SUNOCO LP

May 30, 2024

To whom it may concern,

I am Shore Terminals LLC's point of contact for this matter and have uploaded the Seismic Vulnerability Assessment (Report) for the Portland Terminal. Payment of the $39,000 fee was made on May 29, 2024. Shore Terminals LLC is responsible for this Report.

The Report is submitted in response to the new Fuel Tank Seismic Stability Rules (Rules) recently adopted by the Oregon Department of Environmental Quality (DEQ). The Report summarizes the Terminal's evaluation for all assets at the Terminal. The Report also identifies the tanks, pipelines, and secondary containment that are within the jurisdiction of the Pipeline and Hazardous Materials Safety Administration (PHMSA) of the US Department of Transportation (DOT), and fall under the pre-emption of the federal pipeline safety laws, 49 U.S.C. 60101 et seq., (Pipeline Safety Act). The authorizing statute for the Rules, 2022 Oregon Senate Bill No. 1567 (Senate Bill), states that certain facilities are exempt from the requirements of the Rules due to federal preemption under the Pipeline Safety Act. The Rules are preempted by the Pipeline Safety Act so they do not apply to the tanks, pipelines and secondary containment that are subject to DOT/PHMSA jurisdiction.

By submission of this Report, Shore Terminals does not submit to the jurisdiction of the DEQ with respect to application of the Rules to tanks, pipelines and secondary containment that are subject to DOT/PHMSA jurisdiction. Shore Terminals reserves all of its rights and remedies in this regard. We look forward to discussing the Report with your team. Thank you for the consideration.

You may be aware of Sunoco LP's recent acquisition of NuStar Energy L.P through an all-stock purchase. This transaction provides the combined company with increased stability while continuing our excellent track record of health, safety, and environmental compliance.

Importantly, all NuStar operating companies, including Shore Terminals LLC, still own and operate the same assets as before the transaction. In other words, no asset transfer or change of operational control has occurred. As such, all company names on terminal and pipeline permits and plans will remain the same at this time. However, our communications going forward will be on Sunoco letterhead and from the Sunoco.com email domain.

Sincerely yours,

Tim Fluitt, Sr. Project Engineer

Sunoco LP

SEISMIC VULNERABILITY ASSESSMENT OF SHORE TERMINALS PORTLAND TERMINAL

Shore Terminals LLC
Portland, OR
May 2024

SGH Project 237367

Gayle S. Johnson

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PE Stamp applies to Report Body and Appendix C

PREPARED FOR

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EXECUTIVE SUMMARY

Shore Terminals LLC (Shore Terminals) has contracted Simpson Gumpertz & Heger Inc. (SGH) to perform a Seismic Vulnerability Assessment of the Shore Terminals Portland Terminal to comply with the new "Fuel Tank Seismic Stability Rules" (Rules) recently adopted by the Oregon Department of Environmental Quality (DEQ). This report presents the geotechnical, structural, and safety assessments performed. Key vulnerability findings are summarized below and discussed in further detail in this report.

The report summarizes the terminal's evaluation per the Rules for all assets at the Terminal. The report also identifies the tanks, pipelines, and secondary containment that fall under the pre-emption of the federal pipeline safety laws, 49 U.S.C. 60101 et seq., ("Pipeline Safety Act"), and are within the jurisdiction of the Pipeline and Hazardous Materials Safety Administration (PHMSA) of the US Department of Transportation (DOT). Tables E-1 and E-2 present only assets that are not exempt by DOT/PHMSA jurisdiction. Categorization of all terminal assets is provided in Appendix C.

Items are categorized as Moderate or High Risk based on the full consideration of hazards, including earthquake induced ground deformations. For High Risk items, mitigations should be considered using an As Low As Reasonably Practicable (ALARP) risk reduction philosophy. For Moderate Risk items, further evaluation is recommended to determine if mitigation is necessary. For example, this may include detailed engineering calculations to quantify the seismic capacity of specific, existing components.

Table E-1 - Summary of High Risk Items

Docks	Safety Systems & Buildings
P-2 Wharf	Water Main
P-2 Piping	Foam System
P-3 Wharf	Fire Pump
P-3 Piping	Hydrants

Table E-2 - Summary of Moderate Risk Items

Tank Farm 1	Tank Farm 3 & 4	Other Liquids (outside Main Yards)	Docks	Safety Systems & Buildings
Non-DOT/PHMSA Piping	Non-DOT/PHMSA Piping	Oil Water Separator	P-2 Dock Office	#10 Foam House
T-2113	T-1011	Drummed Waste Storage	P-3 Dock Office	#13 Foam House
T-3605	T-3201	EMS Overfill Tank (Truck Loading Rack)		
		TLR Piping		

Geotechnical

We have determined a peak ground acceleration (PGA_M) of 0.49g for the ASCE 7-16 DLE event. Median estimates of seismically-induced lateral ground deformations varies from about 2.5 ft near the truck loading rack, to over 5 ft in the middle of the tank farms and east toward the river. Corresponding vertical displacements vary from 12 in. to over 30 in. at the site, with the potential for higher localized settlements.

Our structural and safety assessments considered these potential displacements.

Structural

Most tanks have a Moderate Risk due to their Likelihood of damage particularly in the soil flow slide zone.

Pipelines are rated Moderate throughout the terminal due to differential displacements from ground deformation and the anticipated pipe stresses. At the dock, pipelines are rated High due to a higher consequence of damage and spill directly into the river.

The secondary containment walls are rated High due to their importance in containing spills and the uncertainty in their capacity to withstand seismic loads due to their age and construction. However, secondary containment walls and berms are under the DOT/PHMSA jurisdiction, so they are not subject to the Rules.

Safety

The water supply is rated as a High Risk seismic vulnerability. The facility relies on municipal water as its only source for firewater and foam distribution. It is highly unlikely municipal water will be available following the DLE considered by the Rules.

Since the foam system is dependent on municipal water, which is unlikely to be available following the DLE, and the consequence of this system being unavailable, this item is deemed a High Risk.

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APPENDIX

1. INTRODUCTION

Shore Terminals LLC (Shore Terminals) has contracted Simpson Gumpertz & Heger Inc. (SGH) to perform a Seismic Vulnerability Assessment of the Shore Terminals Portland Terminal to comply with the new "Fuel Tank Seismic Stability Rules" (Rules) recently adopted by the Oregon Department of Environmental Quality (DEQ).

The following report summarizes the terminal's evaluation per the Rules for all assets at the Terminal. The report also identifies the tanks, pipelines, and secondary containment that fall under the pre-emption of the federal pipeline safety laws, 49 U.S.C. 60101 et seq., ("Pipeline Safety Act"),[1] and are within the jurisdiction of the Pipeline and Hazardous Materials Safety Administration (PHMSA) of the US Department of Transportation (DOT). The authorizing statute for the Rules, 2022 Oregon Senate Bill No. 1567 (Senate Bill), specifically recognizes that certain facilities are exempt from the requirements of the Rules due to federal preemption under the Pipeline Safety Act. Section 3a of the Senate Bill provides as follows: "The requirements of sections 2 to 6 of this 2022 Act do not apply to a bulk oils or liquid fuels terminal to the extent those requirements are preempted by the federal Pipeline Safety Improvement Act of 2002, 49 U.S.C. 60101 *et seq.*"

Under the Pipeline Safety Act, § 60105(c), only a "State authority that has submitted a current certification under section 60105(a) of this title may adopt additional or more stringent safety standards for intrastate pipeline facilities and intrastate pipeline transportation only if those standards are compatible with the minimum standards prescribed under this chapter. A State authority may not adopt or continue in force safety standards for interstate pipeline facilities or interstate pipeline transportation. Notwithstanding the preceding sentence, a State authority may enforce a requirement of a one-call notification program of the State if the program meets the requirements for one-call notification programs under this chapter or chapter 61." The Oregon DEQ is not a State authority with a certification under section 60105(a). As such, it has no authority to impose pipeline safety regulations like the Rules over DOT/PHMSA regulated

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[1] 2022 Oregon Senate Bill No. 1567, Oregon Eighty-First Legislative Assembly, 2022 Oregon Senate Bill No. 1567, Oregon Eighty-First Legislative Assembly

facilities.[2] By submission of this report Shore Terminals does not submit to the jurisdiction of the DEQ with respect to the Rules and any application to tanks, pipelines, secondary containment, or other assets that are subject to DOT/PHMSA jurisdiction.

