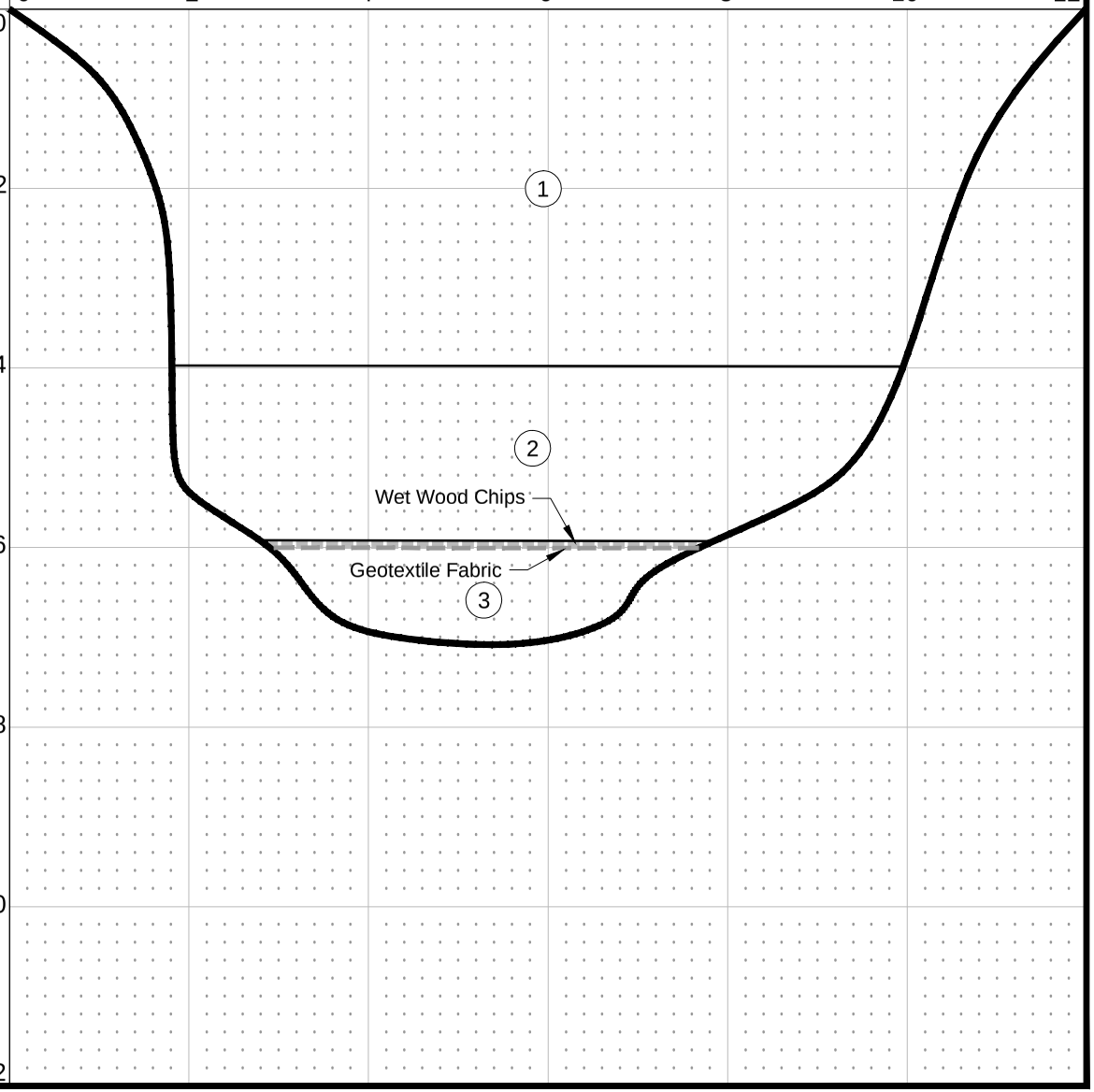
JOB NO: 21-1-21839-001 DATE: 4-15-13 LOCATION: See Site and Exploration Plan

PROJECT: Port of Grays Harbor Terminal

SOIL DESCRIPTION	Ground Water	% Water Content	Samples	Depth, Ft.
(1) Dense, brown, sandy GRAVEL; moist; gravel is imported crushed rock; (Hf) GP.			S-1	0
(2) Very dense, gray, imported crushed quarry rock; wet; GP.	▽			2
(3) Loose, gray, silty SAND; wet; SM.			S-2	4
			S-3	6
			S-4	
				8
				10
				12

Sketch of West Pit Side

Surface Elevation: Approx. ___ Ft.



NOTES

1. Geotextile fabric at 6.0 feet.
2. Groundwater filled excavation to 2.0 feet below ground surface.

FIG. A-10

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

LOG OF TEST PIT TP-10 (HDR #14)

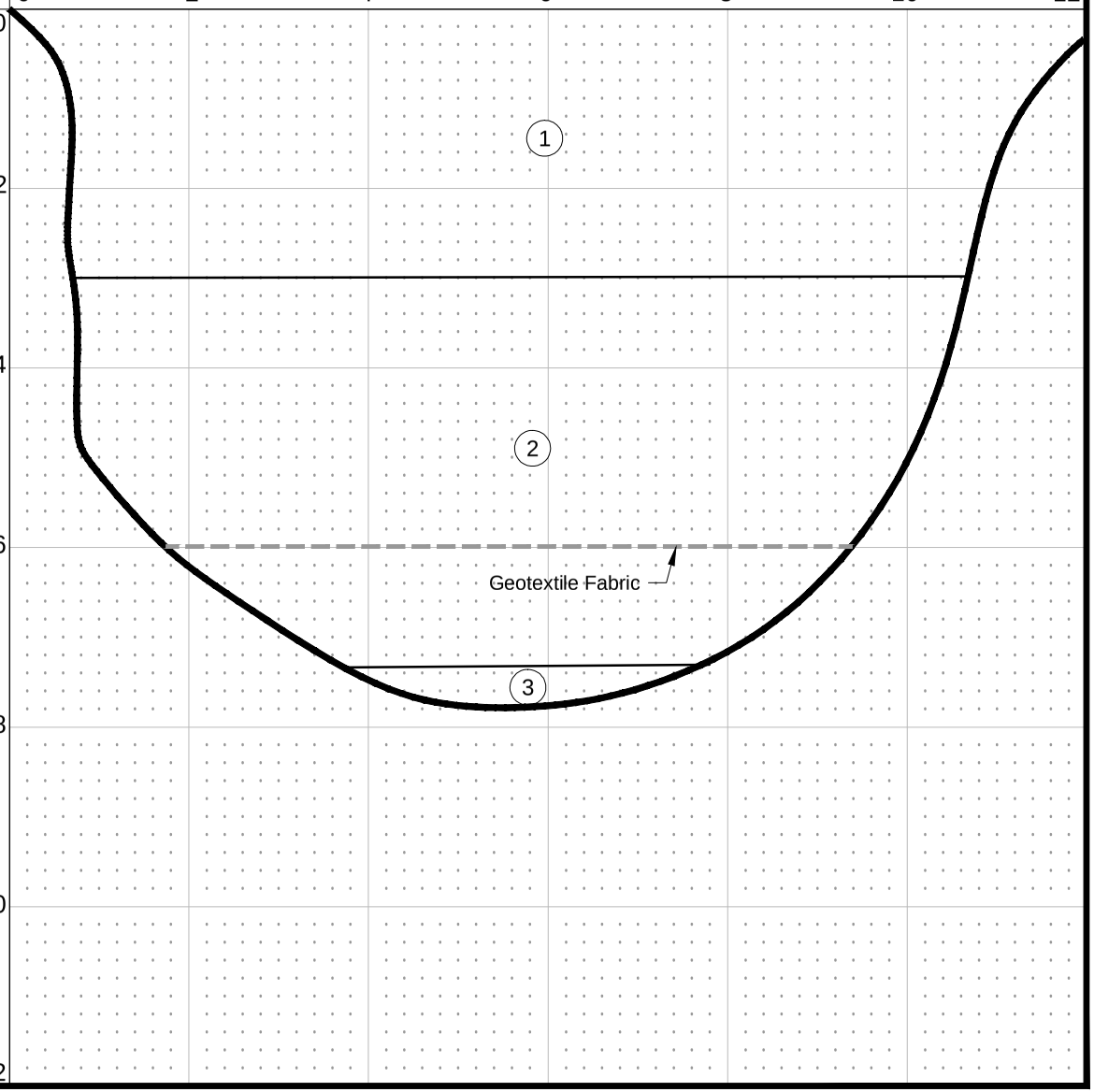
JOB NO: 21-1-21839-001 DATE: 4-15-13 LOCATION: See Site and Exploration Plan

PROJECT: Port of Grays Harbor Terminal

SOIL DESCRIPTION	Ground Water	% Water Content	Samples	Depth, Ft.
(1) Dense, brown, slightly silty, sandy GRAVEL; moist; gravel is imported crushed rock; (Hf) GP.			S-1	0
(2) Very dense, gray, imported crushed quarry rock; moist to wet; (Hf) GP.				2
(3) Loose, gray, silty SAND; moist to wet; SM.			S-2	4
	▽			6
			S-3	8
				10
				12

Sketch of West Pit Side

Surface Elevation: Approx. ___ Ft.



NOTES

1. Geotextile fabric at 6.0 feet.
2. Groundwater flow less than 1 gallon per minute into excavation.

FIG. A-11

Total Depth:	181 ft.	Northing:		Drilling Method:	HSA and Mud Rotary	Hole Diam.:	6 in.
Top Elevation:	~	Easting:		Drilling Company:	Gregory Drilling	Rod Diam.:	
Vert. Datum:		Station:		Drill Rig Equipment:	Track CME	Hammer Type:	Automatic
Horiz. Datum:		Offset:		Other Comments:			

SOIL DESCRIPTION

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

Very dense, gray to brown, sandy GRAVEL; moist; (Hf) GP.

5.0

Very soft to soft, greenish-gray to gray-black, clayey SILT to silty CLAY; moist; scattered interbedded, silty sand at 8 feet and from 30 to 31.5 feet; organic clay from 12.5 to 17 feet; (Estuarine Deposits) (He) ML/CL/OH.

31.5

Very loose to very soft, greenish-gray, interbedded, silty SAND with clayey SILT; moist to wet; (Estuarine Deposits) (He) SM/ML.

50.0

Loose to dense, gray, silty SAND; moist to wet; sandy silt interbed from 80 to 81.5 feet, scattered wood fragments from 60 to 61.5 feet; (Alluvium) (Ha) SM.

90.0

Very soft to medium dense, gray, interbedded, silty SAND and sandy SILT with silt layer from 90 to 91.5 feet; (Alluvium) (Ha) SM/ML.

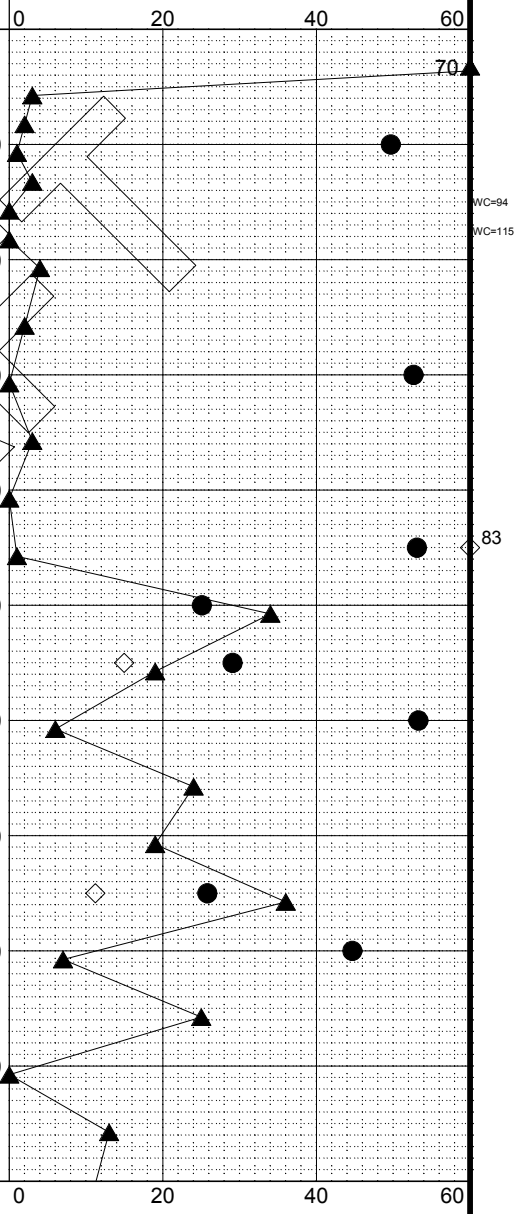
CONTINUED NEXT SHEET

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

During Drilling

PENETRATION RESISTANCE (blows/foot)

▲ Hammer Wt. & Drop: 140 lbs / 30 inches



LEGEND

* Sample Not Recovered

2.0" O.D. Split Spoon Sample

Ground Water Level ATD

◇ % Fines (<0.075mm)

● % Water Content

Plastic Limit ├─●─┤ Liquid Limit

Natural Water Content

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.

USD Crude-by-Rail Project
Port of Grays Harbor
Hoquiam, Washington

LOG OF BORING B-1

September 2013 | 21-1-21839-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-12
Sheet 1 of 2

DRAFT

Total Depth:	181 ft.	Northing:		Drilling Method:	HSA and Mud Rotary	Hole Diam.:	6 in.
Top Elevation:	~	Easting:		Drilling Company:	Gregory Drilling	Rod Diam.:	
Vert. Datum:		Station:		Drill Rig Equipment:	Track CME	Hammer Type:	Automatic
Horiz. Datum:		Offset:		Other Comments:			

SOIL DESCRIPTION

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

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

PENETRATION RESISTANCE (blows/foot)

▲ Hammer Wt. & Drop: 140 lbs / 30 inches

Very soft to soft, gray, clayey SILT and silty CLAY with medium dense, silty sand seam from 141 to 141.5 feet; moist; (Estuarine Deposits) (He) ML/CL.

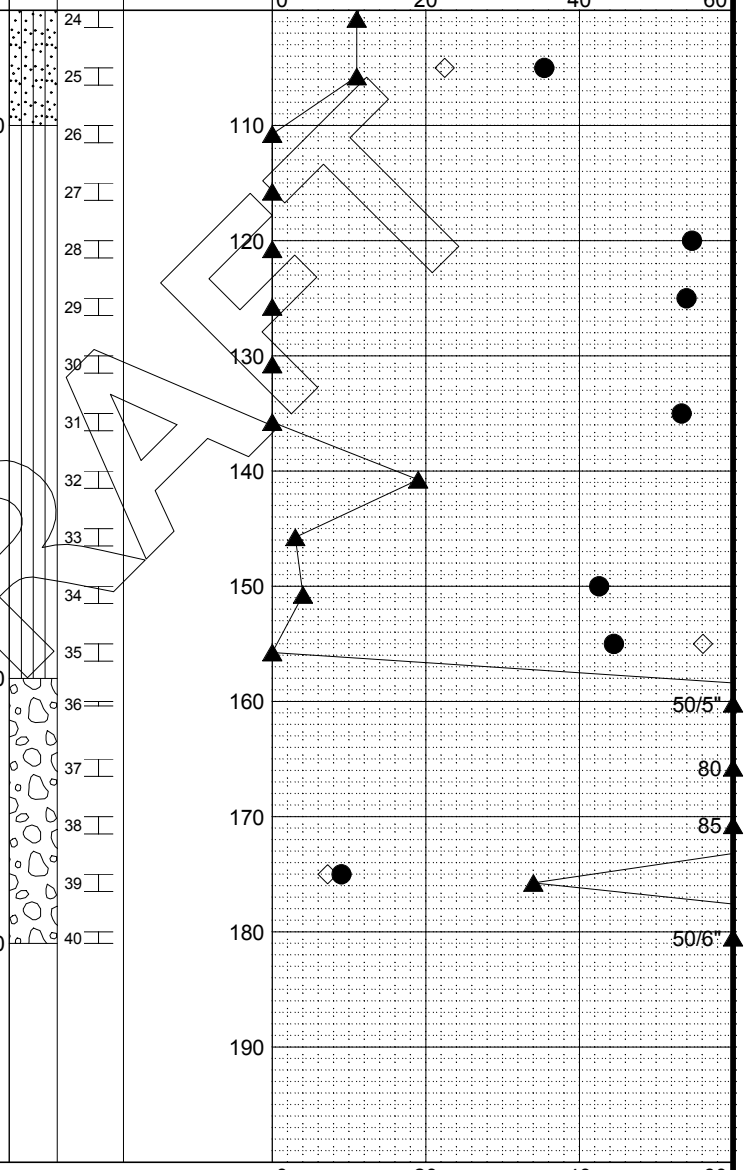
110.0

Dense to very dense, gray, sandy GRAVEL; wet; scattered cobbles; (Recessional Outwash) (Qvro) GP.

158.0

BOTTOM OF BORING COMPLETED 4/2/2013

181.0



LEGEND

* Sample Not Recovered

2.0" O.D. Split Spoon Sample

Ground Water Level ATD

◇ % Fines (<0.075mm)

● % Water Content

Plastic Limit ├─●─┤ Liquid Limit

Natural Water Content

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.

USD Crude-by-Rail Project
Port of Grays Harbor
Hoquiam, Washington

LOG OF BORING B-1

September 2013 | 21-1-21839-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-12
Sheet 2 of 2

DRAFT

Total Depth:	170 ft.	Northing:		Drilling Method:	HSA and Mud Rotary	Hole Diam.:	6 in.
Top Elevation:	~	Easting:		Drilling Company:	Gregory Drilling	Rod Diam.:	
Vert. Datum:		Station:		Drill Rig Equipment:	Track CME	Hammer Type:	Automatic
Horiz. Datum:		Offset:		Other Comments:			

SOIL DESCRIPTION

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

Medium dense to very dense, gray to dark gray, sandy GRAVEL to silty SAND; moist to wet; (Hf) GP/SM.

7.5

Loose to medium dense, gray, silty SAND; moist to wet; organic, silty clay seam from 10 to 10.5 feet; (Estuarine Deposits) (He) SM.

14.0

Very soft, gray to black from 20 to 25.5 feet, sandy SILT to silty CLAY with organic-rich clay from 20 to 25.5 feet (OH); moist; (Alluvium) (Ha) ML/CL.

40.0

Loose to dense, gray, silty SAND; moist to wet; (Estuarine Deposits) (He) SM.

80.5

Loose (from 80.5 to 86.5 feet) to very soft, gray, clayey SILT to silty CLAY with loose, gray, silty SAND layer from 140 to 141.5 feet (SM); moist; (Estuarine Deposits) (He) ML/CL.