1.1 Background

The DEQ developed the Rules to address the risks related to a Cascadia Subduction Zone earthquake impacting large capacity fuel handling facilities in Columbia, Lane, and Multnomah counties in Oregon. Rule 340-300-0003 specifies the requirements and timeline to perform a seismic vulnerability assessment. The Seismic Vulnerability Assessment is a detailed, facility-wide, site-specific evaluation of the risk of seismically induced damage and secondary effects to a facility and environment when subjected to a Design Level Earthquake (DLE). The Rules require that for the purposes of this study, the DLE be determined in accordance with ASCE 7-16. This results in a very large earthquake (with a moment magnitude greater than 9.0) representing the Cascadia Megathrust fault, as described further in Section 3.5.

Rule 340-300-0002(18) defines the "Performance Objective" as limiting structural damage resulting in a spill exceeding the Maximum Allowable Uncontained Spill (MAUS) when the facility experiences DLE ground motions. Rule 340-300-0002 defines the maximum uncontained quantity of spill as one barrel (42 gal) or less for each tank or associated equipment, by reference to the reportable volumes in Oregon Law OAR 340-142.

Rule 340-300-0003 specifies the following elements be included in the Seismic Vulnerability Assessment:

- Description of facility components in terms of construction, age, inspection, maintenance, and operations.
- Summary of currently implemented spill prevention and mitigation measures and their ability to achieve the Performance Objective.

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[2] *See e.g., Olympic Pipeline Co. v. City of Seattle*, 437 F.3d 872 (9th Cir. 2006) (confirming that only states with a current § 60105(a) certificate are permitted to adopt additional or more stringent safety standards for intrastate pipelines, and holding that Seattle's pipeline safety demands were expressly preempted by the PSA because the city did not have a certification with the DOTto regulate the safety of hazardous liquid pipelines.).

- Definition of the DLE.
- Evaluation of the potential for a spill exceeding the MAUS during the DLE for all components in the facility
- Evaluation of the potential for liquefaction, lateral spreading, and seismically- induced settlement
- Evaluation of the safety of operating conditions, safe shutdown procedures, and potential spills
- Evaluation of the availability and integrity of automated sprinkler systems and sufficient supplies of firefighting foam and other emergency response equipment located in seismically resilient locations accessible after an earthquake to mitigate the risk of fire and explosions following an earthquake
- Evaluation of fire control measures such as firewalls surrounding the facility to limit fire spreading into surrounding communities
- Evaluation of the availability of day and night onsite personnel trained in emergency response and able to respond in the event of an earthquake

1.2 Scope of Work

The scope of work consisted of the following assessments consistent with Rule 340-300-0003(6)(a-c):

- Geotechnical Assessment including:
    - Site conditions assessment
    - Seismic hazard evaluation
    - Geotechnical evaluation
- Structural Assessment
- Safety Assessment including:
    - Fire control and suppression systems evaluation
    - Spill containment system evaluation
    - Evaluation of onsite emergency equipment, operational safety measures, and personnel availability

1.3 Assessment Boundaries

The team considered possible scenarios due to earthquakes that may realistically occur and result in an uncontained spill, uncontrolled fire, explosion, or toxic release at the terminal.

The following items were excluded from the scope of this study:

- Failures due to non-earthquake-related causes
- Life-safety considerations that are not directly caused by a spill that occurs due to an earthquake (e.g. life-safety concerns from occupants of a building that collapses)

1.4 Assessment Criteria

Rule 340-300-0002(4) lists codes and standards for use in this assessment. This list includes ASCE 7 for seismic design criteria, building structures, piping and pipe racks, and secondary containment, ASCE 41 for existing buildings, API 650 and API 653 for tanks, and ASCE 61 for piers, wharves, and waterfront structures. As permitted by Rule 340-300-0002(4)(h), the team considers "other applicable standards" to include:

- "Guidance for California Accidental Release Prevention (CalARP) Program Seismic Assessments," prepared for the Unified Program Agency (UPA) Subcommittee of the Region I Local Emergency Planning Committee (LEPC), January 2019, also referred to as the "CalARP Seismic Guidance Document".
- California Building Code (CBC) Chapter 34F, otherwise known as Marine Oil Terminal Engineering and Maintenance Standards (MOTEMS), 2022.
- "Seismic Evaluation and Design of Petrochemical and Other Industrial Facilities, 3rd Edition, American Society of Civil Engineers (ASCE), 2020.

The CalARP Seismic Guidance Document has a long history, being widely used within the industry in California for seismic assessment of existing chemical and process facilities in high seismic zones that contain hazardous materials. Further, MOTEMS is considered the most appropriate code document for assessment of operational procedures and seismic performance at existing marine oil terminals. Both of these documents also reference the ASCE document noted above. That document is widely used throughout industry and is frequently accepted by building officials for its interpretation of building code provisions as specifically relevant to typical structures and systems found in petrochemical and industrial facilities.

1.5 Limitations

SGH has performed the professional services for this project using the degree of care and skill ordinarily exercised under similar circumstances by reputable engineers practicing in the structural and earthquake engineering fields in this or similar localities. SGH makes no other warranty, expressed or implied, as to the professional advice included in this report. We have prepared this report for Shore Terminals to be used solely for the purposes of satisfying the requirements of the DEQ Rules. We have not prepared the report for use by other parties and the report may not contain sufficient information for purposes of other parties or for other uses. The recommendations resulting from this assessment rely on information provided by Shore Terminals to SGH, including soils reports, drawings, and specifications. SGH makes no warranty as to the accuracy and correctness of any such information.

Please note that addressing vulnerabilities identified in our report may reduce the risk, but does not guarantee or assure that a release will not occur in an earthquake. All parties should recognize the lack of complete assurance connected with seismic evaluations, especially of existing facilities. Uncertainties exist associated with material properties and structural behavior (uncertainties that are typically larger for existing facilities than new designs), as well as large uncertainties associated with earthquake motion in terms of amplitude, frequency content, direction, and duration. All parties should also recognize that seismic assessments such as those performed in this review require the significant application of professional experience and engineering judgment. Some amount of uncertainty and variation will always exist with respect to the interpretation of data, notwithstanding the exercise of due professional care.

This assessment emphasized identification of vulnerabilities and not conformance to building codes for new design. We further note that conformance to new design codes does not eliminate seismic risk, and industry standards for seismic evaluation of existing facilities consistently have been developed with the intent of reducing risk, and not for compliance with new design codes.

2. FACILITY DESCRIPTION

The Shore Terminals Portland Terminal is located at 9420 NW St Helens Road in Portland, Oregon. The terminal has two docks that extend into the Willamette River. The facility consists of five tank farms, two docks, truck loading racks, and several buildings, including the main office, operations building, maintenance shop, garage, storage, and foam houses. Note that some individual tanks are permitted for more than one product; this assessment reports the latest products as of the submittal. See Figure 2-1 for the vicinity plan of the Shore Terminals Portland Terminal. See Figure 2-2 for the aerial plan of the facility.