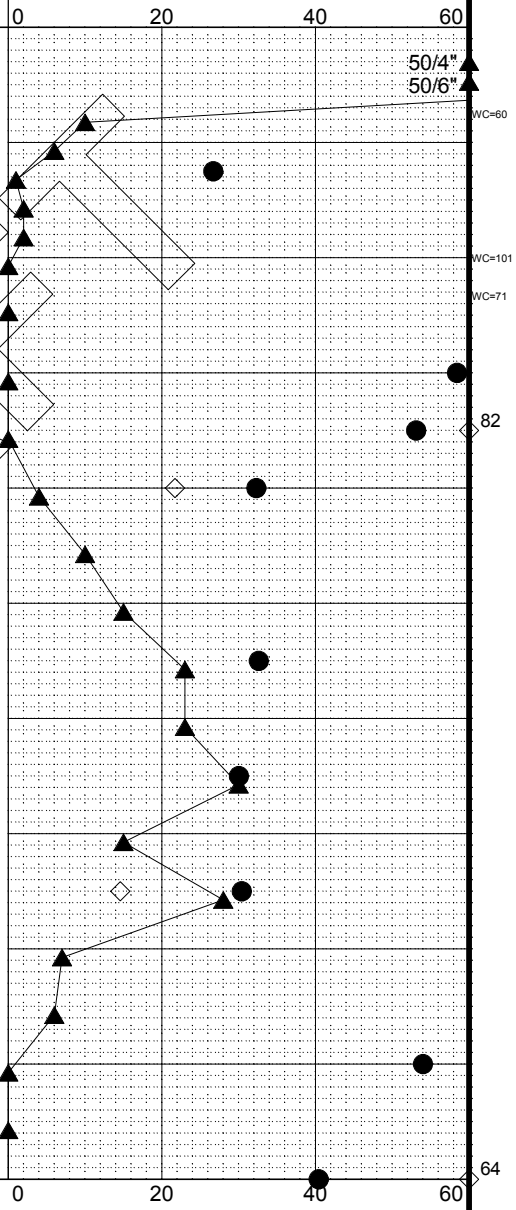
CONTINUED NEXT SHEET

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

During Drilling

PENETRATION RESISTANCE (blows/foot)

▲ Hammer Wt. & Drop: 140 lbs / 30 inches



DRAFT

LEGEND

- * Sample Not Recovered
- 2.0" O.D. Split Spoon Sample
- Thin Wall Sample
- Ground Water Level ATD
- ◇ % Fines (<0.075mm)
- ● % Water Content
- Plastic Limit ├─●─┤ Liquid Limit
- Natural Water Content

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.

USD Crude-by-Rail Project
Port of Grays Harbor
Hoquiam, Washington

LOG OF BORING B-2

September 2013 | 21-1-21839-001

SHANNON & WILSON, INC.
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FIG. A-13
Sheet 1 of 2

MASTER_LOG_E_21-21839.GPJ SHAN_WIL.GDT 3/24/14 Log: AJD Rev: AJD Typ: LKN

Total Depth:	170 ft.	Northing:		Drilling Method:	HSA and Mud Rotary	Hole Diam.:	6 in.
Top Elevation:	~	Easting:		Drilling Company:	Gregory Drilling	Rod Diam.:	
Vert. Datum:		Station:		Drill Rig Equipment:	Track CME	Hammer Type:	Automatic
Horiz. Datum:		Offset:		Other Comments:			

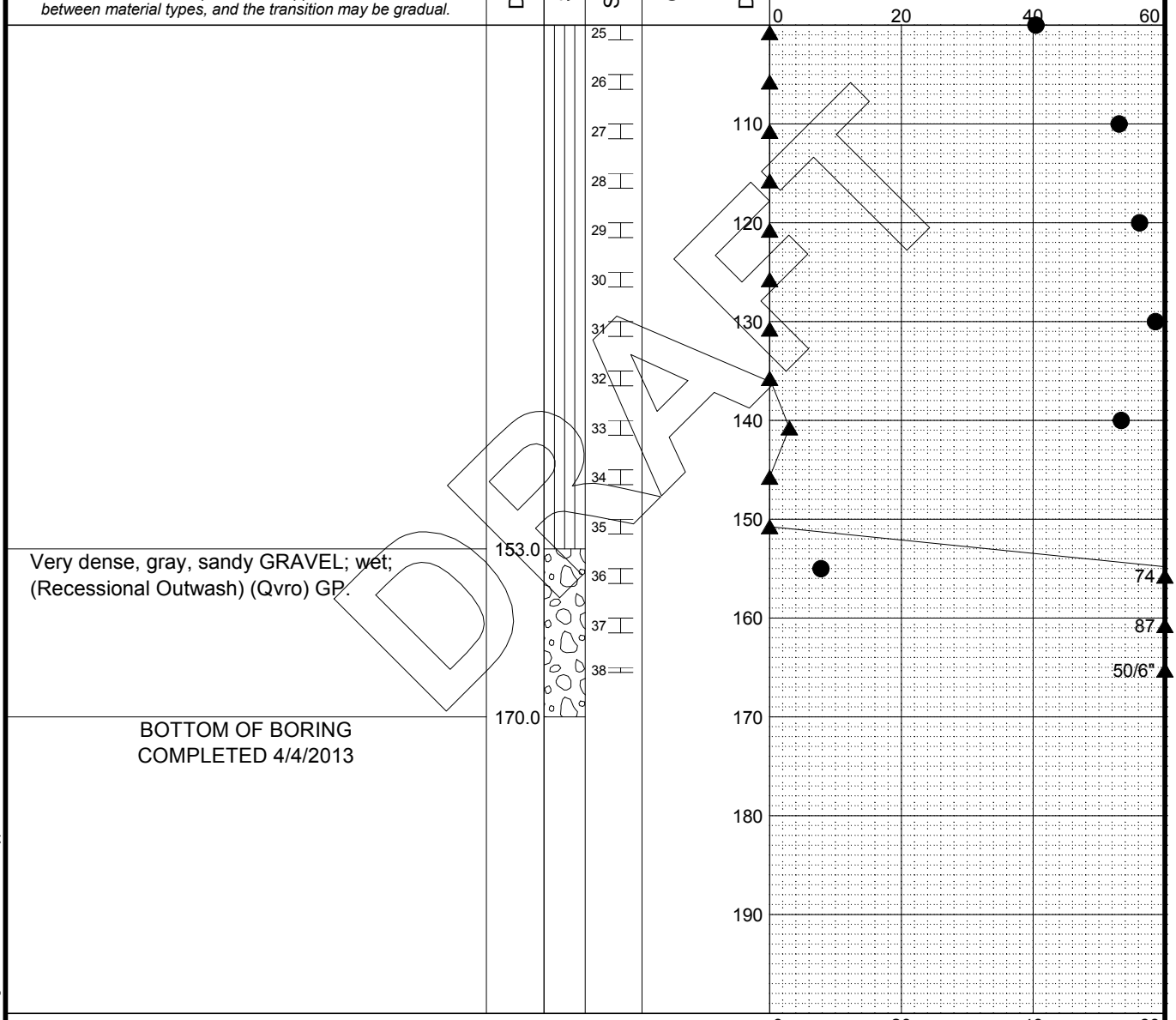
SOIL DESCRIPTION

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

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

PENETRATION RESISTANCE (blows/foot)

▲ Hammer Wt. & Drop: 140 lbs / 30 inches



Very dense, gray, sandy GRAVEL; wet; (Recessional Outwash) (Qvro) GP.

BOTTOM OF BORING
COMPLETED 4/4/2013

DRAFT

LEGEND

- * Sample Not Recovered
- 2.0" O.D. Split Spoon Sample
- Thin Wall Sample
- Ground Water Level ATD
- ◇ % Fines (<0.075mm)
- ● % Water Content
- Plastic Limit ├─●─┤ Liquid Limit
- Natural Water Content

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.

USD Crude-by-Rail Project
Port of Grays Harbor
Hoquiam, Washington

LOG OF BORING B-2

September 2013 | 21-1-21839-001

SHANNON & WILSON, INC.
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FIG. A-13
Sheet 2 of 2

MASTER_LOG_E 21-21839.GPJ SHAN_WIL.GDT 3/24/14 Log: AJD Rev: AJD Typ: LKN

Total Depth:	66.5 ft.	Northing:		Drilling Method:	HSA and Mud Rotary	Hole Diam.:	6 in.
Top Elevation:	~	Easting:		Drilling Company:	Gregory Drilling	Rod Diam.:	
Vert. Datum:		Station:		Drill Rig Equipment:	Track CME	Hammer Type:	Automatic
Horiz. Datum:		Offset:		Other Comments:			

SOIL DESCRIPTION

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

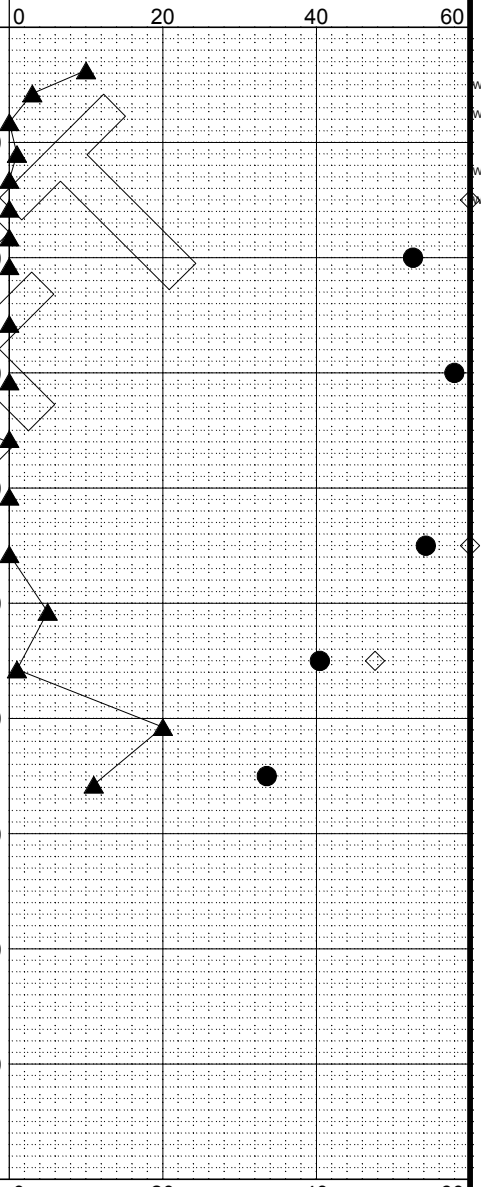
Description	Depth, ft.
Medium dense, gray to black, silty, sandy GRAVEL; wet; (Hf) GM.	5.0
Very soft, dark gray to black, organic-rich, silty CLAY; with wet, silty sand seam from 12.5 to 13 feet; moist; (Estuarine Deposits) (He) OH/CL.	26.5
Very loose to soft, gray, interbedded, silty SAND with clayey SILT; moist; scattered silty clay seams from 35 to 55 feet; (Estuarine Deposits) (He) SM/ML.	58.0
Medium dense, gray, silty SAND; wet; (Alluvium) (Ha) SM.	66.5
BOTTOM OF BORING COMPLETED 4/5/2013	

Samples: 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 13, 14, 15, 16, 17

Ground Water: During Drilling

PENETRATION RESISTANCE (blows/foot)

▲ Hammer Wt. & Drop: 140 lbs / 30 inches



LEGEND

- * Sample Not Recovered
- 2.0" O.D. Split Spoon Sample
- Thin Wall Sample
- Piezometer Screen and Sand Filter
- Bentonite-Cement Grout
- Bentonite Chips/Pellets
- Bentonite Grout
- Ground Water Level ATD

◇ % Fines (<0.075mm)

● % Water Content

Plastic Limit ├─●─┤ Liquid Limit

Natural Water Content

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.

USD Crude-by-Rail Project
Port of Grays Harbor
Hoquiam, Washington

LOG OF BORING B-3

September 2013 | 21-1-21839-001

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FIG. A-14

MASTER_LOG_E 21-21839.GPJ SHAN_WIL.GDT 3/24/14 Log: AJD Rev: AJD Typ: LKN

Total Depth:	170 ft.	Northing:		Drilling Method:	HSA and Mud Rotary	Hole Diam.:	6 in.
Top Elevation:	~	Easting:		Drilling Company:	Gregory Drilling	Rod Diam.:	
Vert. Datum:		Station:		Drill Rig Equipment:	Track CME	Hammer Type:	Automatic
Horiz. Datum:		Offset:		Other Comments:			

SOIL DESCRIPTION

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

Soil Description	Depth, ft.
Very dense, black, sandy GRAVEL; wet; (Hf) GP.	5.0
Very soft, dark gray to black, organic-rich, silty CLAY; moist; (Estuarine Deposits) (He) OH/CL.	20.0
Very loose to very soft, gray, interbedded, silty SAND to clayey SILT/silty CLAY; moist; clay seams from 35 to 50 feet; (Estuarine Deposits) (He) SM/ML.	50.0
Loose to medium dense, gray, silty SAND with scattered sandy SILT interbeds from 55 to 85 feet; moist to wet; (Alluvium) (Ha) SM.	90.0
Very soft to medium stiff, gray, clayey SILT/silty CLAY with scattered silty sand seams; moist; (Estuarine Deposits) (He)	

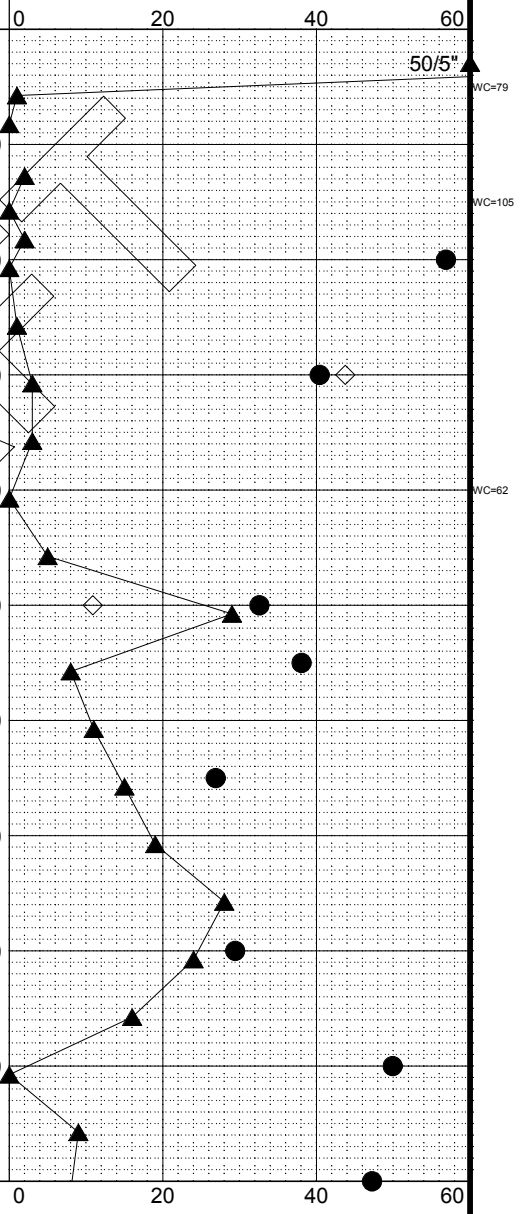
CONTINUED NEXT SHEET

Samples: 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 13, 14, 15, 16, 17, 18, 19, 20, 21, 22, 23

During Drilling

PENETRATION RESISTANCE (blows/foot)

▲ Hammer Wt. & Drop: 140 lbs / 30 inches



LEGEND

- * Sample Not Recovered
- 2.0" O.D. Split Spoon Sample
- Thin Wall Sample
- Ground Water Level ATD
- ◇ % Fines (<0.075mm)
- ● % Water Content
- Plastic Limit ├─●─┤ Liquid Limit
- Natural Water Content

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.

USD Crude-by-Rail Project
Port of Grays Harbor
Hoquiam, Washington

LOG OF BORING B-4

September 2013 | 21-1-21839-001

SHANNON & WILSON, INC.
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FIG. A-15
Sheet 1 of 2

DRAFT

Total Depth:	170 ft.	Northing:		Drilling Method:	HSA and Mud Rotary	Hole Diam.:	6 in.
Top Elevation:	~	Easting:		Drilling Company:	Gregory Drilling	Rod Diam.:	
Vert. Datum:		Station:		Drill Rig Equipment:	Track CME	Hammer Type:	Automatic
Horiz. Datum:		Offset:		Other Comments:			

SOIL DESCRIPTION

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

ML/CH/OH.

Very dense, gray, sandy GRAVEL; wet; (Recessional Outwash) (Qvro) GP.

BOTTOM OF BORING COMPLETED 4/9/2013

Depth, ft.: 152.5, 170.0

Symbol

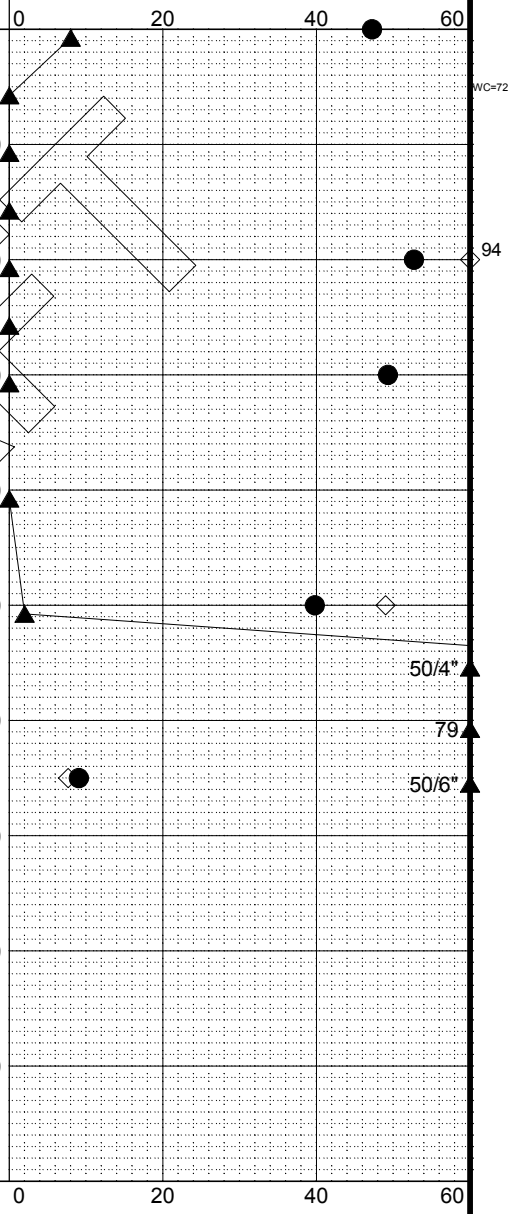
Samples: 24, 25, 26, 27, 28, 29, 30, 31, 32, 33, 34, 35

Ground Water

Depth, ft.: 110, 120, 130, 140, 150, 160, 170, 180, 190

PENETRATION RESISTANCE (blows/foot)

▲ Hammer Wt. & Drop: 140 lbs / 30 inches



DRAFT

LEGEND

* Sample Not Recovered

2.0" O.D. Split Spoon Sample

Thin Wall Sample

Ground Water Level ATD

◇ % Fines (<0.075mm)

● % Water Content

Plastic Limit ├─●─┤ Liquid Limit

Natural Water Content

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.