Figure 2-1: Vicinity Plan of Shore Terminals Portland Terminal

Figure 2-2: Aerial Plan of Shore Terminals Portland Terminal

2.1 Tank Farm #1

Tank Farm #1 consists of eight product tanks, three out of service tanks, and a vapor tank. There are twelve total tanks in the containment area. There are seven tanks with a diameter larger than 60 feet, while the remainder tanks have a diameter less than 48 feet. The tanks are squat tanks with an aspect ratio (height divided by diameter, H/D) less than 1.0. Several pumps and an oil water separator are located within the tank farm. Pipes interconnect the tanks and penetrate the containment walls, leading to the truck loading racks. The containment consists primarily of reinforced concrete walls approximately 15 feet high. Per the terminal's Spill Prevention, Control, and Countermeasure (SPCC) documentation, Tank Farm #1 has a containment volume of about 3,243,800 gallons (77,233 bbl). See Figure 2-3 for an aerial view of the Tank Farm #1. The secondary containment walls, five of the tanks, and associated piping are under the DOT/PHMSA jurisdiction, so they are not subject to the Rules.

Figure 2-3: Aerial Plan of Tank Farm 1 & 2, P-2 Dock and Truck Loading Rack

2.2 Tank Farm #2

Tank Farm #2 consists of six product tanks. There are three tanks with a diameter larger than or equal to 80 feet, while the remainder of the tanks have a diameter of 60 feet. The tanks are squat tanks with an aspect ratio (height divided by diameter, H/D) less than 1.0. Several pumps and other mechanical equipment are located within the tank farm. Pipes interconnect the tanks and penetrate the containment walls, leading out of the containment area. The containment consists primarily of reinforced concrete walls approximately 10 feet high. Per the SPCC, Tank Farm #2 has a containment volume of around 3,970,400 gallons (94,534 bbl). See Figure 2-3 for an aerial view of the Tank Farm #2. The secondary containment walls, all the tanks, and associated piping are under the DOT/PHMSA jurisdiction, so they are not subject to the Rules.

2.3 Tank Farm #3

Tank Farm #3 consists of four products tanks. There are three tanks with a diameter larger than 105 feet, while the fourth tank has a diameter of 90 feet. Several pumps and other mechanical equipment are located within the tank farm. Pipes interconnect the tanks and penetrate the containment walls, leading out of the containment area. The containment consists of reinforced concrete walls approximately 5 feet high on the north, west, and south side, and an earthen, asphalt-covered berm on the east side. Tank Farm #3 and Tank Farm #4 are connected to allow spills to flow contained from one farm to the other. Per the SPCC their shared containment volume is about 5,292,600 gallons (126,015 bbls). See Figure 2-4 for an aerial view of the Tank Farm #3. The secondary containment walls, containment berm, all the tanks, and associated piping are under the DOT/PHMSA jurisdiction, so they are not subject to the Rules.

Figure 2-4: Aerial of Tank Farm 3

2.4 Tank Farm #4

Tank Farm #4 consists of eleven products tanks. Eight of the tanks have diameters larger than 70 ft with an aspect ratio less than 1.0. The remainder of tanks have a diameter of 40 feet and an aspect ratio larger than 1.0. Several pumps and other mechanical equipment are located within the tank farm. Pipes interconnect the tanks and penetrate the containment walls, leading out of the containment area The containment consists primarily of reinforced concrete walls approximately 5 feet high and one earthen ramp. Tank Farm #3 and Tank Farm #4 are connected to allow spills to flow contained from one farm to the other. Per the SPCC their shared containment volume is about 5,292,600 gallons (126,015 bbls). See Figures 2-5 for an aerial view of Tank Farm #4. Secondary containment walls, nine of the tanks, and associated piping are under the DOT/PHMSA jurisdiction, so they are not subject to the Rules.

Figure 2-5: Aerial of Tank Farm 4 & 5 and P-3 Dock

2.5 Tank Farm #5

Tank Farm #5 has two product tanks. Both tanks have a diameter of 120 ft and a height of 58 ft, resulting in an aspect ratio of 0.48. Piping interconnects the tanks and penetrates the containment walls, leading out of the containment area. The containment consists primarily of reinforced concrete walls approximately 10 feet high. Per the SPCC, Tank Farm #5 has a containment volume of about 5,444,700 gallons (129,636 bbl). See Figure 2-5 for an aerial view of Tank Farm #5. Secondary containment walls, both tanks, and associated piping are under the DOT/PHMSA jurisdiction, so they are not subject to the Rules.

2.6 Docks

The P-2 Dock (North Wharf) is located north of Tank Farms #1 and #2. It extends approximately 105 feet into the Willamette River and consists of concrete, steel, and timber piles, beams, and decking. Piping runs beneath the concrete decking towards the shore. Four mooring dolphins with concrete piles are adjacent to P-2.

The P-3 Dock (South Wharf) is located north of Tank Farms #4 and #5. The dock extends approximately 155 feet into the Willamette River and consists of steel and timber piles, beams, and decking. Piping runs on the north side of the dock on steel supports. There are four mooring dolphins with steel piles adjacent to the P-3 Dock. Gangways connect the mooring dolphins to the main portion of the P-3 Dock.

Both docks have containment berms around the deck and walkways to provide containment of stormwater runoff or released product.

See Figure 2-3 for P-2 Dock. See Figure 2-5 for P-3 Dock.

2.7 Loading Racks

The truck loading rack is located west of Tank Farm #2. The rack consists of four lanes (one of which is currently out of service) for unloading products. Piping from the tank farms runs underground to each drive lane. The truck loading rack consists of steel framed construction with corrugated metal deck roofing. See Figures 2-3 for an aerial view of the Truck Loading Rack.

2.8 Buildings

The Main Office is a two-story building located near the facility's entrance. Building #10 Foam House and Building #11 are small, light-gauge steel prefabricated buildings south of the truck loading racks. Building #13 Foam House and Building #1 are similar small buildings located between Tank Farms #2 and #3, and between Tank Farms #3 and #4, respectively. Building #4 Operations is a steel-framed building with corrugated steel roofing located west of Tank Farm #2. Building #5 Maintenance, Building #8 Garage, and Building #13 Storage are located along the shore east of Tank Farms #1 and #2. The Garage building has concrete masonry walls with steel framing and corrugated steel roofing. The Maintenance building and Storage building are steel-framed buildings with corrugated steel roofing. The foundation system is unknown for all buildings.

The buildings on site do not contain or store fuels, and therefore, hazardous material release is not an issue.

See Figure 2-6 for an aerial view of the Shore Terminals Portland Terming buildings, with the exception of the Main Office and the Building #1 which are further south, as described above.

Figure 2-6: Aerial view of Shore Terminals Portland Terminal Buildings

Detailed plot plans and tank inventory are provided in Appendix A.

3. GEOTECHNICAL ASSESSMENT

A geotechnical assessment was performed to provide input for the Seismic Vulnerability Assessment. The assessment included consideration of existing site-specific geotechnical information, other existing data, and data from a geotechnical exploration performed by Gannett Fleming Inc. (Gannett Fleming) in 2024. The full geotechnical assessment report is included in Appendix B.

3.1 Site Conditions

The terminal is located on the east side of NW St. Helens Road just east of the foothills of the Tualatin Mountains and west of the Willamette River as shown in Figure 2-1. Liquid products are transferred out of and into the terminal via several modes including the rail rack, truck loading rack, pipeline, and piers. The rail lines bisect the site and are aligned roughly parallel to the shoreline. The tank yards are located between the waterfront and existing rail lines, with a ground surface at roughly elevation 35 feet (NAVD88). The ground surface west of the tank yards slopes up gently to about elevation 45 feet at the location of the truck loading rack, with higher ground and steeper slopes near the truck loading rack on the west boundary of the site adjacent to NW St. Helens Road. Data from a bathymetric survey completed by AKS Engineering & Forestry, LLC (AKS) in 2023 indicate the waterfront slope is roughly 70 feet high.