USD Crude-by-Rail Project
Port of Grays Harbor
Hoquiam, Washington

LOG OF BORING B-4

September 2013 | 21-1-21839-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-15
Sheet 2 of 2

Total Depth:	176 ft.	Northing:		Drilling Method:	HSA and Mud Rotary	Hole Diam.:	6 in.
Top Elevation:	~	Easting:		Drilling Company:	Gregory Drilling	Rod Diam.:	
Vert. Datum:		Station:		Drill Rig Equipment:	Track CME	Hammer Type:	Automatic
Horiz. Datum:		Offset:		Other Comments:			

SOIL DESCRIPTION

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

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

PENETRATION RESISTANCE (blows/foot)

▲ Hammer Wt. & Drop: 140 lbs / 30 inches

Medium dense to very dense, gray, sandy GRAVEL; wet; GP.

7.5

Very soft to soft, gray-black from 17.5 to 21.5 feet, slightly sandy, clayey SILT to silty CLAY, organic-rich, silty CLAY (OH) from 17.5 to 21.5 feet; moist; (Estuarine Deposits) (He) ML/CL.

31.5

Very soft to loose, gray, interbedded, clayey SILT/silty CLAY with scattered sand seams; (Estuarine Deposits) (He) ML/CL.

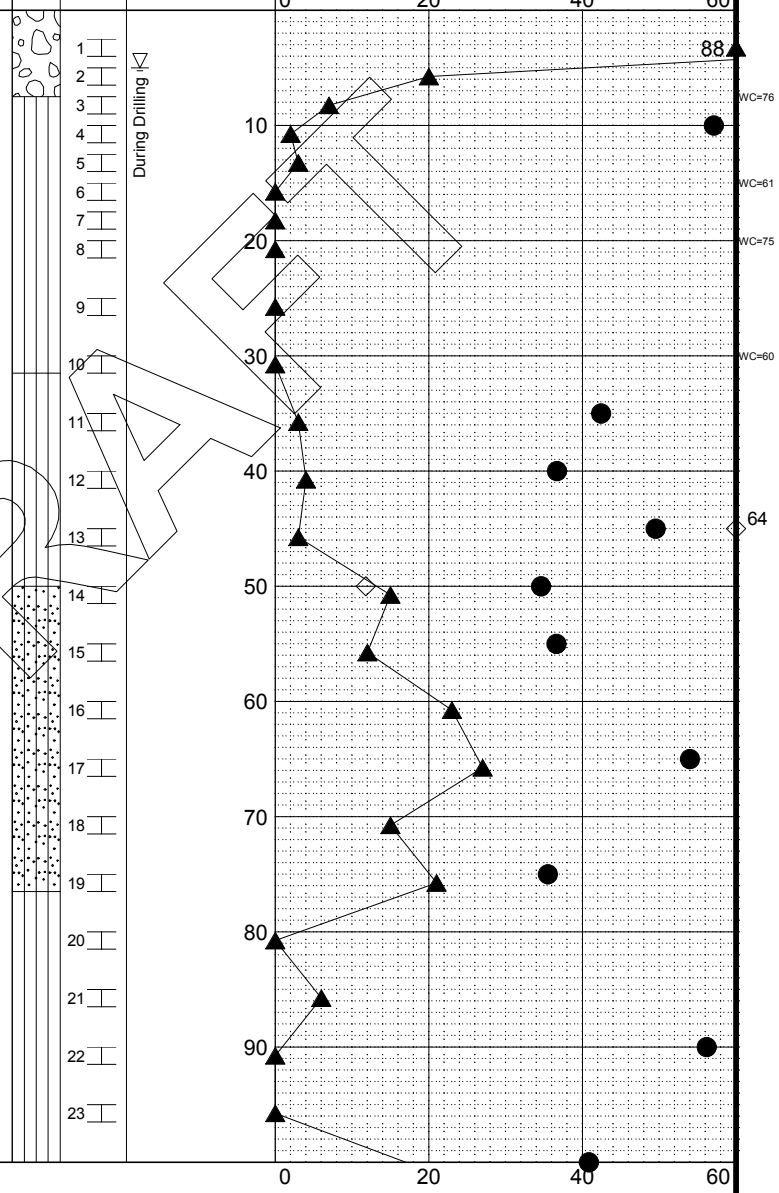
50.0

Medium dense, gray, silty SAND; moist to wet; scattered silty clay seams at 56 feet; (Alluvium) (Ha) SM.

76.5

Very soft to soft, gray, clayey SILT/silty CLAY; moist; medium dense, silty sand layer from 100 to 101.5 feet, scattered sand seams at 85 and 96.5 feet; (Estuarine Deposits) (He) ML/CL.

CONTINUED NEXT SHEET



LEGEND

* Sample Not Recovered

2.0" O.D. Split Spoon Sample

Thin Wall Sample

Ground Water Level ATD

◇ % Fines (<0.075mm)

● % Water Content

Plastic Limit ├─●─┤ Liquid Limit

Natural Water Content

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.

USD Crude-by-Rail Project
Port of Grays Harbor
Hoquiam, Washington

LOG OF BORING B-5

September 2013 | 21-1-21839-001

SHANNON & WILSON, INC.
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FIG. A-16
Sheet 1 of 2

DRAFT

MASTER_LOG_E 21-21839.GPJ SHAN_WIL.GDT 3/24/14 Log: AJD Rev: AJD Typ: LKN

Total Depth:	176 ft.	Northing:		Drilling Method:	HSA and Mud Rotary	Hole Diam.:	6 in.
Top Elevation:	~	Easting:		Drilling Company:	Gregory Drilling	Rod Diam.:	
Vert. Datum:		Station:		Drill Rig Equipment:	Track CME	Hammer Type:	Automatic
Horiz. Datum:		Offset:		Other Comments:			

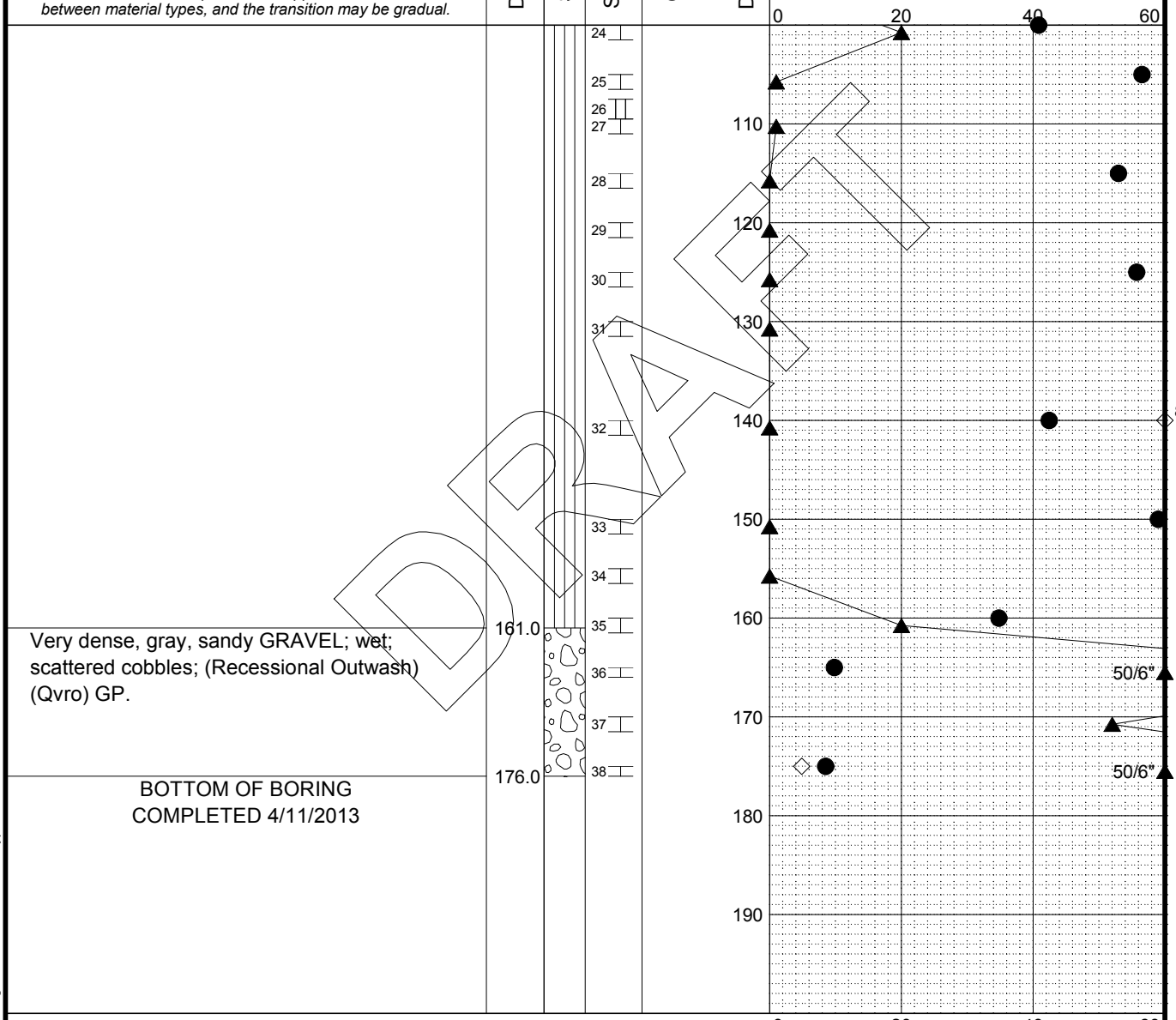
SOIL DESCRIPTION

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

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

PENETRATION RESISTANCE (blows/foot)

▲ Hammer Wt. & Drop: 140 lbs / 30 inches



Very dense, gray, sandy GRAVEL; wet; scattered cobbles; (Recessional Outwash) (Qvro) GP.

BOTTOM OF BORING
COMPLETED 4/11/2013

DRAFT

LEGEND

- * Sample Not Recovered
- 2.0" O.D. Split Spoon Sample
- Thin Wall Sample
- ▽ Ground Water Level ATD
- ◇ % Fines (<0.075mm)
- ● % Water Content
- Plastic Limit ├─●─┤ Liquid Limit
- Natural Water Content

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.

USD Crude-by-Rail Project
Port of Grays Harbor
Hoquiam, Washington

LOG OF BORING B-5

September 2013 | 21-1-21839-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A-16
Sheet 2 of 2

MASTER_LOG_E 21-21839.GPJ SHAN_WIL.GDT 3/24/14 Log: AJD Rev: AJD Typ: LKN

Total Depth:	61.5 ft.	Northing:		Drilling Method:	HSA and Mud Rotary	Hole Diam.:	6 in.
Top Elevation:	~	Easting:		Drilling Company:	Gregory Drilling	Rod Diam.:	
Vert. Datum:		Station:		Drill Rig Equipment:	Track CME	Hammer Type:	Automatic
Horiz. Datum:		Offset:		Other Comments:			

SOIL DESCRIPTION

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

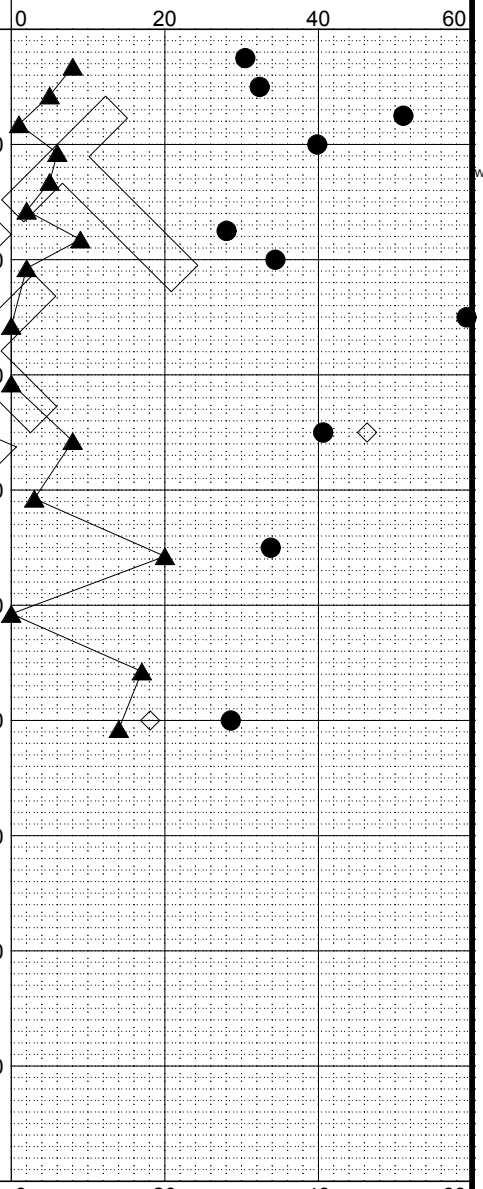
Description	Depth, ft.
Gravel roadway.	1.0
Very loose to loose, gray, silty SAND with scattered clayey silt and silty clay seams; wet; (Estuarine Deposits) (He) SM.	14.0
Very loose to loose, gray, silty SAND; wet; (Estuarine Deposits) (He) SM.	21.5
Very soft to soft, gray, interbedded, sandy, clayey SILT and silty CLAY, medium dense, silty sand layer from 45 to 46.5 feet; moist to wet (sand layer); (Estuarine Deposits) (He) ML/CL.	55.0
Medium dense, gray, silty SAND; wet; (Alluvium) (Ha) SM.	61.5
BOTTOM OF BORING 4/11/2013	

Samples: 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 13, 14, 15, 16

Ground Water: During Drilling

PENETRATION RESISTANCE (blows/foot)

▲ Hammer Wt. & Drop: 140 lbs / 30 inches



LEGEND

- * Sample Not Recovered
- 2.0" O.D. Split Spoon Sample
- Piezometer Screen and Sand Filter
- Bentonite-Cement Grout
- Bentonite Chips/Pellets
- Bentonite Grout
- Ground Water Level ATD

◇ % Fines (<0.075mm)

● % Water Content

Plastic Limit ├─●─┤ Liquid Limit

Natural Water Content

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.

USD Crude-by-Rail Project
Port of Grays Harbor
Hoquiam, Washington

LOG OF BORING B-6

September 2013 | 21-1-21839-001

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FIG. A-17

DRAFT

Total Depth:	*61.5 ft.*	Northing:		Drilling Method:	*HSA and Mud Rotary*	Hole Diam.:	*6 in.*
Top Elevation:	*~*	Easting:		Drilling Company:	*Gregory Drilling*	Rod Diam.:	
Vert. Datum:		Station:		Drill Rig Equipment:	*Track CME*	Hammer Type:	*Automatic*
Horiz. Datum:		Offset:		Other Comments:			

SOIL DESCRIPTION

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

Soil Description	Depth, ft.
Gravel roadway.	1.0
Loose to very loose, greenish-gray, silty SAND with silty clay seam from 5 to 5.3 feet; moist to wet; (Estuarine Deposits) (He) SM.	6.5
Very soft, gray, interbedded, clayey SILT and silty CLAY (scattered silty sand seams); wet; (Estuarine Deposits) (He) ML/CL.	14.0
Very soft, gray to black (from 15 to 21.5 feet), clayey SILT to silty CLAY, organic-rich silty clay from 15 to 21.5 feet; moist; (Estuarine Deposits) (He) ML/CL.	31.5
Very soft to medium dense, gray, interbedded, silty SAND and slightly sandy, clayey SILT with scattered silty clay sams; (Estuarine Deposits) (He) SM/ML.	55.0
Medium dense, gray, silty SAND; wet; (Alluvium) (Ha) SM.	61.5
BOTTOM OF BORING COMPLETED 4/11/2013	

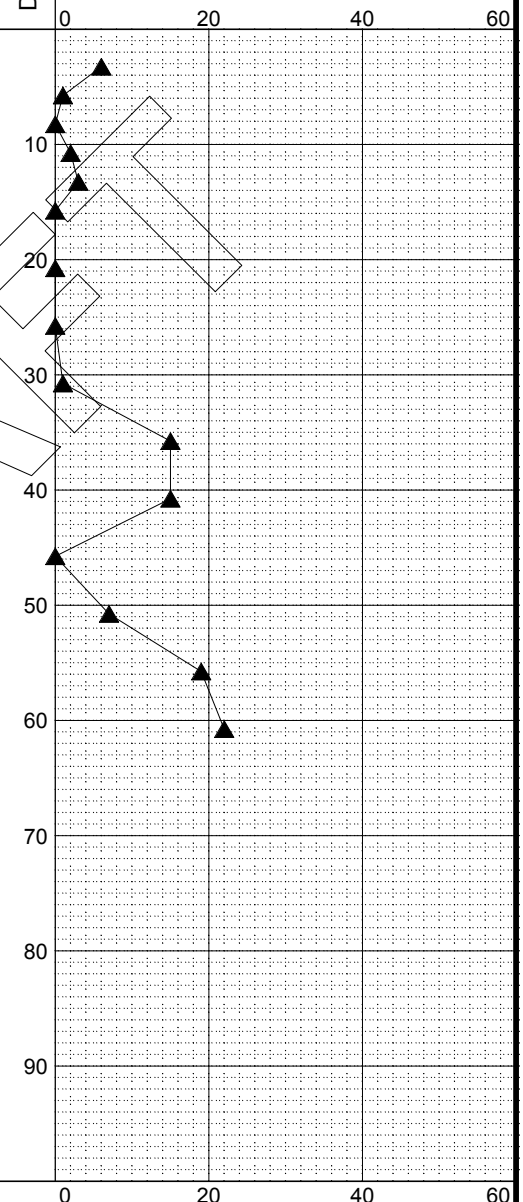
Symbol | Samples | Ground Water | Depth, ft.

Samples: 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 13, 14, 15

During Drilling

PENETRATION RESISTANCE (blows/foot)

▲ Hammer Wt. & Drop: *140 lbs / 30 inches*



◇ % Fines (<0.075mm)

● % Water Content

LEGEND

* Sample Not Recovered

2.0" O.D. Split Spoon Sample

Ground Water Level ATD

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.

USD Crude-by-Rail Project
Port of Grays Harbor
Hoquiam, Washington

LOG OF BORING B-7

September 2013 21-1-21839-001

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FIG. A-18

DRAFT

MASTER_LOG_E 21-21839.GPJ SHAN_WIL.GDT 3/24/14 Log: AJD Rev: AJD Typ: LKN

APPENDIX B

GEOTECHNICAL LABORATORY TEST RESULTS

APPENDIX B

GEOTECHNICAL LABORATORY TEST RESULTS

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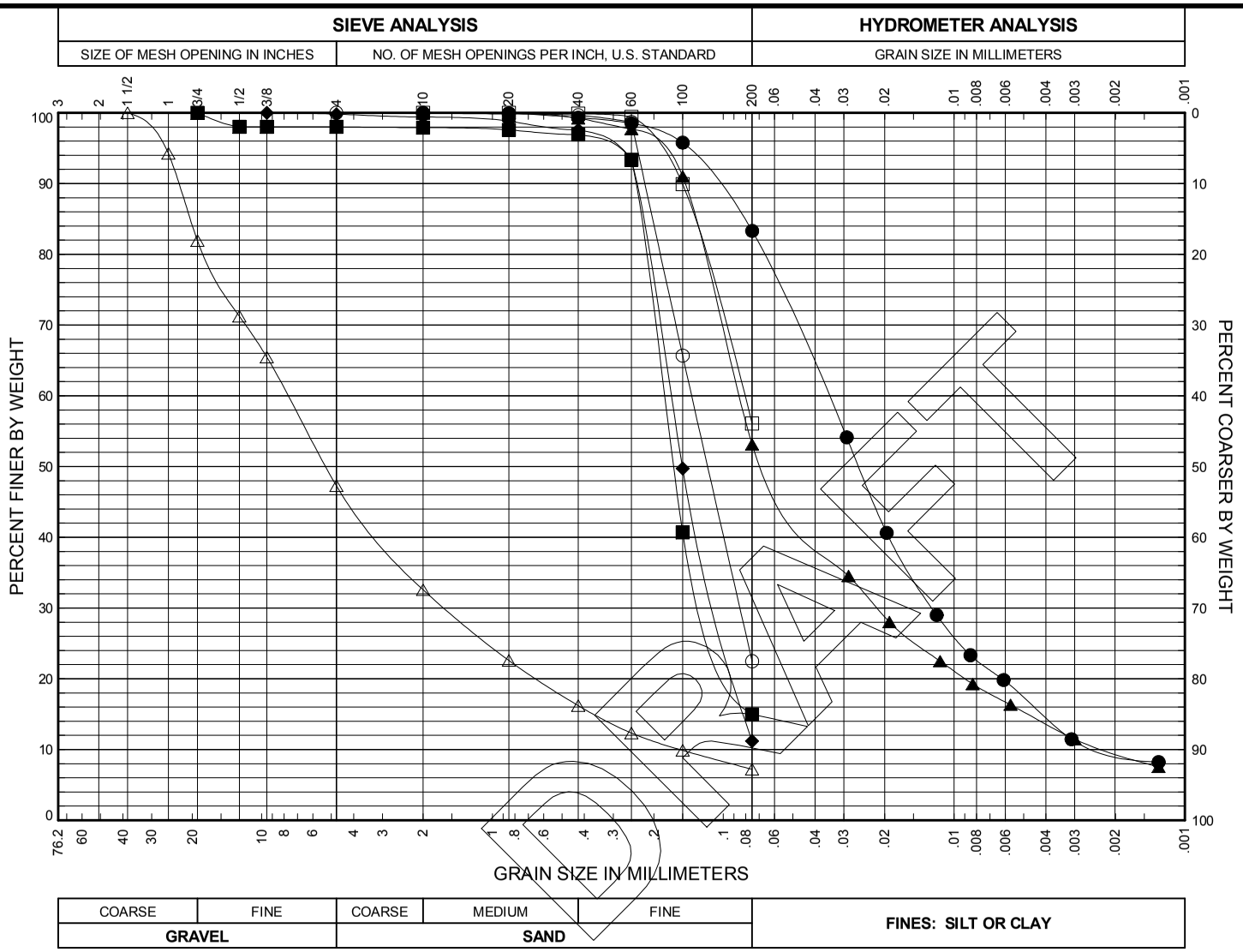
B-12	Plasticity Chart Boring B-6
B-13	One Dimensional Consolidation Test Summary, Boring B-2, Sample S-9 @ 23.3 ft
B-14	One Dimensional Consolidation Void Ratio vs Stress Plot, Boring B-2, Sample S-9 @ 23.3 ft
B-15	One Dimensional Consolidation Percent Settlement vs Stress Plot, Boring B-2, Sample S-9 @ 23.3 ft
B-16	One Dimensional Consolidation Test Summary, Boring B-2, Sample S-9 @ 23.3 ft
B-17	One Dimensional Consolidation Void Ratio vs Stress Plot, Boring B-3, Sample S-4 @ 11.8 ft
B-18	One Dimensional Consolidation Percent Settlement vs Stress Plot, Boring B-3, Sample S-4 @ 11.8 ft
B-19	One Dimensional Consolidation Test Summary, Boring B-5, Sample S-26 @ 108.2 ft
B-20	One Dimensional Consolidation Void Ratio vs Stress Plot, Boring B-5, Sample S-26 @ 108.2 ft
B-21	One Dimensional Consolidation Percent Settlement vs Stress Plot, Boring B-5, Sample S-26 @ 108.2 ft



LEGEND

USCS: Unified Soil Classification System

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SG: Specific gravity of soil solids < No. 4 sieve per ASTM D854

NAT WC %: Natural water content

Cu: Coefficient of uniformity

Cc: Coefficient of curvature

ASTM DES: ASTM International test standard designation

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

BORING AND SAMPLE NO.	DEPTH (feet)	U.S.C.S. SYMBOL	SOIL CLASSIFICATION	GRAVEL %	SAND %	FINES %	COBBLE REM %	SG	NAT WC %	Cu	Cc	TEST BY	REVIEW BY	ASTM DES
● B-1, S-13	45.0	ML	Gray, clayey, fine sandy SILT		17	83.3			53.1			AKV	JFL	D422
■ B-1, S-15*	55.0	SM	Gray, silty, fine SAND, trace of fine gravel; trace of organics, trace of shell fragments	2	83	14.9			29.1			AKV	JFL	D422
▲ B-1, S-16	60.0	ML	Gray, fine sandy SILT, trace of clay		47	53.1			53.3			AKV	JFL	D422
◆ B-1, S-19	75.0	SP-SM	Gray, slightly silty, fine SAND; trace of organics, trace of shell fragments	0	89	11.2			25.8	2.3	0.9	AKV	JFL	D422
○ B-1, S-25	105.0	SM	Gray, silty, fine SAND; trace of organics, trace of shell fragments		78	22.5			35.4			AKV	JFL	D422
□ B-1, S-35	155.0	ML	Gray, fine sandy SILT; trace of organics		44	56.1			44.5			AKV	JFL	D422
△ B-1, S-39*	175.0	GW-GM	Gray, slightly silty, sandy GRAVEL	53	40	7.2			9.0	50.1	2.2	AKV	JFL	D422

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GRAIN SIZE DISTRIBUTION
BORING B-1

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FIG. B-1
Sheet 1 of 1

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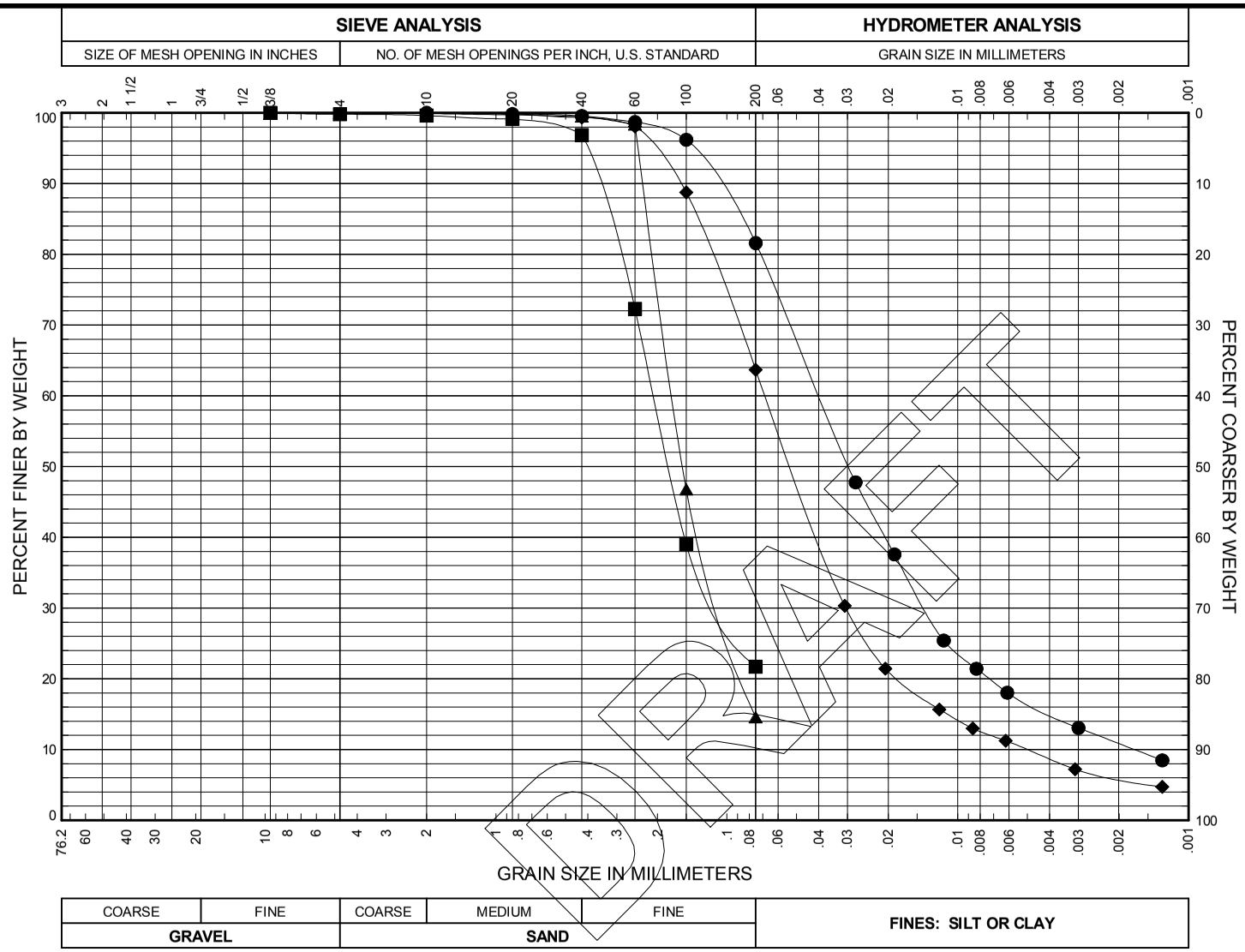

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● B-2, S-12	35.0	ML	Gray, fine sandy SILT, trace of clay; scattered organics		18	81.6			53.1			AKV	JFL	D422
■ B-2, S-13	40.0	SM	Gray, silty, fine SAND; scattered organics, scattered shell fragments	0	78	21.7			32.3			AKV	JFL	D422
▲ B-2, S-20	75.0	SM	Gray, silty, fine SAND; trace of fine organics		85	14.6			30.4			AKV	JFL	D422
◆ B-2, S-25	100.0	ML	Gray, fine sandy SILT; trace of organics, trace of shell fragments		36	63.7			40.4			AKV	JFL	D422

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GRAIN SIZE DISTRIBUTION BORING B-2

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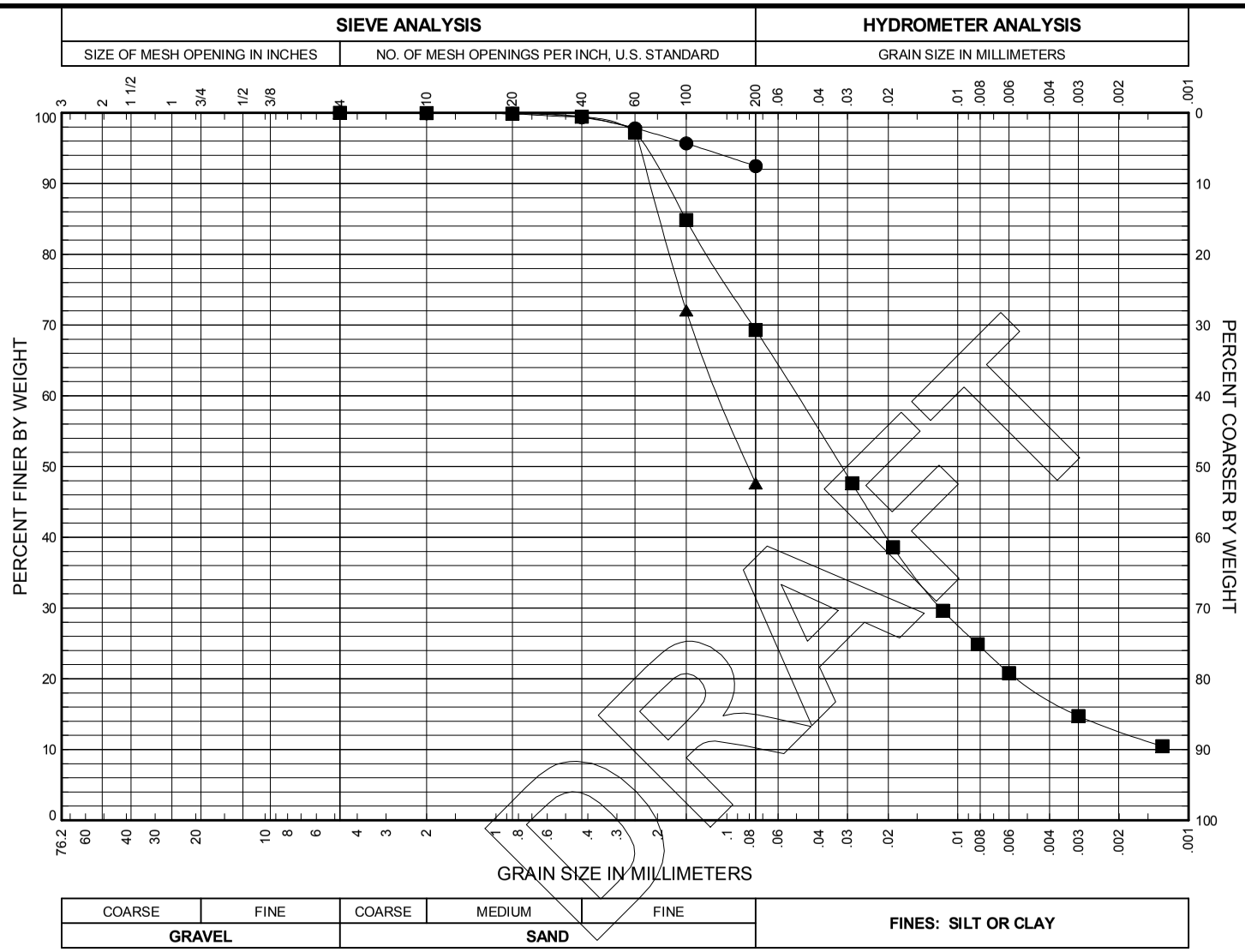
FIG. B-2
Sheet 1 of 1

FIG. B-2

AASHTO_GSA_MAIN_3INCHMINUS_21-21839.GPJ 22-02944.GPJ 9/10/13



LEGEND

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ASTM DES: ASTM International test standard designation

BORING AND SAMPLE NO.	DEPTH (feet)	U.S.C.S. SYMBOL	SOIL CLASSIFICATION	GRAVEL %	SAND %	FINES %	COBBLE REM %	SG	NAT WC %	Cu	Cc	TEST BY	REVIEW BY	ASTM DES
● B-3, S-6	15.0	OH	Dark gray-brown, slightly fine sandy, organic SILT		8	92.5			106.0			AKV	JFL	D422
■ B-3, S-13	45.0	ML	Gray, fine sandy SILT		31	69.3			54.2			AKV	JFL	D422
▲ B-3, S-15	55.0	SM	Gray, silty, fine SAND; trace of organics, trace of shell fragments		52	47.6			40.4			AKV	JFL	D422

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GRAIN SIZE DISTRIBUTION
BORING B-3

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FIG. B-3
Sheet 1 of 1

FIG. B-3

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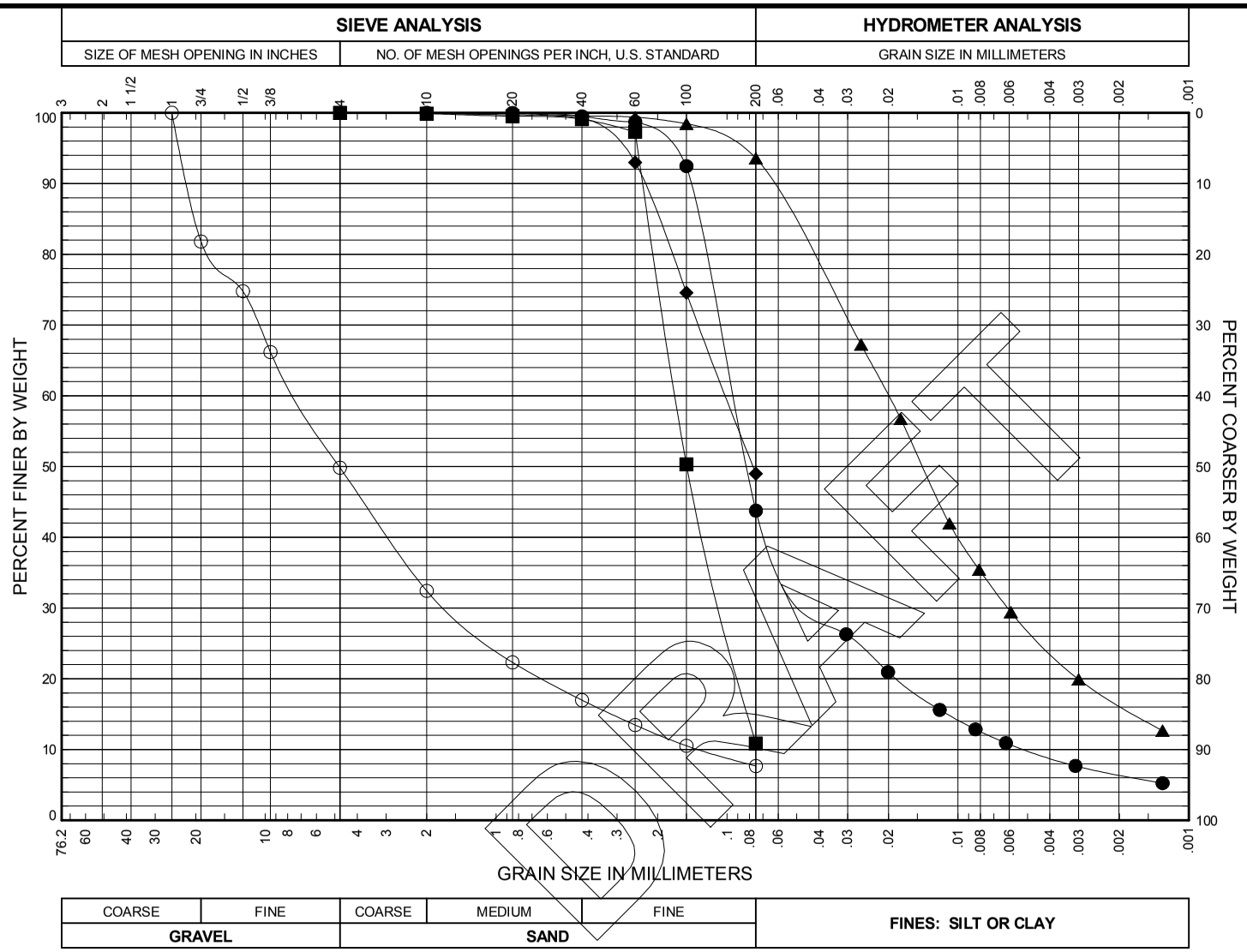

LEGEND

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Cc: Coefficient of curvature