A bulkhead wall and concrete revetment were previously constructed along a portion of the waterfront slope from the north boundary of the terminal to roughly 600 feet south of this location. The bulkhead wall is about 8 ½ feet tall and founded on a shallow footing. The wall supports fill placed to facilitate construction of adjacent buildings and infrastructure adjacent to Pier P-2. In addition, concrete revetment was constructed on the waterfront slope up to roughly 7 feet (measured vertically) below the toe of the wall.

In addition, a subsurface slurry cutoff wall was previously constructed parallel to the shoreline near the top of the waterfront slope adjacent to Tank Farm #2 and Tank Farm #3 in support of groundwater remediation. We understand that ground improvement elements have been installed at Tank Farm #5 to address seismic stability impacts on the tanks and facilities at this location. As-built plans for the ground improvement are not available.

3.2 Existing Data

We reviewed existing reports for geotechnical investigations performed by others. This data includes three Cone Penetration Tests (CPTs), two test pits, and one hand boring in 2019 as reported by Landau Associates, Inc. (Landau), as well as two CPTS and five borings completed by CH2M Hill, Inc. (CH2M Hill) in 2006.

Data from these previous geotechnical investigations including exploration logs and laboratory test results are included in Appendix B. This information was considered as part of our geotechnical assessment.

3.3 Field Exploration

To supplement the existing data, Gannett Fleming performed a field exploration including three Seismic CPTs (SCPTs). The SCPTs were located at Tank Yard 3, between Tank Yard 2 and Tank Yard 3, and southeast of the Truck Loading Rack.

Three SCPTs were performed by ConeTec, Inc. on February 22, 2024. The SCPTs were advanced to refusal encountered at depths of about 20 to 50 feet. Considering the depth to bedrock encountered in previous explorations at the site, we anticipate the SCPTs encountered refusal at or near the top of bedrock. The interior of the cone penetrometer is instrumented with strain gauges that allow simultaneous measurements of cone tip and friction sleeve resistance during penetration. The testing creates computer-generated graphical logs of cone resistance, friction resistance, and friction ratio; which is used to interpret soil behavior type.

Seismic shear wave tests were also performed by stopping the penetration of the cone and the rods and decoupling them from the rig. A sledge hammer is used to manually trigger a shear wave into the soil. The distance from the source to the cone is calculated based on the total depth of the cone and the horizontal offset distance between the source and the cone. An interval velocity is calculated using a minimum of two tests performed at two different depths.

CPT and shear wave velocity data is included in Appendix B.

3.4 Subsurface Conditions

The site is underlain by various amounts of fill materials placed during site development. Regional geologic mapping indicates the fill is underlain by young Quaternary alluvium comprised of river and stream deposits of silt, sand, clay, and peat of present flood-plains.

The previous and current exploration indicate subsurface conditions encountered that are generally consistent with site development and regional geology. Subsurface soils are primarily comprised of fill, alluvial deposits, and bedrock. The fill encountered at the site varies in thickness from about 9 to 23 feet, with greater thickness adjacent to the shoreline and decreased thickness on the west side of the site. The fine-grained alluvium encountered is up to about 30 feet thick, and the underlying coarse-grained alluvium deposits are up to about 8 feet thick and are underlain by bedrock (Columbia River basalt). Shallow bedrock was encountered in the west portion of the site near NW St. Helens Road at a depth of about 2 to 6 feet southwest of the truck loading rack, with deeper bedrock up to about 64 feet encountered adjacent to the shoreline.

Pore pressure dissipation tests (PPDTs) performed during SCPTs completed as part of the current investigation indicate groundwater depths of about 3 ½ to 13 ½ feet, with shallower groundwater encountered on the west side of the site. PPDTs performed during the 2019 Landau investigation indicate groundwater depths of about 10 to 18 ½ feet. Groundwater levels were not measured during the 2006 investigation by CH2M Hill. Fluctuations in groundwater levels likely occur due to variations in the Willamette River water level, rainfall, underground drainage patterns, regional influence, and other factors.

3.5 Seismic Hazard Evaluation

We have evaluated seismic hazards including ground shaking, liquefaction, lateral spreading, and seismic densification. A summary of our conclusions regarding the potential for liquefaction and lateral spreading is provided below.

As required by the Rules, we developed seismic design parameters in accordance with the 2016 American Society of Civil Engineers (ASCE) Standard 7-16 (ASCE 7-16): Minimum Design Loads for

Buildings and Other Structures (ASCE 2016) for the purposes of evaluating liquefaction potential and lateral spreading. Based on the existing geotechnical data, the site can be characterized as Site Class C or D in conformance with ASCE 7-16. Using the ASCE 7 Hazard Tool, we calculated a maximum considered earthquake geometric mean (MCEG) peak ground acceleration adjusted for site class (PGAM) of 0.49g, corresponding to a moment magnitude (Mw) of 9.3 on the Cascadia Megathrust fault, which governs the seismic hazard at the site.

The results of our evaluation indicate the potential for liquefaction is high during the design earthquake. Related effects include ground surface settlements, sediment ejecta and settlement from ground loss. In addition to settlement from reconsolidation and sediment ejecta, liquefaction-induced foundation settlement can occur when shear-induced deformations driven by cyclic loading occur due to ratcheting and bearing capacity types of movement caused by soil structure interaction (SSI).

Lateral spreading is a phenomenon where a soil mass moves laterally on liquefied soil down a gentle slope or toward a free face, such as the adjacent Willamette River channel. Displacement occurs in response to gravitational and earthquake-induced forces acting on soils within and above the liquefied layer. The magnitudes of lateral displacement are expected to be significant near the Willamette River shoreline, reducing in magnitude with increasing distance from the waterfront slope. To estimate liquefaction-induced lateral displacements, we used a semiempirical approach developed by Zhang, et al. (2004).

During lateral spreading, surface layers commonly break into large blocks, which progressively migrate toward a free face. This development of ground fissures can promote ground loss for sediment ejecta and increase the likelihood of associated settlement.

3.6 Seismically-Induced Ground Deformations

We have developed preliminary estimates of vertical and lateral seismically-induced ground deformations to approximate the range of movements expected at the site.

Lateral displacements

Lateral deformations due to lateral spreading are depicted as geographic contours in Figure 3-1. These estimates consider the proximity of the site to the free face slope of the waterfront along the Willamette River and a slope height of 70 feet.

As shown in Figure 3-1, the estimated lateral spread deformations range from about 0 feet at west side of terminal, near NW St Helen’s Road to greater than 5 feet on the east side the terminal. In the flow slide zone, unlimited shear strains may develop leading to a flow-type failure. In this case, large masses of ground may travel long distances (likely more than 5 feet) in the form of liquefied flows or blocks of ground riding on liquefied flows. Most of the tanks are located within this flow slide zone.

It should be noted that the approach developed by Zhang, et al. (2004) and used to estimate deformations, could underestimate or overestimate lateral displacements by up to a factor of 2.