ASTM DES: ASTM International test standard designation

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

BORING AND SAMPLE NO.	DEPTH (feet)	U.S.C.S. SYMBOL	SOIL CLASSIFICATION	GRAVEL %	SAND %	FINES %	COBBLE REM %	SG	NAT WC %	Cu	Cc	TEST BY	REVIEW BY	ASTM DES
● B-4, S-10	30.0	SM	Gray, silty, fine SAND; scattered organics		56	43.8			40.4			AKV	JFL	D422
■ B-4, S-14	50.0	SP-SM	Gray, slightly silty, fine SAND; scattered organics		89	10.9			32.6	2.3	0.9	AKV	JFL	D422
▲ B-4, S-28	120.0	ML	Gray, slightly fine sandy, clayey SILT; scattered organics		6	93.6			52.7			AKV	JFL	D422
◆ B-4, S-32	150.0	SM	Gray, silty, fine SAND; scattered organics		51	49.0			39.8			AKV	JFL	D422
○ B-4, S-35*	165.0	GW-GM	Gray, slightly silty, sandy GRAVEL	50	42	7.7			9.1	55.4	2.7	AKV	JFL	D422

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GRAIN SIZE DISTRIBUTION
BORING B-4

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FIG. B-4
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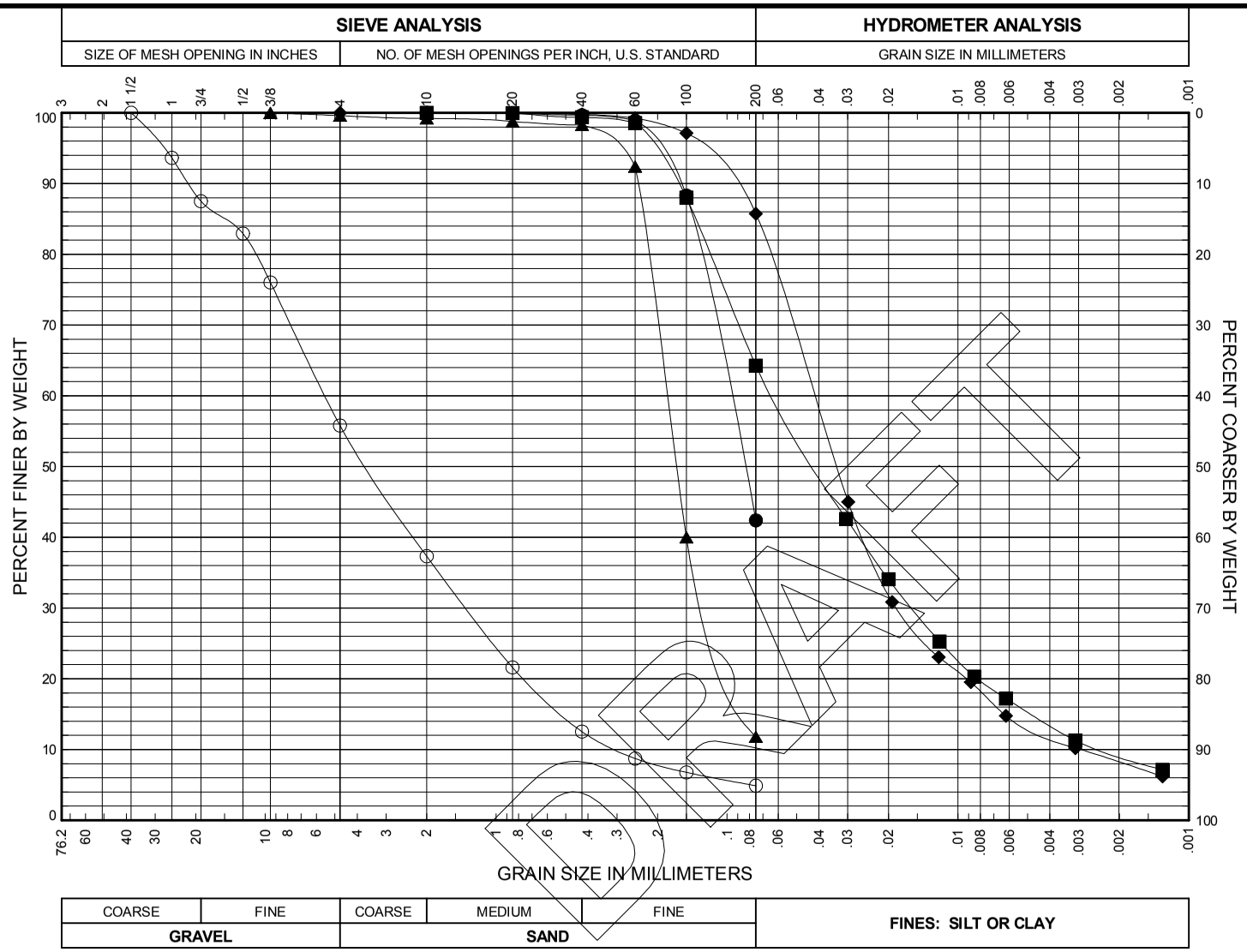

LEGEND

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BORING AND SAMPLE NO.	DEPTH (feet)	U.S.C.S. SYMBOL	SOIL CLASSIFICATION	GRAVEL %	SAND %	FINES %	COBBLE REM %	SG	NAT WC %	Cu	Cc	TEST BY	REVIEW BY	ASTM DES
● B-5, S-11	35.0	SM	Gray, silty, fine SAND; trace of organics		58	42.4			42.4			AKV	JFL	D422
■ B-5, S-13	45.0	ML	Gray, fine sandy SILT		36	64.3			49.5			AKV	JFL	D422
▲ B-5, S-14	50.0	SP-SM	Gray, slightly silty, fine SAND; trace of organics	0	88	11.8			34.6	2.5	1.1	AKV	JFL	D422
◆ B-5, S-32	140.0	ML	Gray, clayey, fine sandy SILT		14	85.8			42.4			AKV	JFL	D422
○ B-5, S-38*	175.0	SW	Gray, gravelly SAND, trace of silt	44	51	4.9			8.5	18.4	1.1	AKV	JFL	D422

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GRAIN SIZE DISTRIBUTION
BORING B-5

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FIG. B-5
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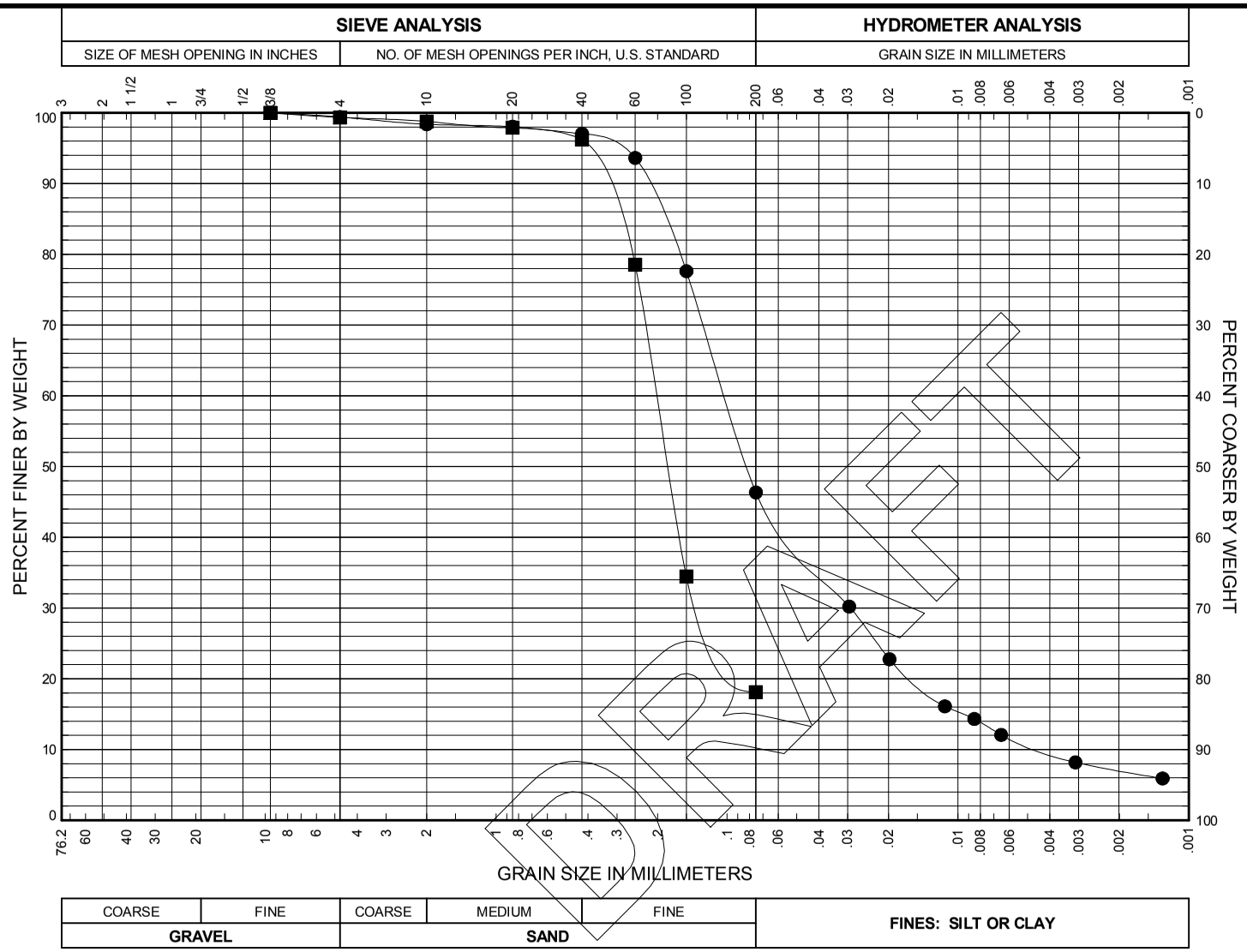
FIG. B-5



LEGEND

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ASTM DES: ASTM International test standard designation

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

BORING AND SAMPLE NO.	DEPTH (feet)	U.S.C.S. SYMBOL	SOIL CLASSIFICATION	GRAVEL %	SAND %	FINES %	COBBLE REM %	SG	NAT WC %	Cu	Cc	TEST BY	REVIEW BY	ASTM DES
● B-6, S-11*	35.0	SM	Gray, silty, fine SAND, trace of fine gravel; trace of organics	1	53	46.3			40.6			AKV	JFL	D422
■ B-6, S-16	60.0	SM	Gray, silty, fine SAND, trace of fine gravel; trace of organics	1	81	18.1			28.6			AKV	JFL	D422

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GRAIN SIZE DISTRIBUTION
BORING B-6

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FIG. B-6
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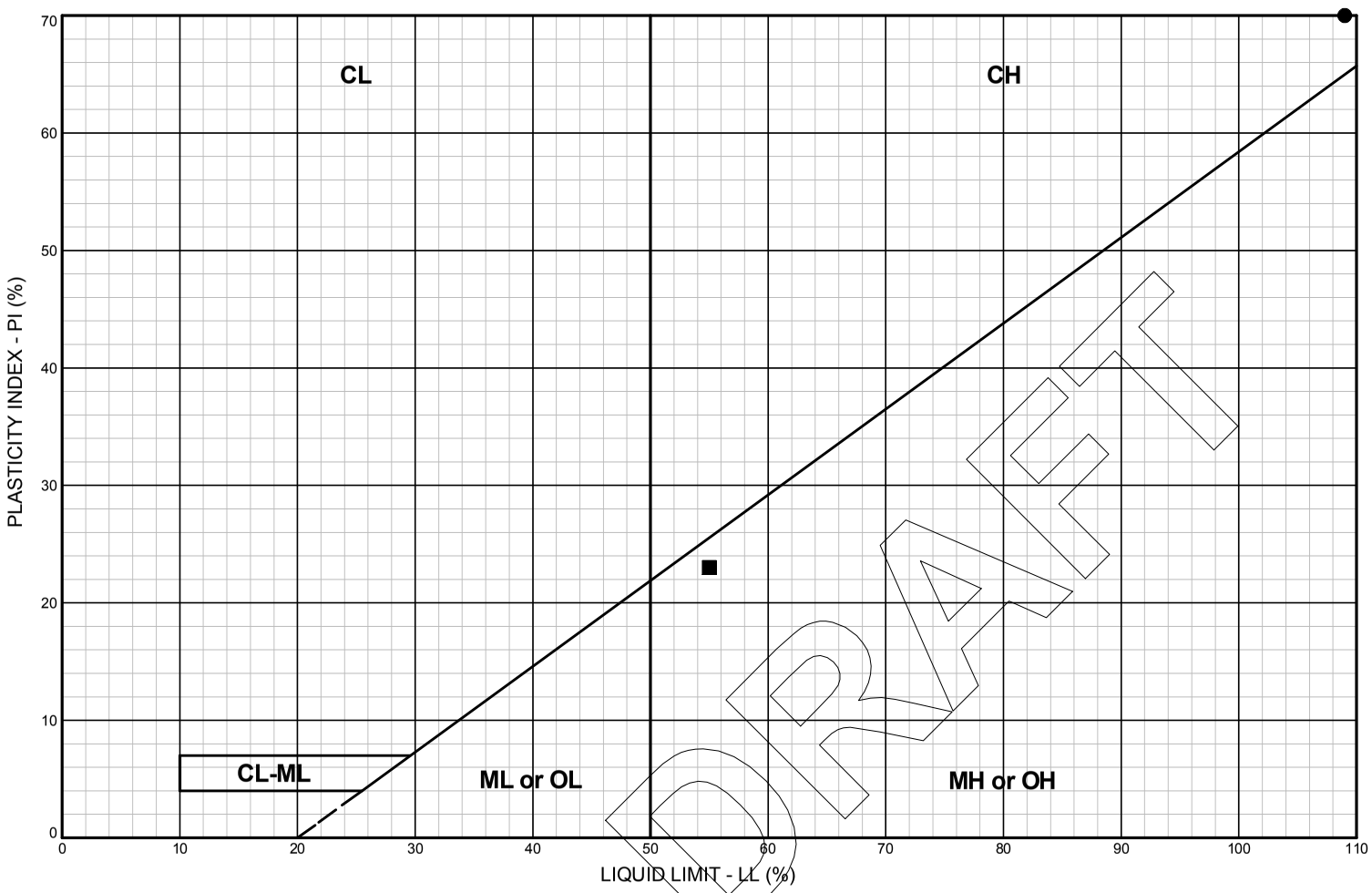
FIG. B-6



DRAFT

LEGEND

CL: Low plasticity inorganic clays; sandy and silty clays

CH: High plasticity inorganic clays

ML: Inorganic silts and clayey silts of low plasticity

MH: Inorganic silts and clayey silts of high plasticity

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OH: Organic silts and clays of high plasticity

LL: Liquid limit

PL: Plastic limit

PI: Plasticity index; PI=LL-PL

NP: Nonplastic

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

NAT WC%: Natural water content

^,>>: Test value exceeds limit of graph

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BORING AND SAMPLE NO.	DEPTH (feet)	U.S.C.S. SYMBOL	SOIL CLASSIFICATION	LL %	PL %	PI %	NAT WC %	FINES %	TEST BY	REVIEW BY	ASTM DES
● B-1, S-6	15.0	CH/OH	Dark gray-brown, silty CLAY/organic CLAY	109	39	70	94.5		AKV	JFL	D4318
■ B-1, S-29	125.0	MH	Gray, clayey SILT; trace to scattered fine organics	55	32	23	53.9		AKV	JFL	D4318

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PLASTICITY CHART
BORING B-1

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FIG. B-7
Sheet 1 of 1

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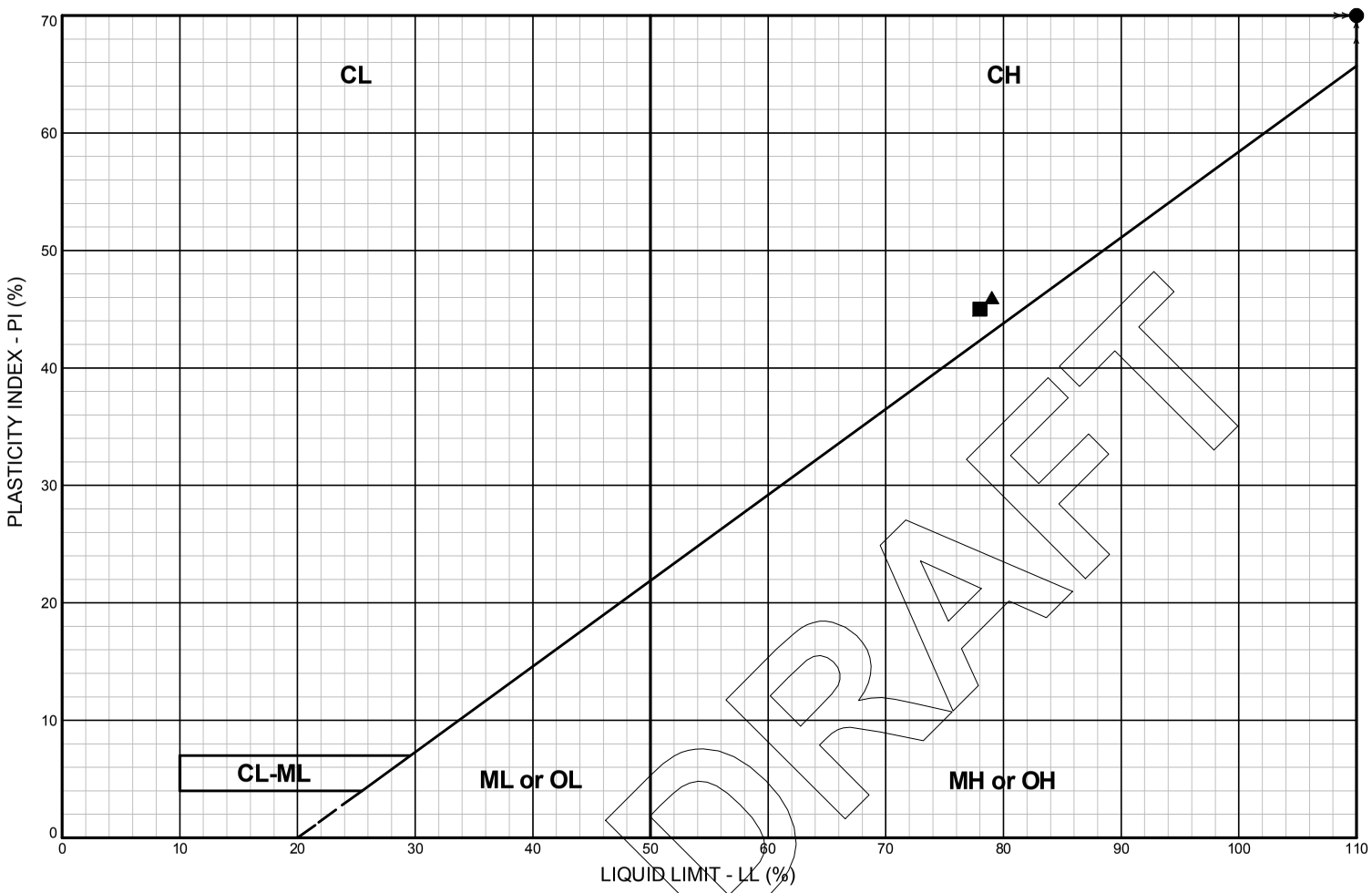
FIG. B-7