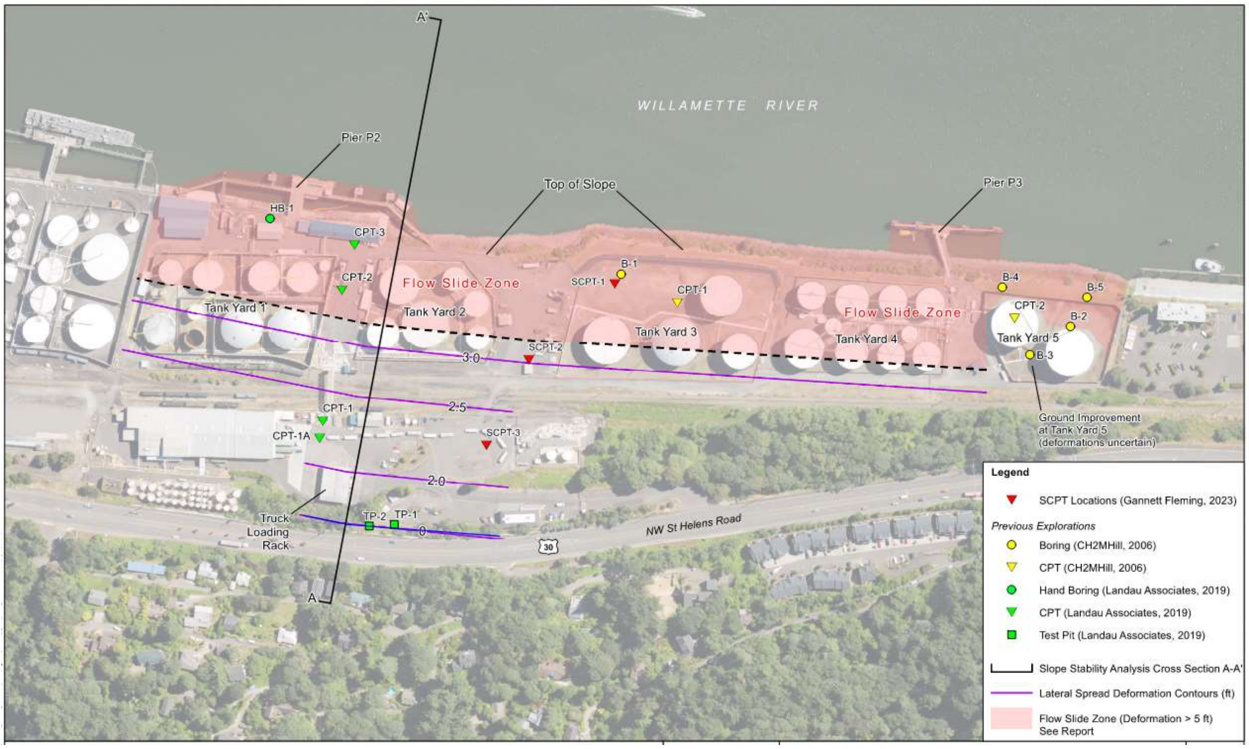


Figure 3-1: Estimated Lateral Spread Deformations (from Appendix B)

Vertical Settlements

Lateral spreading also results in ground settlement, which can be as much as about one-third to one-half of the magnitude of lateral displacement.

Additionally, liquefaction-induced settlement can occur. The primary mechanisms of liquefaction-induced settlement are reconsolidation (estimated as 2 to 6 inches), localized ejecta-induced settlements (up to 12 inches), and shear-induced foundation settlement (not estimated).

Combined with the vertical component of lateral spreading, the total estimated settlement, with free-field conditions, ranges from 2 to over 30 inches.

4. STRUCTURAL ASSESSMENT

Rule 340-300-0003(6)(b) identifies that a structural assessment is to be performed for all onsite structures where damage could result in a potential release of fuel.

The key structural assessment consisted of a walkdown evaluation of the entire facility, supplemented by limited reviews of available drawings and other documentation, such as tank inventory tables.

Our evaluation is based on the "expected" or "most likely" conditions at the time of an earthquake rather than the worst-case or conditions that might be considered for new design. This includes consideration of existing deterioration or damage and any modifications made since construction, as observed during the walkdown.

Considering the variability of tank operation (i.e. tanks are filled or emptied over days, weeks, or months) and input from Shore Terminals Operations regarding the likely fill heights based on actual operating procedures, a reasonable assumption for all tanks is that they are half full.

4.1 Walkdown Assessment

The walkdown assessment is a primarily visual review that considers the actual conditions of each installation in a systematic, methodical manner. The engineers performing the review investigate potential seismic vulnerabilities, focusing on proven failure modes from past earthquake experience, basic engineering principles, and engineering judgment. The walkdown review emphasizes the primary seismic load-resisting elements and the potential areas of weakness due to design, construction, modification practices, historical deterioration, or existing damage. A special emphasis is placed on details that may have been designed without consideration of seismic loads.

This walkdown assessment approach is widely used within industry, and in particular is used in California for assessing existing chemical and process facilities that contain hazardous materials.

The approach is documented in the "CalARP Seismic Guidance Document", which recommends that the walkdown follow the guidance provided by the American Society of Civil Engineers (ASCE) in their document, "*Guidelines for Seismic Evaluation and Design of Petrochemical Facilities*, 2nd Edition", published by ASCE, 2011. We also considered that document, as well as the 3rd Edition, published in 2020.

Our walkdown assessment considered the likely response due to ground shaking (inertial effects), as well as the likely damage due to liquefaction and lateral spreading associated with the DLE.

4.2 Likelihood of Spill from Seismic Structural Damage

We assigned a judgment-based, qualitative likelihood of spill to each structure, tank, and other installation within the terminal based on our walkdown assessment and associated document review.

For storage tanks, we have taken into consideration the historical performance of storage tanks regardless of whether designed to modern code requirements, emphasizing those details that have been proven by experience to increase the likelihood of damage that could lead to a spill. For this assessment, we considered criteria such as tank construction (i.e. riveted versus welded), whether the tank is anchored (anchored tanks historically perform very well), the aspect ratio of the tank (fill height to diameter ratio), and whether any piping, stairs, or other attachments are restrained in a manner that would over-constrain movement of the tank and cause stress concentrations or damage to attached piping.

For containment walls, the likelihood of structural failure in a seismic event is based on the type of containment (i.e. concrete wall versus soil berm), liner details, depth of wall foundations, geometries (i.e. width and toe), reinforcing details, and era of construction. We also considered the present condition as well as modifications made to containment walls, such as penetrations or reinforcing buttresses, if applicable.

For buildings and other building-like structures, we first considered whether damage to the structure would result directly in an uncontained spill, uncontrolled fire, or explosion or would damage a critical safety or control system, leading to the same effect. Buildings that do not store fuel products or contain critical safety systems were screened from further assessment. None of the buildings at this terminal store fuel products. For structures that contain products or critical systems within the scope of these rules, we considered the structure system, visible condition, and era of construction to determine a qualitative likelihood of damage that could lead to a spill.

We first determined a likelihood of spill due to earthquake-induced structural damage, without any consideration of the geotechnical ground displacements associated with liquefaction and lateral spreading. We then adjusted likelihood scores for individual elements, considering the estimated ground displacements within the geographic area where the equipment is located and the specifics of that structure (such as aspect ratio and foundation type). For example, significant ground displacement will increase the likelihood for overturning on unanchored tanks with a high aspect ratio, so we increased the Likelihood category accordingly.

5. SAFETY ASSESSMENT

We reviewed the fire systems and procedures, oil spill containment systems and procedures, and other emergency systems that would be affected by a major earthquake.

We also performed a walkdown of the site, met with the operator and held discussions, and participated in the risk assessment discussed in Section 6.

We considered realistic general earthquake effects that are likely to occur in a DLE, such as:

- Shaking of the entire facility simultaneously without prior warning.
- Lengthy duration of shaking (15 seconds or longer).
- Loss of grid power.
- Loss of municipal water.
- Multiple alarms triggered.
- Off-site emergency services may not be available due to infrastructure problems (bridges and highways) or regional needs for the general community.
- Unpredictable human response.

5.1 Spill Containment Systems, Equipment and Procedures

This section addresses Rule 340-300-0003(6)(c)(B) and Rule 340-300-0003(1)(d).

Primary Containment and Maintenance Procedures for Bulk Storage

All bulk storage tanks are constructed of steel and meet American Petroleum Institute (API) standards for oil storage tank construction. In addition, bulk storage tanks are operated according to API 650 or 653 and are inspected in accordance with industry standards, including:

- API Standard 653 for atmospheric storage tanks with a capacity of 50,000 or more.
- Steel Tank Institute (STI) Standard SP001 for atmospheric storage tanks for storage tanks with a capacity of 50,000 or less
- API Standard 510 for pressurized storage vessels.

Inspection intervals for all oil storage tanks have been established based on the referenced industry standards, including monthly visual inspections to verify integrity.

Personnel monitor all filling operations by using direct “side reading” level gauges and manual gauging techniques in order to prevent overfill of bulk storage tanks. In addition, the tanks are equipped with high level alarms, which operate independently of the manual gauges and are tested monthly to verify proper operation. The high-level alarms have both visible and audible indicators.

The tank truck loading rack is equipped with an overfill protection system which shuts down the loading automatically to prevent overflow. Totes of additives are located within the tank truck loading area, all within secondary containment.

Maintenance and Operation of Terminal Piping

All aboveground piping, valves and appurtenances are inspected monthly, including an assessment of the general condition of flange joints, expansion joints, valve glands and bodies, catch pans, pipeline supports, locking of valves and external surfaces.

Terminal piping includes buried sections of piping. All new buried piping is provided with protective coating and protected with Cathodic Protection. Integrity and leak testing of buried piping is performed at the time of installation, modification, relocation or replacement.

The terminal has pipeline surge control systems in place, which includes day valves designed to divert product if a pressure surge is present in the system.

Secondary Spill Containment Systems and Response Procedures

All bulk storage tanks are protected by a secondary containment system designed to contain the volume of the largest tank inside plus a 25-year, 24-hour storm. Secondary containment is composed, with one exception, of a dike made of concrete walls with a compacted clay floor.

Tank Farm #3 has one side comprised of a compacted earthen wall with an erosion prevention material on its surface.

Storage tank diked areas are equipped with drain valves to discharge rainwater to an Oil Water Separator (OWS). However, the drain valves remain closed except when actively discharging rainwater.

The truck loading rack is protected by 20,007 gallons of secondary containment, which exceeds the maximum 9,000 gallon tanker truck capacity.

Spill procedures are described in Figure 2.1-2 of the Facility Response Plan. In the event of a spill, initial response procedures include securing the area, shutting down terminal operations and shutting down motor operated valves to isolate sources of fuel. Manual valves are to be closed if safe to do so.

After initial response, terminal procedures include notification of first responders, Qualified Individuals, the Oil Spill Contractor, etc.

The terminal has an Incident Command System (ICS) in place.

Spill Mitigation Procedures are provided in Figure 2.1-3 of the Facility Response Plan, and include spill procedures for the following situations:

- Failure of transfer equipment
- Tank overfill / failure
- Piping Rupture / leak
- Fire / explosion
- Manifold Failure

Guidelines to prepare for an earthquake, and steps to take following an earthquake are provided in Section 2.9 of the Facility Response Plan.

Summary of Current Spill Prevention and Mitigation Measures

Tank design and maintenance is in accordance with industry standards. In addition, the terminal provides secondary containment for all petroleum or renewable fuels stored on site.

The terminal is equipped with containment boom, spill pads / diapers and absorbent material / boom. Terminal personnel is available and trained for boom deployment and use of absorbent materials and spill pads.

5.1.1 Seismic Vulnerabilities

Tanks in the tank farms are susceptible to damage following an earthquake from shaking or differential displacements. Similarly, piping is susceptible to damage from differential displacements of supports and anchor points.

If tanks or piping are damaged in an earthquake, the concrete containment walls that form part of the secondary containment are critical in controlling the spill and its associated environmental and safety hazards. These walls are also susceptible to damage during an earthquake. From a safety standpoint, loss of containment for a spill would potentially spread the life safety hazards over a larger area, including fire and exposure to hazardous materials.

5.2 Fire Control and Suppression Systems

This section addresses Rule 340-300-0003(6)(c)(A) and Rule 340-300-0003(1)(i).

Terminal firewater is provided by a municipal main located on Helens Rd. Firewater is boosted by a diesel fire pump, rated at 1000 gpm located inside a fire pump room. The fire pump is equipped with a dedicated diesel day tank and provides firewater to a series of loops serving Tank Farms #1 through #5. The fire main also serves a series of fire hydrants and fire hose standpipes located throughout the facility.

The fire control system includes a foam system located adjacent to the fire pump house. This foam system provides Ansulite AR-AFFF 3% foam to a majority of storage tanks in Tank Farms #1 through #3 and Tank Farm #5. Foam for tanks located in Tank Farm #4 is provided by a dedicated bladder foam tank proportioning system located between Tank Farm #4 and #5.

The firewater and foam available through most of the terminal meets or exceeds industry standards. A project is underway to extend firewater and foam coverage for the areas of the terminal currently underserved by firewater or foam.

The terminal includes two dock structures, the P-2 Dock located on the north side of the facility, and the P-3 Dock located on the South. The P-2 Dock is equipped with a standpipe system and an under dock dry pipe sprinkler system.

The terminal secondary containment dikes can act as firewalls, limiting spread of fire into adjacent areas.

5.2.1 Seismic Vulnerabilities

The firewater system and foam distribution system are dependent on municipal water, which might not be available following an earthquake.

Firewalls depend on the concrete walls and earthen berm that make up the secondary containment at the tank farms.

5.3 Emergency Response Equipment

This section addresses Rule 340-300-0003(6)(c)(C) and Rule 340-300-0003(1)(h).

Automated Sprinkler Systems

The truck loading rack is equipped with a foam sprinkler deluge system. The system includes both roof level foam / water sprinklers and ground level foam / water nozzles. In addition, the P2 Dock is equipped with an automatic dry pipe fire sprinkler system. Finally, Building #10 is provided with an automatic fire sprinkler system.

Firefighting Foam

The truck rack is protected by a foam suppression system. The foam is activated automatically by fire detectors but can also be manually activated by way of manual actuation stations.

The terminal is equipped with a dedicated fire pump and foam distribution system that can provide foam throughout the terminal. Most of the terminal areas have adequate foam to meet NFPA 11 requirements. There is a project underway to bring the rest of the terminal into compliance.

Spill Response Kits

The terminal is equipped with spill response kits strategically located throughout the terminal, which store oil spill absorbent materials and containment equipment.

Power and Communications

The terminal maintains emergency response equipment including handheld radios, a radio base unit, marine radio and a satellite phone for emergency use. Handheld radios can be used to make direct radio-to-radio calls and do not depend on municipal power to operate.

Yard lighting is not protected by emergency backup power and is susceptible to loss of municipal power.

5.3.1 Seismic Vulnerabilities

The firewater system and foam distribution system are dependent on municipal water, which might not be available following an earthquake.

Terminal yard lights depends on municipal power, which may not be available following an earthquake.

5.4 Safety of Operating Conditions

This section addresses Rule 340-300-0003(1)(g).

There are manual block valves isolating tanks and tank farm piping that are normally closed unless there is an active cargo movement operation underway. All tank farm isolation valves are manual and do no depend on municipal power to operate.

The tank truck loading rack is equipped with an automatic emergency shutdown system (ESD) which can be activated automatically by fire detectors or by manual actuation stations.

Both docks are protected by emergency isolation valves. P-2 has a motored operated isolation valve, while P-3 has a manually operated valve for emergency isolation.

5.4.1 Seismic Vulnerabilities

Motorized and automatic emergency shutdown systems depend on municipal power, which may not be available after a large earthquake.

5.5 Terminal Staffing, Monitoring, and Response

This section addresses Rule 340-300-0003(1)(j).

The terminal is manned 24 hours a day, 7 days a week.

5.5.1 Seismic Vulnerabilities

None identified.