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ASTM DES: ASTM International test standard designation

BORING AND SAMPLE NO.	DEPTH (feet)	U.S.C.S. SYMBOL	SOIL CLASSIFICATION	LL %	PL %	PI %	NAT WC %	FINES %	TEST BY	REVIEW BY	ASTM DES
● B-2, S-8	20.0	CH/OH	Dark gray-brown, silty CLAY/organic CLAY	113	41	72	100.8		AKV	JFL	D4318
■ B-2, S-9	23.3	CH/OH	Dark gray-brown, silty CLAY/organic CLAY	78	33	45	70.6		AKV	JFL	D4318
▲ B-2, S-31	130.0	CH/OH	Gray, silty CLAY/organic CLAY	79	33	46	58.6		AKV	JFL	D4318

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PLASTICITY CHART
BORING B-2

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FIG. B-8
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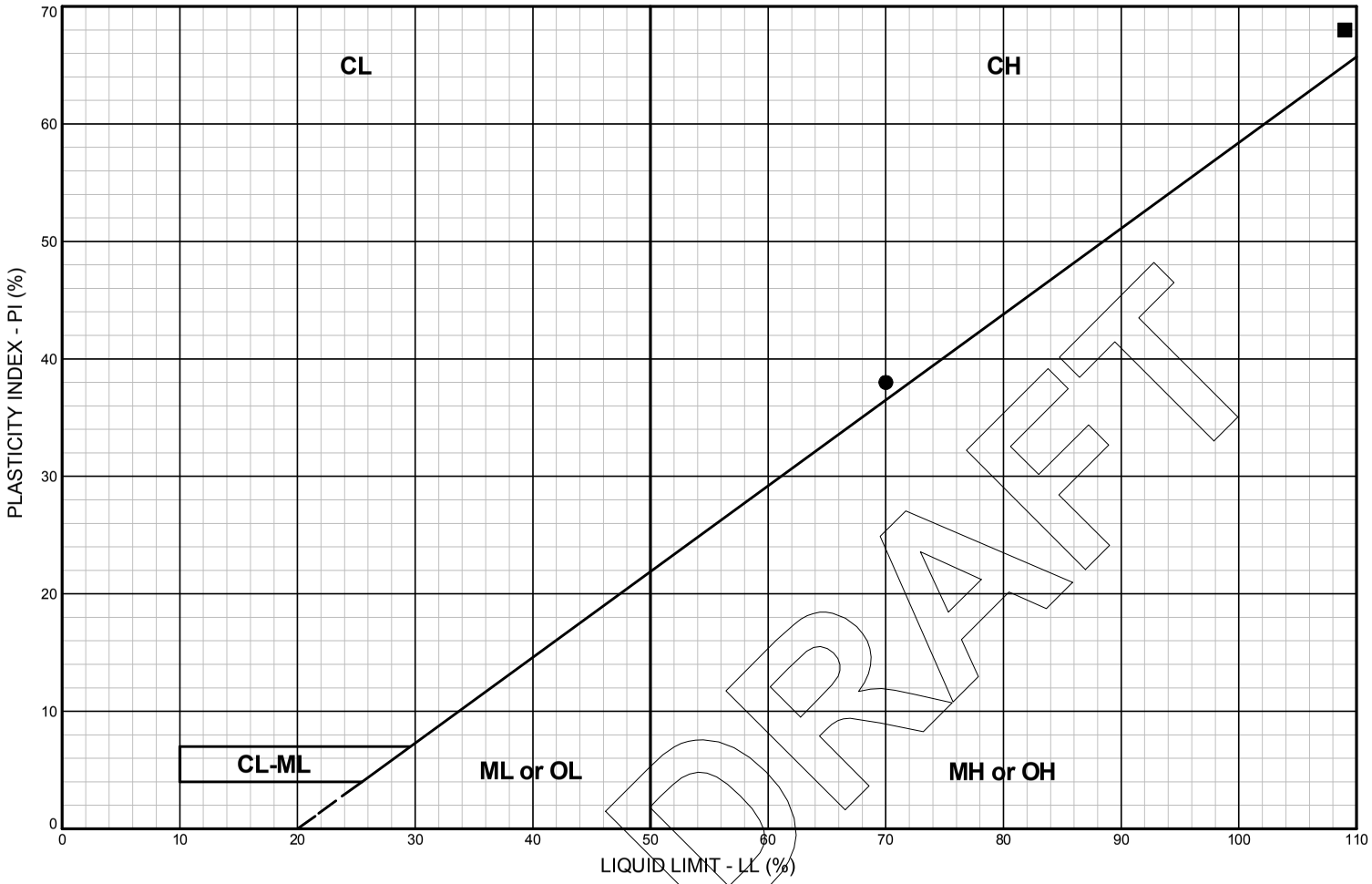
FIG. B-8



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BORING AND SAMPLE NO.	DEPTH (feet)	U.S.C.S. SYMBOL	SOIL CLASSIFICATION	LL %	PL %	PI %	NAT WC %	FINES %	TEST BY	REVIEW BY	ASTM DES
● B-3, S-3	7.5	CH/OH	Gray, silty CLAY/organic CLAY	70	32	38	84.5		AKV	JFL	D4318
■ B-3, S-4	11.8	CH/OH	Gray-brown, silty CLAY/organic CLAY	109	41	68			AKV	JFL	D4318

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PLASTICITY CHART
BORING B-3

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FIG. B-9
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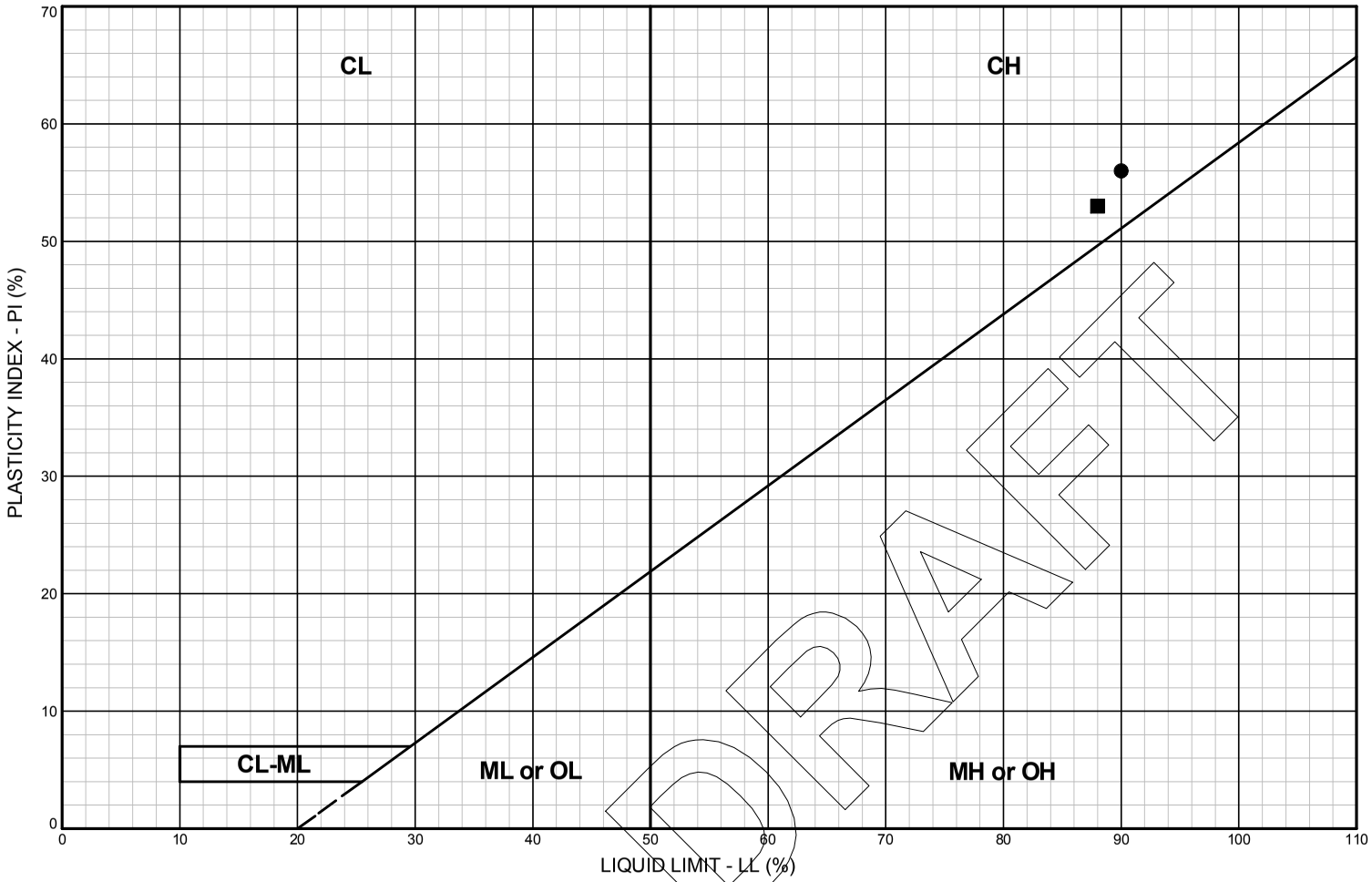
FIG. B-9



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BORING AND SAMPLE NO.	DEPTH (feet)	U.S.C.S. SYMBOL	SOIL CLASSIFICATION	LL %	PL %	PI %	NAT WC %	FINES %	TEST BY	REVIEW BY	ASTM DES
● B-4, S-6	15.0	CH/OH	Dark gray-brown, silty CLAY/organic CLAY	90	34	56	105.2		AKV	JFL	D4318
■ B-4, S-25	105.0	CH/OH	Gray, silty CLAY/organic CLAY	88	35	53	72.4		AKV	JFL	D4318

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PLASTICITY CHART
BORING B-4

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FIG. B-10
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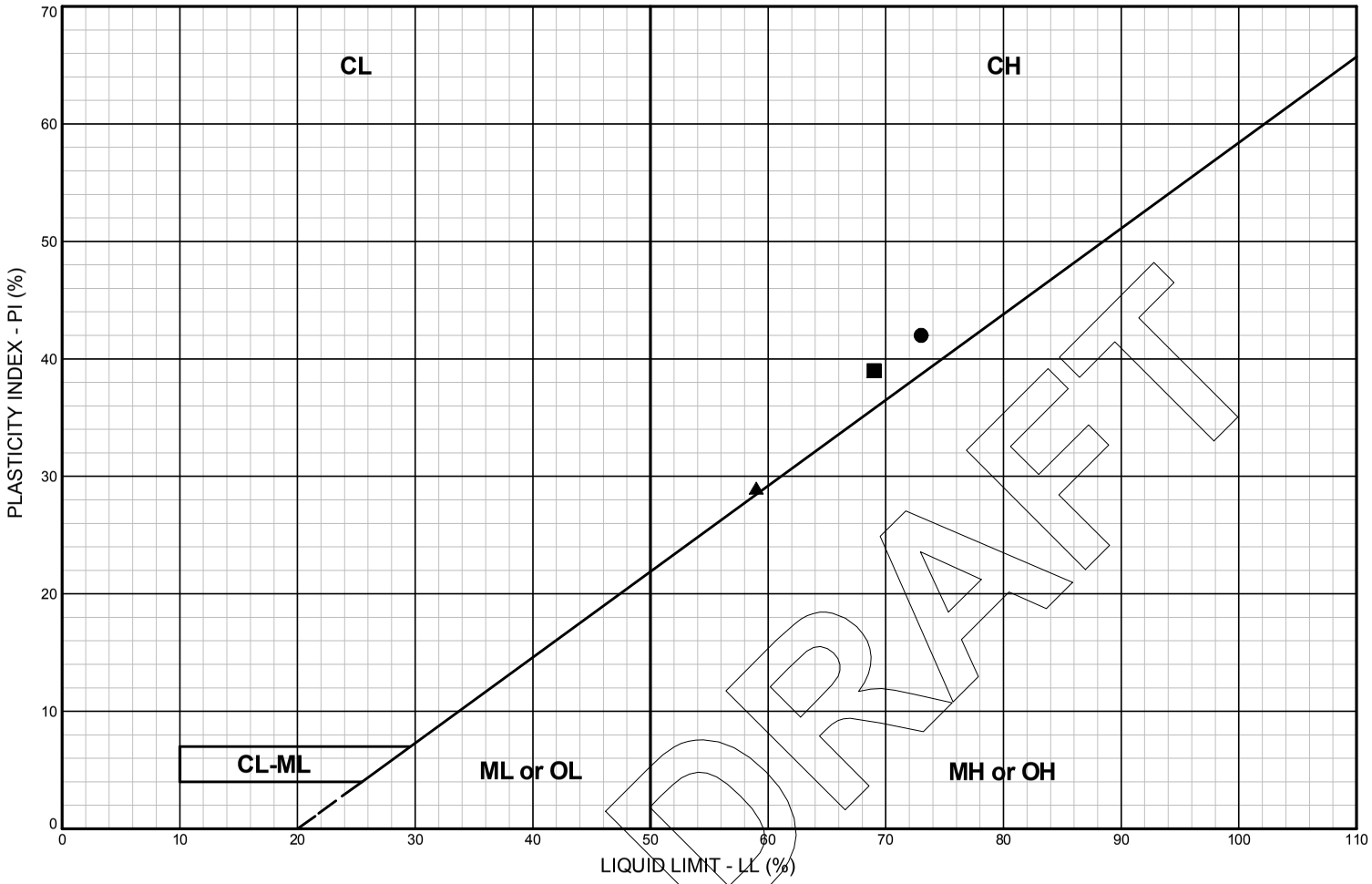

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^,>>: Test value exceeds limit of graph

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BORING AND SAMPLE NO.	DEPTH (feet)	U.S.C.S. SYMBOL	SOIL CLASSIFICATION	LL %	PL %	PI %	NAT WC %	FINES %	TEST BY	REVIEW BY	ASTM DES
● B-5, S-8	20.0	CH/OH	Dark gray-brown, silty CLAY/organic CLAY	73	31	42	74.9		AKV	JFL	D4318
■ B-5, S-25	105.0	CH/OH	Gray, silty CLAY/organic CLAY	69	30	39	56.5		AKV	JFL	D4318
▲ B-5, S-26	108.2	CH/OH	Gray-brown, silty CLAY/organic CLAY	59	30	29			AKV	JFL	D4318

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PLASTICITY CHART
BORING B-5

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FIG. B-11
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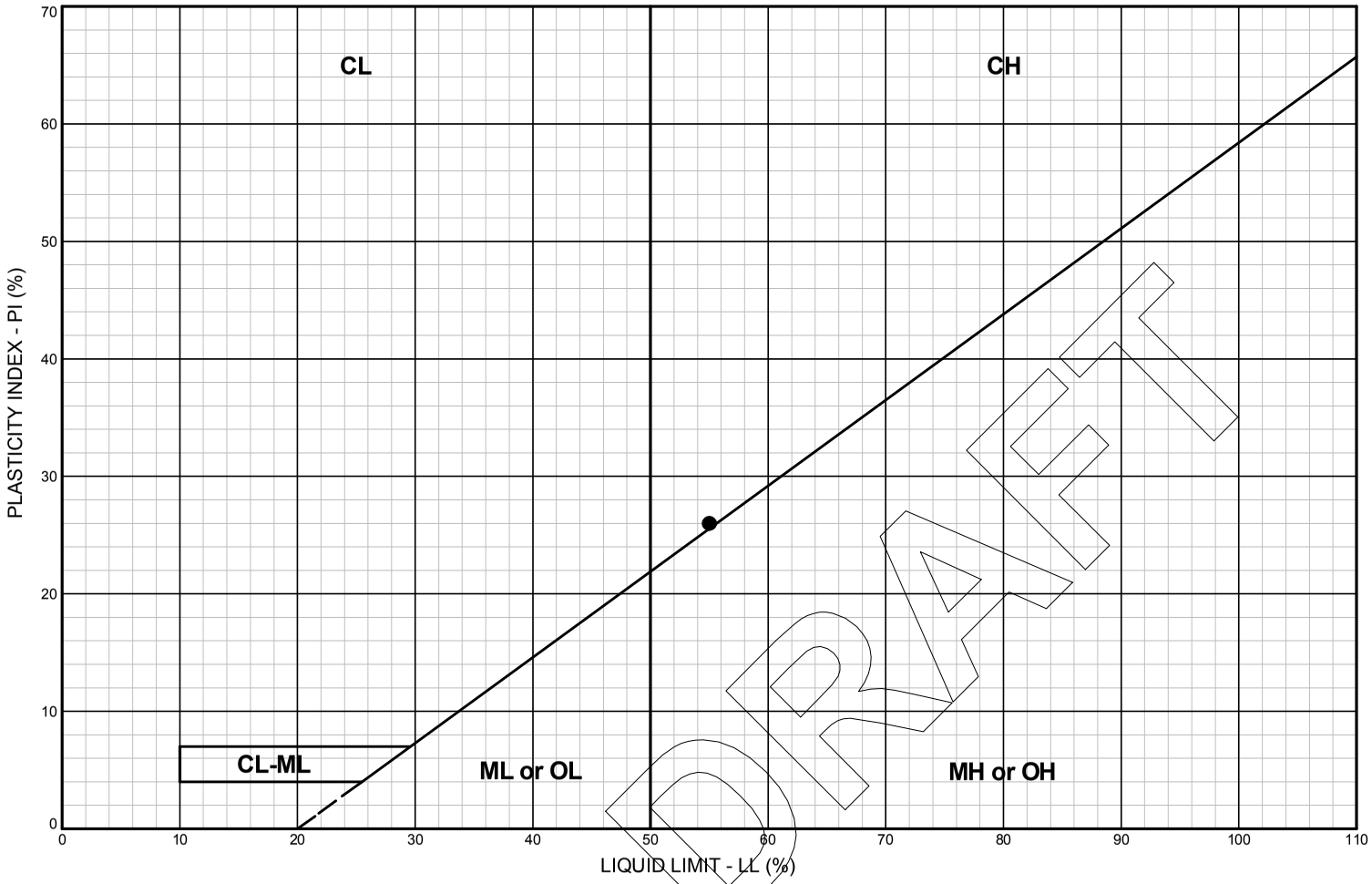

DRAFT

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BORING AND SAMPLE NO.	DEPTH (feet)	U.S.C.S. SYMBOL	SOIL CLASSIFICATION	LL %	PL %	PI %	NAT WC %	FINES %	TEST BY	REVIEW BY	ASTM DES
● B-6, S-9	25.0	CH/OH	Gray, silty CLAY/organic CLAY	55	29	26	59.3		AKV	JFL	D4318

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PLASTICITY CHART
BORING B-6

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FIG. B-12
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ONE DIMENSIONAL CONSOLIDATION TEST

Boring	B-2	Tested By	AKV
Sample	S-9	Calculated By	AKV
Depth, ft	23.3	Checked By	JFL

SAMPLE CLASSIFICATION:
Dark gray brown to black, organic SILT; specific gravity estimated - suggest organic content; OH

SAMPLE DATA:

Specific Gravity (estimated)	2.4
Liquid Limit	78
Plastic Limit	33
Plasticity Index	45

SPECIMEN DATA:	Before Inundation	First Load	Final Load
Height, inches	0.784	0.784	0.525
Diameter, inches	2.501	2.501	2.501
Sample Volume, cuin	3.853	3.853	2.580
Wet Density, pcf	98.9	98.9	120.1
Dry Density, pcf	58.9	58.9	88.0
Water Content, %	68%	68%	37%
Void Ratio	1.54	1.54	0.70
Saturation, %	100%	100%	100%

Increment	Applied Stress, tsf	ΔH at t_{100}, in	ΔH / H_o	Void Ratio	t_{50}, min	Coeff. of Comp., MPa^{-1}	Coeff. of Consol., cm^2/sec	Coeff. of Perm., cm/sec
1	0.13	0.001	0.1%	1.541	0.2	0.19	1.9E-02	1.4E-07
2	0.26	0.006	0.8%	1.523	0.8	1.51	3.9E-03	2.3E-07
3	0.52	0.015	2.0%	1.494	0.4	1.16	8.3E-03	3.7E-07
4	1.03	0.026	3.4%	1.458	0.3	0.72	9.8E-03	2.8E-07
5	2.06	0.074	9.4%	1.304	2.3	1.56	1.2E-03	7.6E-08
6	4.13	0.139	17.7%	1.093	2.1	1.07	1.2E-03	5.2E-08
7	1.03	0.144	18.3%	1.077	0.4	-0.05	5.6E-03	1.4E-08
8	0.26	0.128	16.3%	1.129	3.0	0.70	7.5E-04	2.5E-08
9	0.06	0.114	14.5%	1.175	24.7	2.47	9.5E-05	1.1E-08
10	0.26	0.115	14.6%	1.171	1.2	0.19	2.0E-03	1.8E-08
11	1.03	0.128	16.3%	1.130	1.0	0.57	2.4E-03	6.2E-08
12	4.13	0.152	19.3%	1.052	0.6	0.26	3.8E-03	4.6E-08
13	8.25	0.200	25.5%	0.894	2.2	0.40	8.9E-04	1.7E-08
14	16.51	0.249	31.7%	0.737	2.3	0.20	7.1E-04	7.3E-09
15	33.02	0.295	37.6%	0.588	2.7	0.09	5.0E-04	2.7E-09
16	8.25	0.300	38.2%	0.572	1.0	-0.01	1.2E-03	5.0E-10
17	2.06	0.2897	36.93%	0.604	6.7	0.054	1.9E-04	6.4E-10
18	0.52	0.2759	35.17%	0.649	26.1	0.302	5.1E-05	9.5E-10
19	0.13	0.2557	32.60%	0.714	196.4	1.764	7.3E-06	7.6E-10

DRAFT

NOTES:
1. Abbreviations:
   cm = centimeter
   cm^2 = square centimeter
   Coeff. = Coefficient
   Comp. = Compressibility
   Consol. = Consolidation
   cu in = cubic inch
   ft = feet
   H_o = initial height
   ΔH = change in height
   in = inch
   min = minute
   MPa = megapascal
   pcf = pounds per cubic foot
   Perm. = Permeability
   sec = second
   t_n = time at n% of primary consolidation
   tsf = tons per square foot

Port of Grays Harbor Terminal
Hoquiam, Washington

ONE DIMENSIONAL CONSOLIDATION TEST SUMMARY
BORING B-2, SAMPLE S-9 @23.3ft

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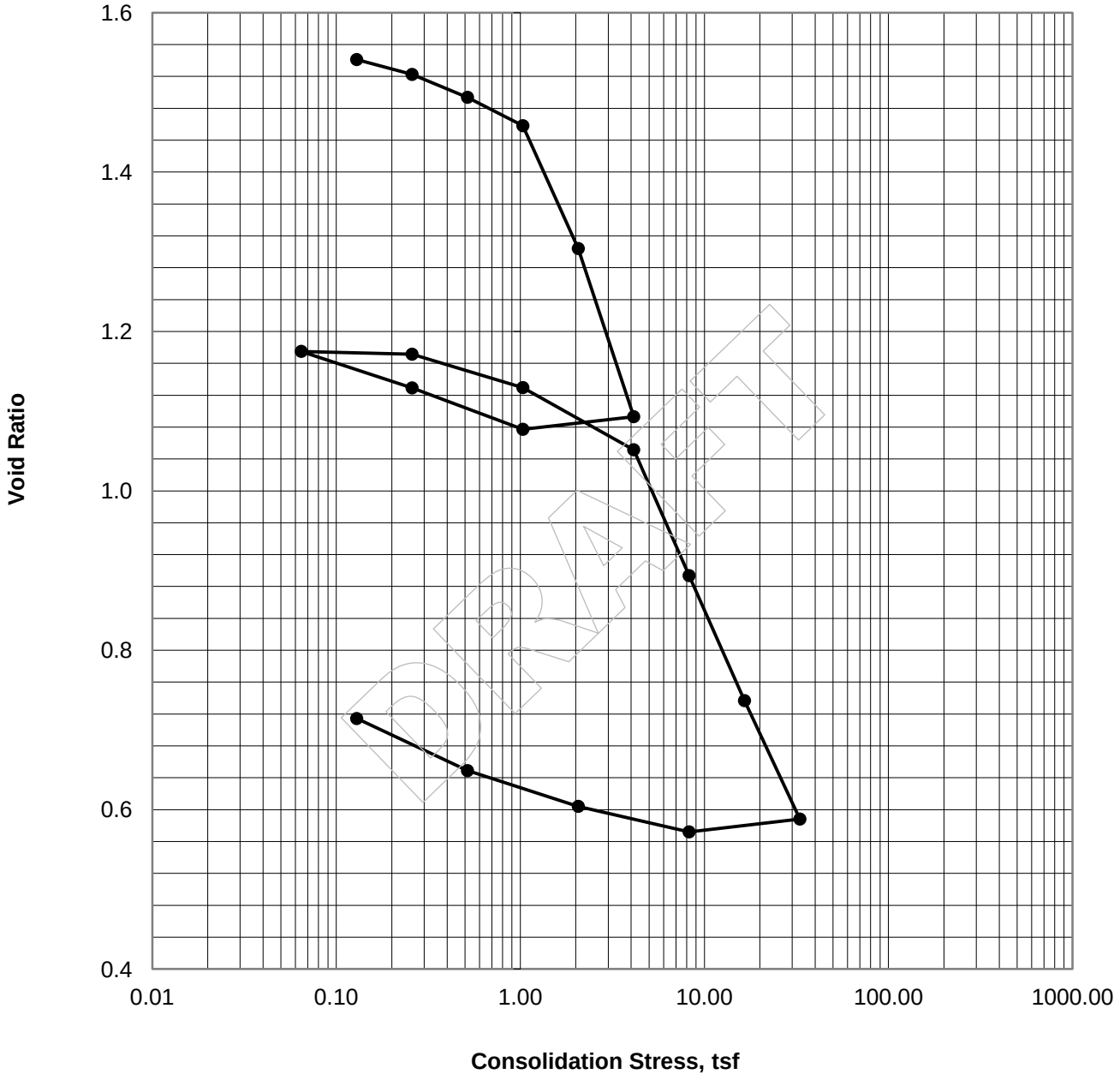
FIG. B-13

ONE DIMENSIONAL CONSOLIDATION TEST

Boring	B-2	Tested By	AKV
Sample	S-9	Calculated By	AKV
Depth, ft	23.3	Checked By	JFL



Maximum Load, tsf 33.02

NOTES:
1. Abbreviations:
   ft = feet
   tsf = tons per square foot

Port of Grays Harbor Terminal
Hoquiam, Washington

ONE DIMENSIONAL CONSOLIDATION
VOID RATIO vs STRESS PLOT
BORING B-2, SAMPLE S-9 @23.3ft

September 2013 21-1-21839-001

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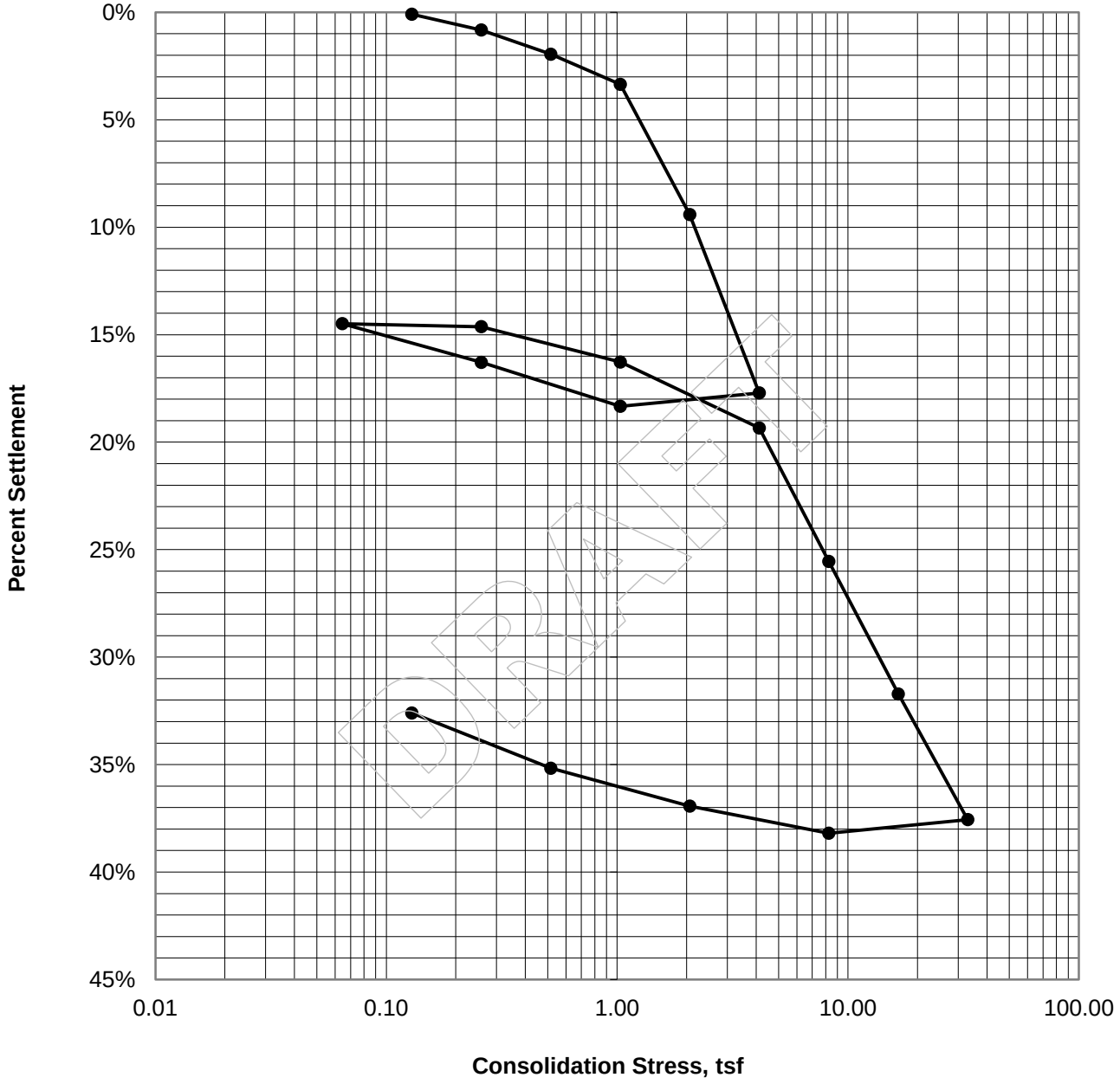
FIG. B-14

ONE DIMENSIONAL CONSOLIDATION TEST

Boring	B-2	Tested By	AKV
Sample	S-9	Calculated By	AKV
Depth, ft	23.3	Checked By	JFL



Maximum Load, tsf 33.02

NOTES:
1. Abbreviations:
   ft = feet
   tsf = tons per square foot

Port of Grays Harbor Terminal
Hoquiam, Washington

ONE DIMENSIONAL CONSOLIDATION PERCENT SETTLEMENT vs STRESS PLOT BORING B-2, SAMPLE S-9 @23.3ft

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FIG. B-15

ONE DIMENSIONAL CONSOLIDATION TEST

Boring	B-3	Tested By	AKV
Sample	S-4	Calculated By	JFL
Depth, ft	11.8	Checked By	JFL

SAMPLE CLASSIFICATION:
Gray-brown, silty CLAY/organic CLAY; CH/OH

SAMPLE DATA:

Specific Gravity (estimated)	2.4
Liquid Limit	109
Plastic Limit	41
Plasticity Index	68

SPECIMEN DATA:

	Before Inundation	First Load	Final Load
Height, inches	0.786	0.786	0.487
Diameter, inches	2.503	2.503	2.503
Sample Volume, cuin	3.867	3.867	2.396
Wet Density, pcf	91.8	91.7	113.6
Dry Density, pcf	47.3	47.2	76.3
Water Content, %	94%	94%	49%
Void Ratio	2.17	2.17	0.96
Saturation, %	100%	100%	100%

Increment	Applied Stress, tsf	ΔH at t_{100}, in	ΔH / H_o	Void Ratio	t_{50}, min	Coeff. of Comp., MPa^{-1}	Coeff. of Consol., cm^2/sec	Coeff. of Perm., cm/sec
1	0.10	0.000	0.0%	2.169	72.5	0.03	4.5E-05	4.8E-11
2	0.16	0.002	0.3%	2.161	1.3	1.31	2.5E-03	1.0E-07
3	0.32	0.010	1.3%	2.130	1.6	2.03	2.0E-03	1.2E-07
4	0.64	0.023	3.0%	2.076	1.5	1.76	2.0E-03	1.1E-07
5	1.29	0.058	7.4%	1.935	3.6	2.28	8.1E-04	5.9E-08
6	2.58	0.116	14.8%	1.702	5.6	1.89	4.5E-04	2.9E-08
7	5.15	0.178	22.7%	1.451	5.0	1.02	4.2E-04	1.6E-08
8	1.29	0.173	22.0%	1.472	2.9	0.06	6.7E-04	1.5E-09
9	0.32	0.152	19.3%	1.557	14.7	0.92	1.4E-04	5.1E-09
10	0.08	0.127	16.2%	1.656	94.3	4.26	2.4E-05	3.8E-09
11	0.32	0.129	16.4%	1.650	4.2	0.25	5.5E-04	5.1E-09
12	1.29	0.151	19.3%	1.559	4.2	0.98	5.3E-04	1.9E-08
13	5.15	0.192	24.5%	1.394	2.5	0.45	7.8E-04	1.3E-08
14	10.30	0.242	30.8%	1.192	6.7	0.41	2.5E-04	4.3E-09
15	20.61	0.294	37.5%	0.983	7.3	0.21	1.9E-04	1.8E-09
16	41.22	0.343	43.7%	0.786	8.8	0.10	1.3E-04	6.4E-10
17	10.30	0.3415	43.44%	0.793	6.9	0.002	1.5E-04	1.9E-11
18	1.29	0.3120	39.69%	0.912	40.3	0.138	2.8E-05	2.1E-10
19	0.32	0.2862	36.41%	1.016	431.9	1.124	2.9E-06	1.7E-10

DRAFT

NOTES:
1. Abbreviations:
   cm = centimeter
   cm^2 = square centimeter
   Coeff. = Coefficient
   Comp. = Compressibility
   Consol. = Consolidation
   cu in = cubic inch
   ft = feet
   H_o = initial height
   ΔH = change in height
   in = inch
   min = minute
   MPa = megapascal
   pcf = pounds per cubic foot
   Perm. = Permeability
   sec = second
   t_n = time at n% of primary consolidation
   tsf = tons per square foot

Port of Grays Harbor Terminal
Hoquiam, Washington

ONE DIMENSIONAL CONSOLIDATION TEST SUMMARY
BORING B-3, SAMPLE S-4 @11.8ft

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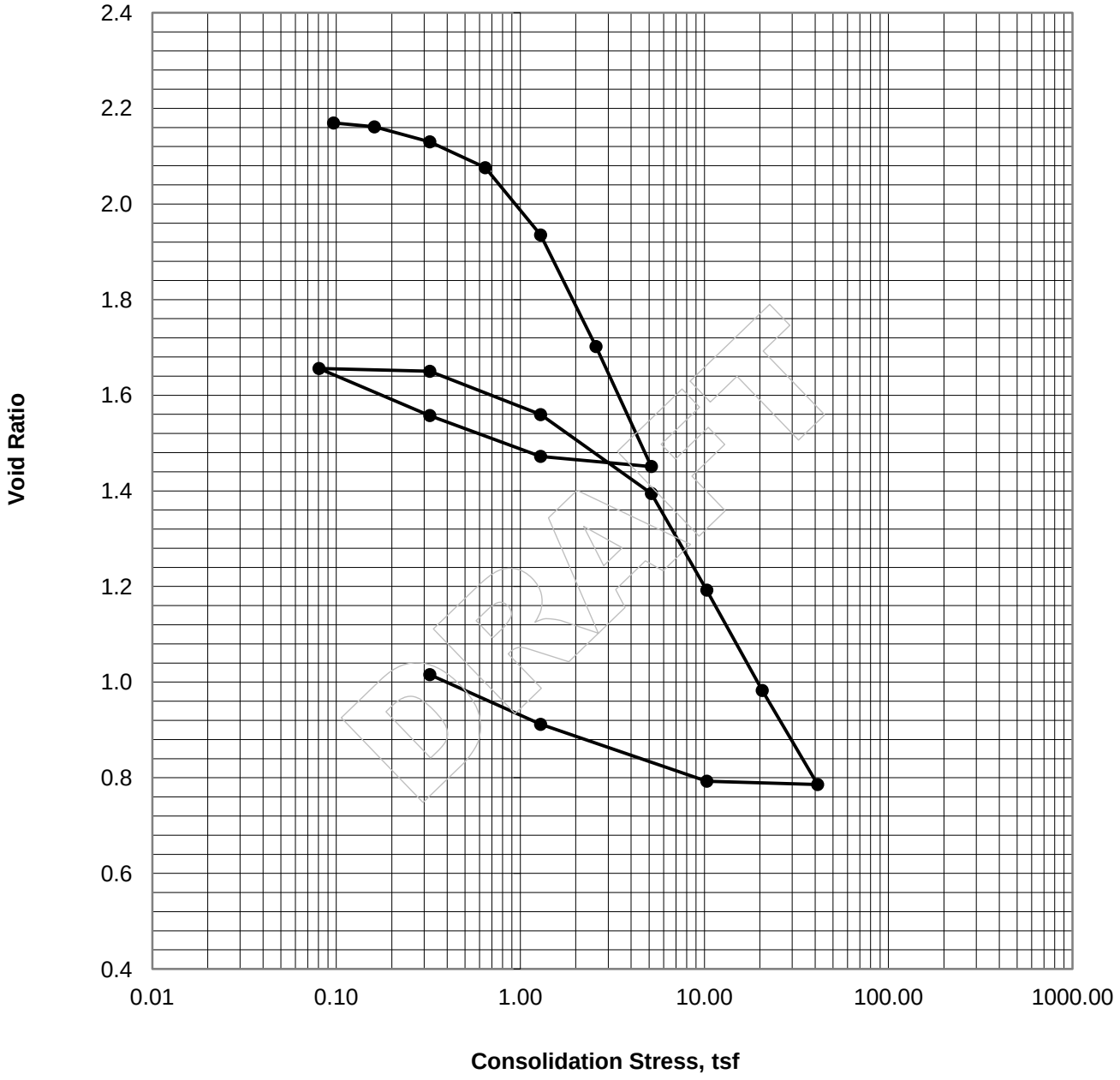
FIG. B-16