6. RISK ASSESSMENT

We used a critical systems risk assessment process to identify, prioritize, and assess the seismic vulnerabilities of critical equipment, structures, and procedures. This analysis considered the performance of critical systems during and after the DLE event, and how their seismic vulnerabilities impact the prevention and containment of oil spills.

This risk assessment was in the form of a workshop including terminal operations and safety specialists, along with structural/seismic engineering specialists who understand the historic seismic performance of systems in earthquakes. With this experience, we can consider realistic damage and failure scenarios rather than assessing strict conformance to current codes for new design. See Appendix C for a list of attendees.

The team considered possible scenarios due to earthquakes that could realistically occur and result in an uncontained spill, uncontrolled fire, explosion, or toxic release at the terminal. The workshop was used to risk rank and prioritize the criticality of various structures and systems during and following a seismic event in terms of the likelihood and consequences of a potential release of fuel from a spill caused by a DLE event.

The risk ranking was done through a risk matrix approach, using the risk matrices shown in Figures 6-1 and 6-2 for Environmental and Life-Safety risks, respectively.

We assigned structures and equipment a Likelihood of damage in a DLE that could lead to a spill, with ratings of 1 to 5 from "Very Unlikely" to "Very Likely", as defined in Appendix C. During the workshop, we assigned a Severity rating from A to E, from the least severe environmental or life-safety consequences to the most severe.

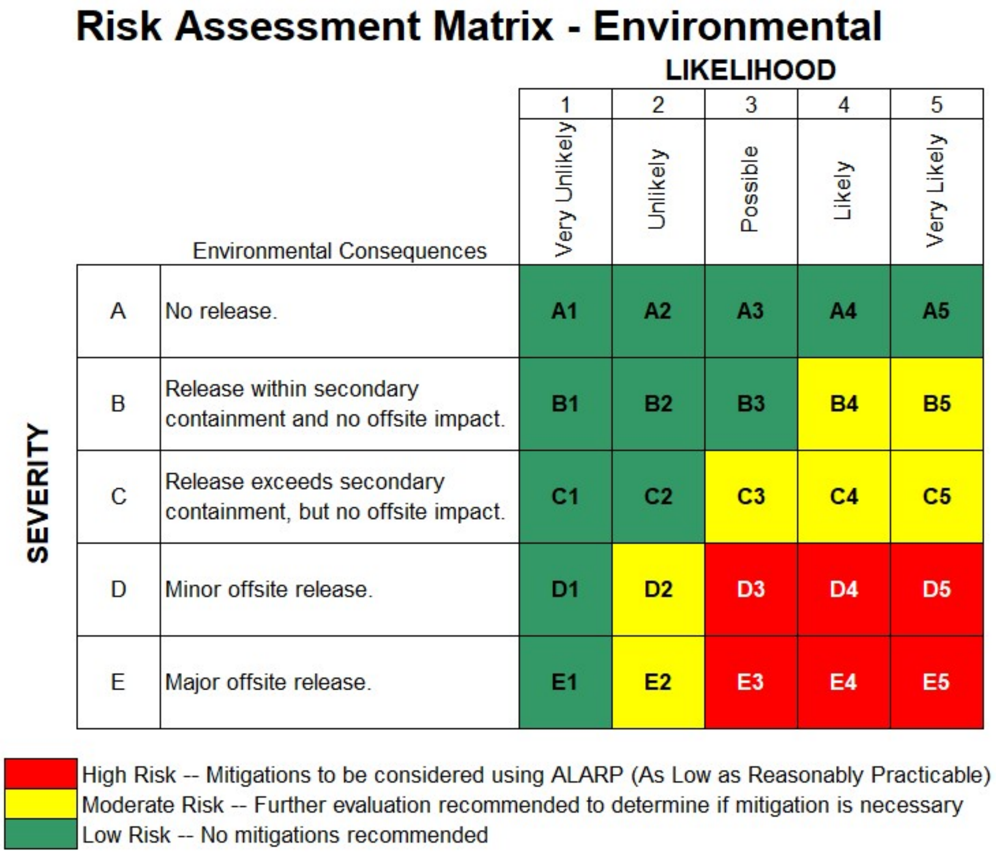
The Severity rating considered potential spill volumes, secondary containment mechanisms, operational or other safeguards that are in place, type of contents (i.e. flammability or combustibility of contents), and criticality of the component in emergency response. The potential impact on public health and safety are also considered within the Life Safety severity, by considering whether the consequences would extend beyond property lines and into publicly accessible areas. For example, the spill of a more volatile substance has a higher Life Safety consequence due to its fire potential. .

We use the Severity and Likelihood to assign each item two risk ranking matrix scores. The environmental score relates to the quantity of spill and its impact on, or extent into, the neighboring community. The life-safety score relates to life-safety consequences that occur directly as a result of the spill.

For most items, the scores are specific to that item (e.g. based on an individual tank's Likelihood of structural failure and Severity of consequences). For secondary containment walls, the score considers all the tanks, piping, and other fuel storage within that area. We considered the magnitude of expected probable volume of spill within the yard and the likelihood of structural failure. We also considered the relative size and location of the tank farm, particularly proximity to the Willamette River in assigned the severity of consequences.

We also assigned two sets of scores, representing vulnerability with and without the considerations of geotechnical soil displacements. This is to inform the terminal of relative risks associated with the global liquefaction and lateral spreading hazard versus those associated with ground shaking.

We provide the complete risk assessment, including a table of all items and resulting risk assessment scores in Appendix C.