ONE DIMENSIONAL CONSOLIDATION TEST

Boring	B-3	Tested By	AKV
Sample	S-4	Calculated By	JFL
Depth, ft	11.8	Checked By	JFL



Maximum Load, tsf 41.22

NOTES:
1. Abbreviations:
   ft = feet
   tsf = tons per square foot

Port of Grays Harbor Terminal
Hoquiam, Washington

ONE DIMENSIONAL CONSOLIDATION
VOID RATIO vs STRESS PLOT
BORING B-3, SAMPLE S-4 @11.8ft

September 2013 21-1-21839-001

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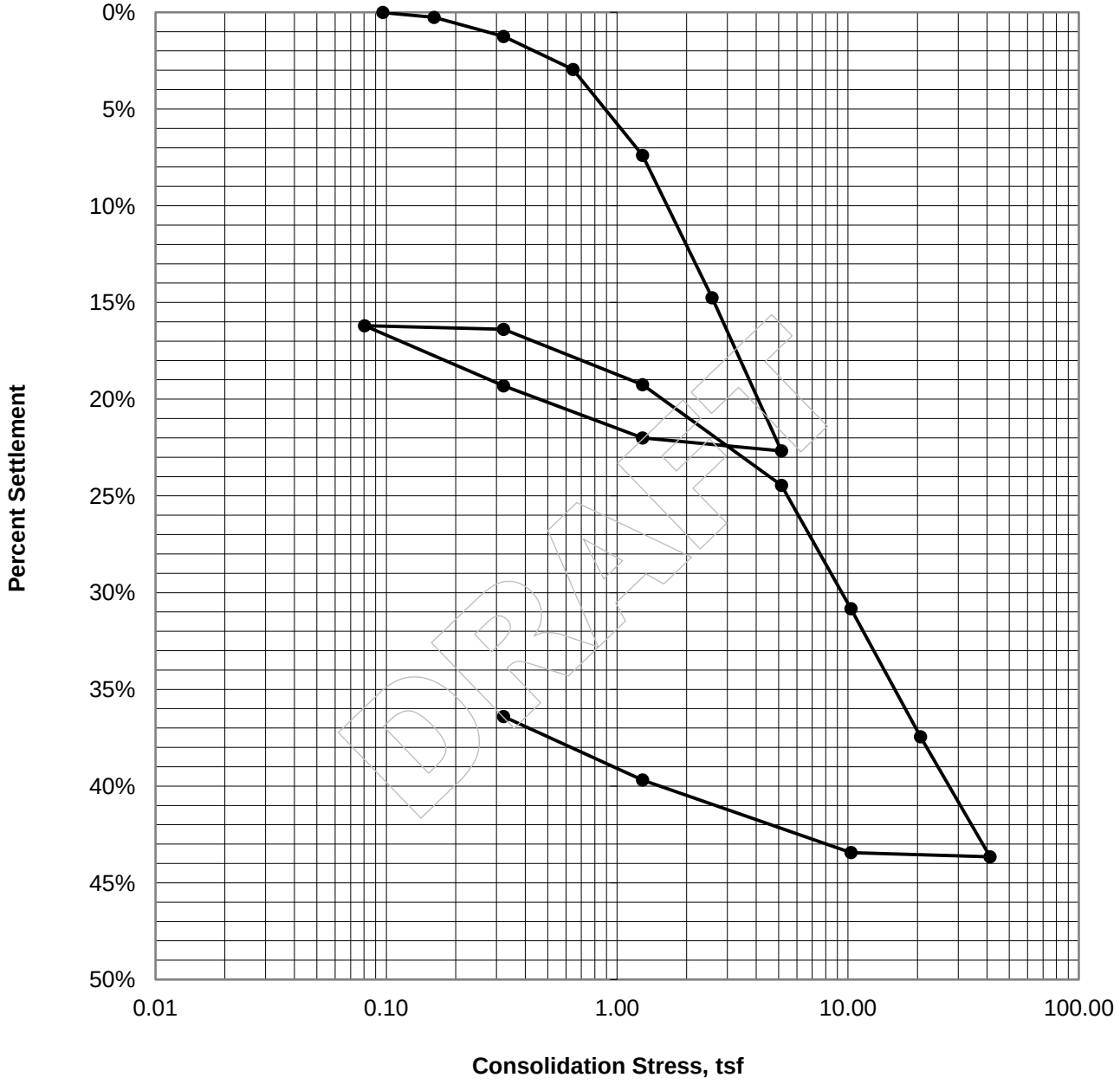
FIG. B-17

ONE DIMENSIONAL CONSOLIDATION TEST

Boring	B-3	Tested By	AKV
Sample	S-4	Calculated By	JFL
Depth, ft	11.8	Checked By	JFL



Maximum Load, tsf 41.22

NOTES:
1. Abbreviations:
   ft = feet
   tsf = tons per square foot

Port of Grays Harbor Terminal
Hoquiam, Washington

ONE DIMENSIONAL CONSOLIDATION PERCENT SETTLEMENT vs STRESS PLOT BORING B-3, SAMPLE S-4 @11.8ft

September 2013 21-1-21839-001

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FIG. B-18

ONE DIMENSIONAL CONSOLIDATION TEST

Boring	B-5	Tested By	AKV
Sample	S-26	Calculated By	AKV
Depth, ft	108.2	Checked By	JFL

SAMPLE CLASSIFICATION:
Gray-brown, silty CLAY/organic CLAY; scattered organics; specific gravity estimated; CH/OH

SAMPLE DATA:

Specific Gravity (estimated)	2.6
Liquid Limit	59
Plastic Limit	30
Plasticity Index	29

SPECIMEN DATA:

	Before Inundation	First Load	Final Load
Height, inches	0.786	0.786	0.585
Diameter, inches	2.503	2.503	2.503
Sample Volume, cuin	3.870	3.870	2.877
Wet Density, pcf	103.9	103.9	121.7
Dry Density, pcf	67.5	67.4	90.8
Water Content, %	54%	54%	34%
Void Ratio	1.41	1.41	0.79
Saturation, %	100%	100%	100%

Increment	Applied Stress, tsf	ΔH at t_{100}, in	ΔH / H_o	Void Ratio	t_{50}, min	Coeff. of Comp., MPa^{-1}	Coeff. of Consol., cm^2/sec	Coeff. of Perm., cm/sec
1	0.16	0.001	0.1%	1.402	0.4	0.23	7.8E-03	7.2E-08
2	0.32	0.005	0.6%	1.391	0.4	0.70	8.7E-03	2.5E-07
3	0.64	0.010	1.2%	1.376	0.4	0.51	8.4E-03	1.8E-07
4	1.29	0.017	2.1%	1.355	0.2	0.34	1.4E-02	2.0E-07
5	2.58	0.026	3.3%	1.327	0.1	0.23	2.4E-02	2.3E-07
6	5.15	0.055	7.0%	1.237	0.4	0.37	7.3E-03	1.1E-07
7	1.29	0.065	8.3%	1.206	0.2	-0.08	1.6E-02	5.9E-08
8	0.32	0.054	6.9%	1.240	0.8	0.37	3.7E-03	6.1E-08
9	0.08	0.045	5.8%	1.267	2.1	1.14	1.4E-03	7.0E-08
10	0.32	0.045	5.7%	1.268	0.4	-0.07	7.2E-03	2.3E-08
11	1.29	0.054	6.9%	1.239	0.3	0.32	1.1E-02	1.5E-07
12	5.15	0.073	9.3%	1.181	0.1	0.16	1.9E-02	1.3E-07
13	10.30	0.116	14.8%	1.050	0.6	0.27	4.1E-03	4.9E-08
14	20.60	0.171	21.7%	0.883	0.5	0.17	4.2E-03	3.4E-08
15	41.20	0.223	28.3%	0.724	0.8	0.08	2.4E-03	9.9E-09
16	10.30	0.231	29.4%	0.698	0.7	-0.01	2.3E-03	1.2E-09
17	1.29	0.2127	27.05%	0.755	4.1	0.065	4.1E-04	1.6E-09
18	0.32	0.1972	25.08%	0.802	43.3	0.511	4.1E-05	1.2E-09

DRAFT

NOTES:
1. Abbreviations:
   cm = centimeter
   cm^2 = square centimeter
   Coeff. = Coefficient
   Comp. = Compressibility
   Consol. = Consolidation
   cu in = cubic inch
   ft = feet
   H_o = initial height
   ΔH = change in height
   in = inch
   min = minute
   MPa = megapascal
   pcf = pounds per cubic foot
   Perm. = Permeability
   sec = second
   t_n = time at n% of primary consolidation
   tsf = tons per square foot

Port of Grays Harbor Terminal
Hoquiam, Washington

ONE DIMENSIONAL CONSOLIDATION TEST SUMMARY
BORING B-5, SAMPLE S-26 @108.2ft

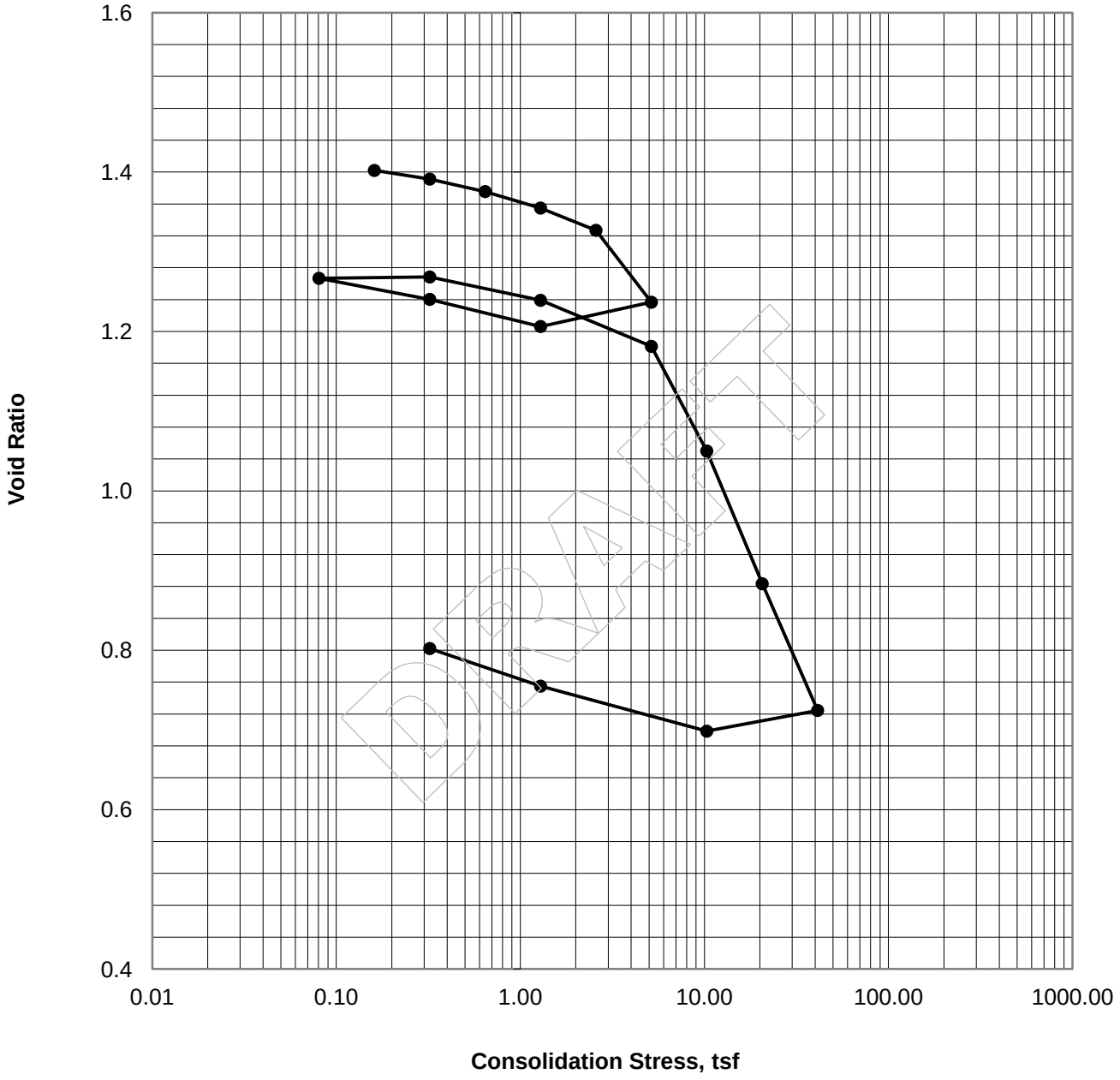
September 2013 21-1-21839-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. B-19

ONE DIMENSIONAL CONSOLIDATION TEST

Boring	B-5	Tested By	AKV
Sample	S-26	Calculated By	AKV
Depth, ft	108.2	Checked By	JFL



Maximum Load, tsf 41.20

NOTES:
1. Abbreviations:
   ft = feet
   tsf = tons per square foot

Port of Grays Harbor Terminal
Hoquiam, Washington

ONE DIMENSIONAL CONSOLIDATION
VOID RATIO vs STRESS PLOT
BORING B-5, SAMPLE S-26 @108.2ft

September 2013 21-1-21839-001

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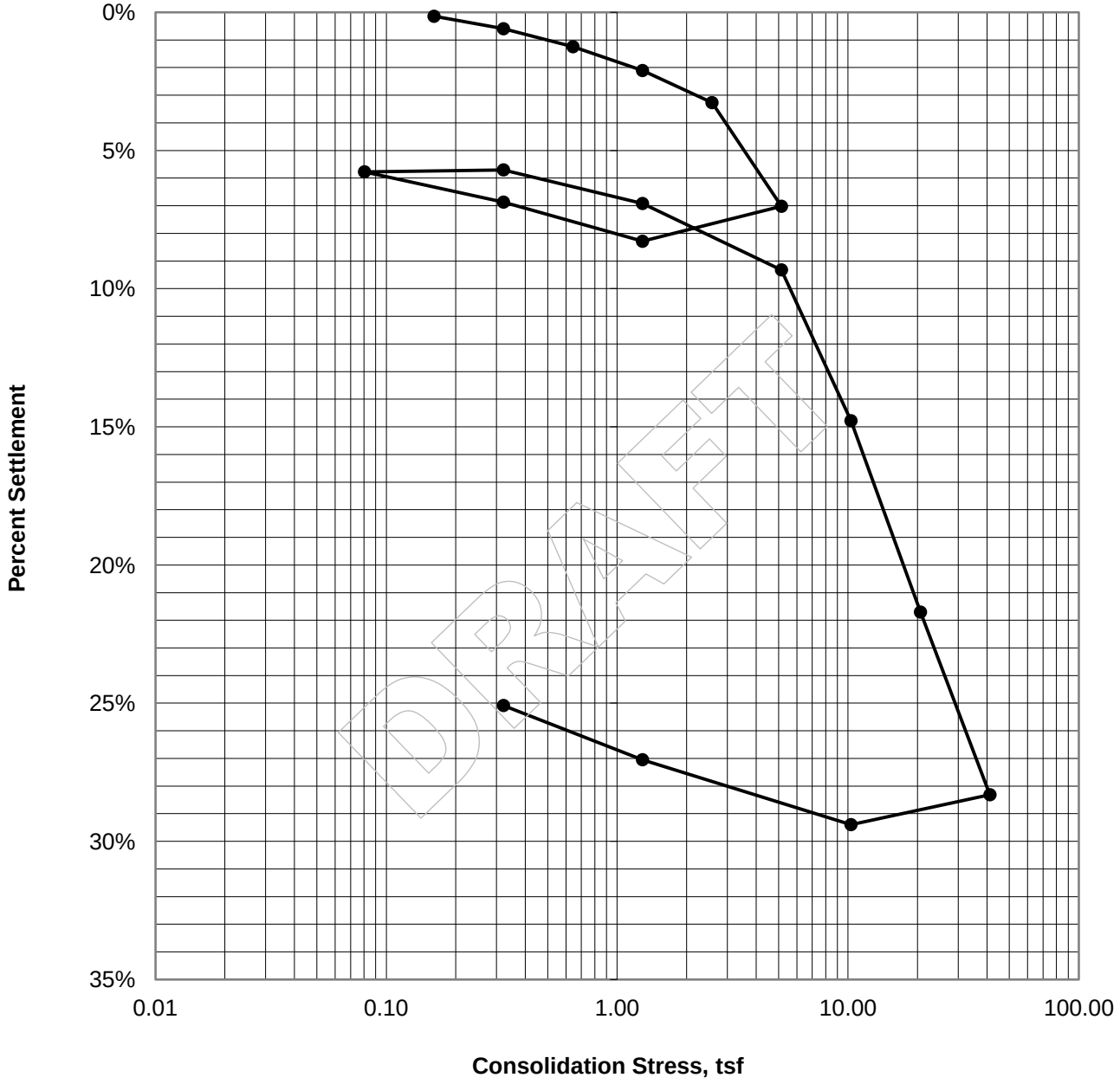
FIG. B-20

ONE DIMENSIONAL CONSOLIDATION TEST

Boring	B-5	Tested By	AKV
Sample	S-26	Calculated By	AKV
Depth, ft	108.2	Checked By	JFL



Maximum Load, tsf 41.20

NOTES:
1. Abbreviations:
   ft = feet
   tsf = tons per square foot

Port of Grays Harbor Terminal
Hoquiam, Washington

ONE DIMENSIONAL CONSOLIDATION PERCENT SETTLEMENT vs STRESS PLOT BORING B-5, SAMPLE S-26 @108.2ft

September 2013 21-1-21839-001

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FIG. B-21

APPENDIX C

IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL/ENVIRONMENTAL REPORT

IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL/ENVIRONMENTAL REPORT

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

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

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

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

SUBSURFACE CONDITIONS CAN CHANGE.

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

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

MOST RECOMMENDATIONS ARE PROFESSIONAL JUDGMENTS.

Site exploration and testing identifies actual surface and subsurface conditions only at those points where samples are taken. The data were extrapolated by your consultant, who then applied judgment to render an opinion about overall subsurface conditions. The actual interface between materials may be far more gradual or abrupt than your report indicates. Actual conditions in areas not sampled may differ from those predicted in your report. While nothing can be done to prevent such situations, you and your consultant can work together to help reduce their impacts. Retaining your consultant to observe subsurface construction operations can be particularly beneficial in this respect.

A REPORT'S CONCLUSIONS ARE PRELIMINARY.

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

THE CONSULTANT'S REPORT IS SUBJECT TO MISINTERPRETATION.

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

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

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

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

READ RESPONSIBILITY CLAUSES CLOSELY.

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

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