Risk Assessment Matrix - Environmental

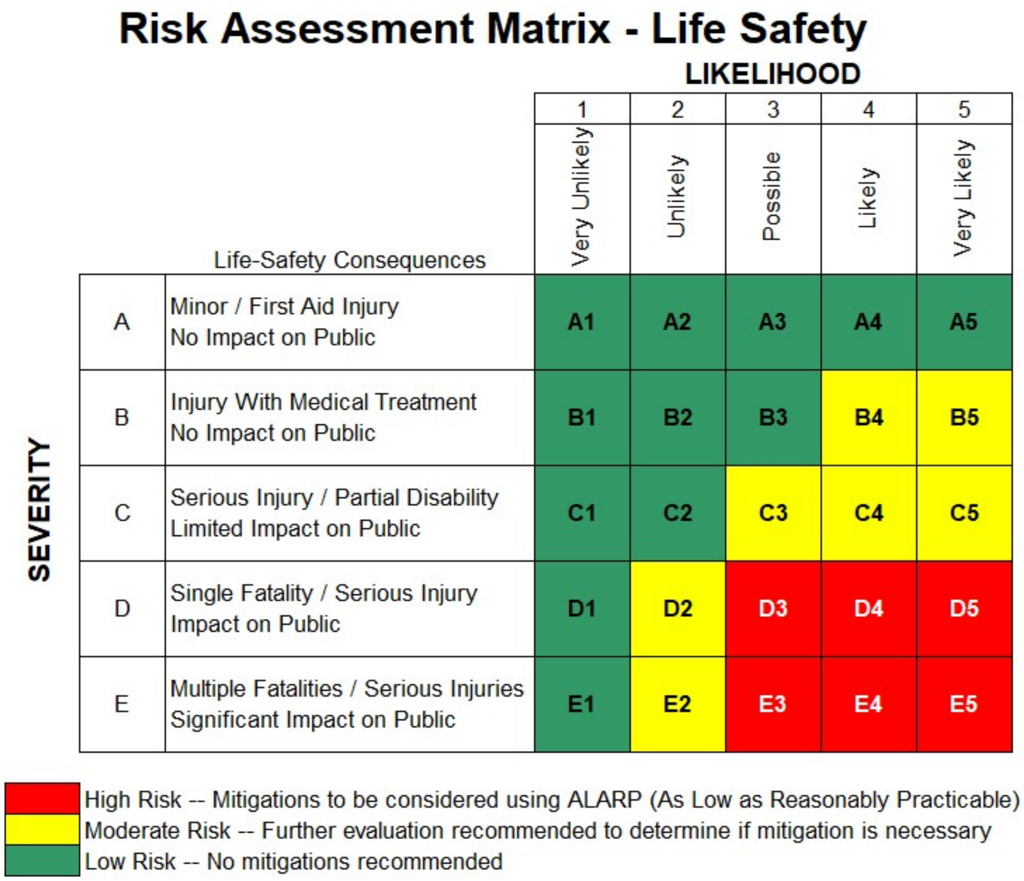
LIKELIHOOD

SEVERITY

	Environmental Consequences	1 Very Unlikely	2 Unlikely	3 Possible	4 Likely	5 Very Likely
A	No release.	🟩 A1	🟩 A2	🟩 A3	🟩 A4	🟩 A5
B	Release within secondary containment and no offsite impact.	🟩 B1	🟩 B2	🟩 B3	🟨 B4	🟨 B5
C	Release exceeds secondary containment, but no offsite impact.	🟩 C1	🟩 C2	🟨 C3	🟨 C4	🟨 C5
D	Minor offsite release.	🟩 D1	🟨 D2	🟥 D3	🟥 D4	🟥 D5
E	Major offsite release.	🟩 E1	🟨 E2	🟥 E3	🟥 E4	🟥 E5

🟥 High Risk -- Mitigations to be considered using ALARP (As Low as Reasonably Practicable)
🟨 Moderate Risk -- Further evaluation recommended to determine if mitigation is necessary
🟩 Low Risk -- No mitigations recommended

Figure 6-1 – Environmental Risk Assessment Matrix



Risk Assessment Matrix - Life Safety

			LIKELIHOOD				
			1	2	3	4	5
		Life-Safety Consequences	Very Unlikely	Unlikely	Possible	Likely	Very Likely
SEVERITY	A	Minor / First Aid Injury No Impact on Public	A1	A2	A3	A4	A5
	B	Injury With Medical Treatment No Impact on Public	B1	B2	B3	B4	B5
	C	Serious Injury / Partial Disability Limited Impact on Public	C1	C2	C3	C4	C5
	D	Single Fatality / Serious Injury Impact on Public	D1	D2	D3	D4	D5
	E	Multiple Fatalities / Serious Injuries Significant Impact on Public	E1	E2	E3	E4	E5

🟥 High Risk -- Mitigations to be considered using ALARP (As Low as Reasonably Practicable)
🟨 Moderate Risk -- Further evaluation recommended to determine if mitigation is necessary
🟩 Low Risk -- No mitigations recommended

Figure 6-2 – Life-Safety Risk Assessment Matrix

7. FINDINGS

Based upon the geotechnical, structural, and safety assessments as described herein, we have identified the key vulnerability findings as summarized below.

Items are categorized as Moderate or High Risk based on the full consideration of hazards, including earthquake induced ground deformations. Although the Likelihood of a spill may increase as a result of ground deformations, severity of consequences are typically the same. Thus, the risk categorization (or color) does not necessarily change due to the addition of ground deformations. Where the with- and without- ground deformation score results in a difference in categorization, the without ground deformation categorization is also indicated.

For High Risk items, mitigations should be considered using As Low As Reasonably Practicable (ALARP) risk reduction philosophy. For Moderate Risk items, further evaluation is recommended to determine if mitigation is necessary. For example, this may include detailed engineering calculations to quantify the seismic capacity of specific, existing components.

Tables 7-1 and 7-2 present only assets that are not exempt by DOT/PHMSA jurisdiction. Categorization of all terminal assets is provided in Appendix C.

Table 7-1 - Summary of High Risk Items

Docks	Safety Systems & Buildings
P-2 Wharf	Water Main
P-2 Piping	Foam System
P-3 Wharf	Fire Pump
P-3 Piping	Hydrants

Table 7-2 - Summary of Moderate Risk Items

Tank Farm 1	Tank Farm 3 & 4	Other Liquids (outside Main Yards)	Docks	Safety Systems & Buildings
Non-DOT/PHMSA Piping[1]	Non-DOT/PHMSA Piping[1]	Oil Water Separator	P-2 Dock Office[2]	#10 Foam House[2]
T-2113	T-1011	Drummed Waste Storage	P-3 Dock Office[2]	#13 Foam House[2]
T-3605	T-3201	EMS Overfill Tank (Truck Loading Rack)[2]		
		TLR Piping[1]		

1. All piping (except at the dock) is Moderate with ground deformations due to Likelihood. Non-flammable product piping is Low Risk without ground displacements. Piping for flammable fuels are Moderate Risk with- or without- ground deformation due to Life Safety Severity.

2. These items are Low Risk without consideration of ground deformation and elevated to Moderate with ground deformation due to increased Likelihood of damage.

7.1 Geotechnical

We have determined a peak ground acceleration (PGA_M) of 0.49g for the ASCE 7-16 DLE event. Median estimates of seismically-induced lateral ground deformations varies from about 2.5 ft near the truck loading rack, to over 5 ft in the middle of the tank farms and east toward the river. Corresponding vertical displacements vary from 12 in. to over 30 in. at the site, with the potential for higher localized settlements.

Our structural and safety assessments considered these potential displacements.

7.2 Structural

Most tanks have a Moderate Risk due to their Likelihood of damage particularly in the soil flow slide zone.

Pipelines are rated Moderate throughout the terminal due to differential displacements from ground deformation and the anticipated pipe stresses. At the dock, pipelines are rated High due to a higher consequence of damage and spill directly into the river.

The secondary containment walls are rated High due to their importance in containing spills and the uncertainty in their capacity to withstand seismic loads due to their age and construction. However, secondary containment walls and berms are under the DOT/PHMSA jurisdiction, so they are not subject to the Rules.

7.3 Safety

The water supply is rated as a High Risk seismic vulnerability. The facility relies on municipal water as its only source for firewater and foam distribution. It is highly unlikely municipal water will be available following the DLE considered by the Rules.

Since the foam system is dependent on municipal water, which is unlikely to be available following the DLE, and the consequence of this system being unavailable, this item is deemed a High Risk.

8. REFERENCES

1. API Standard 570 “Piping Inspection Code: Inspection, Repair, Alteration and Rerating of In-Service Piping Systems, American Petroleum Institute (API),” 5th Edition, 2024.

2. API Standard 650 ”Welded Tanks for Oil Storage, American Petroleum Institute (API),” 13th Edition, 2020.

3. API Standard 653 “Tank Inspection, Repair, Alteration and Reconstruction, American Petroleum Institute (API),” 5th Edition, 2020.

4. ASCE 7-16 “Minimum Design Loads and Associated Criteria for Buildings and Other Structures, American Society of Civil Engineers (ASCE), Structural Engineering Institute (SEI),” 2016.

5. ASCE 41-17 “Seismic Evaluation and Retrofit of Existing Buildings, American Society of Civil Engineers (ASCE), Structural Engineering Institute (SEI),” 2017.

6. ASCE 61-14 “Seismic Design of Piers and Wharves, American Society of Civil Engineers (ASCE), Coasts, Oceans, Ports & Rivers Institute, (COPRI),” 2014.

7. California Building Code (CBC) Chapter 31F, “Marine Oil Terminal Engineering and Maintenance Standards” (MOTEMS), 2022.

8. “Fuel Tank Stability Rules,” Division 300, State of Oregon Department of Environmental Quality (DEQ), 2023.

9. "Guidance for California Accidental Release Prevention (CalARP) Program Seismic Assessments." Prepared for the Unified Program Agency (UPA) Subcommittee of the Region I Local Emergency Planning Committee (LEPC) by the CalARP Program Seismic Guidance Committee, January 2019.

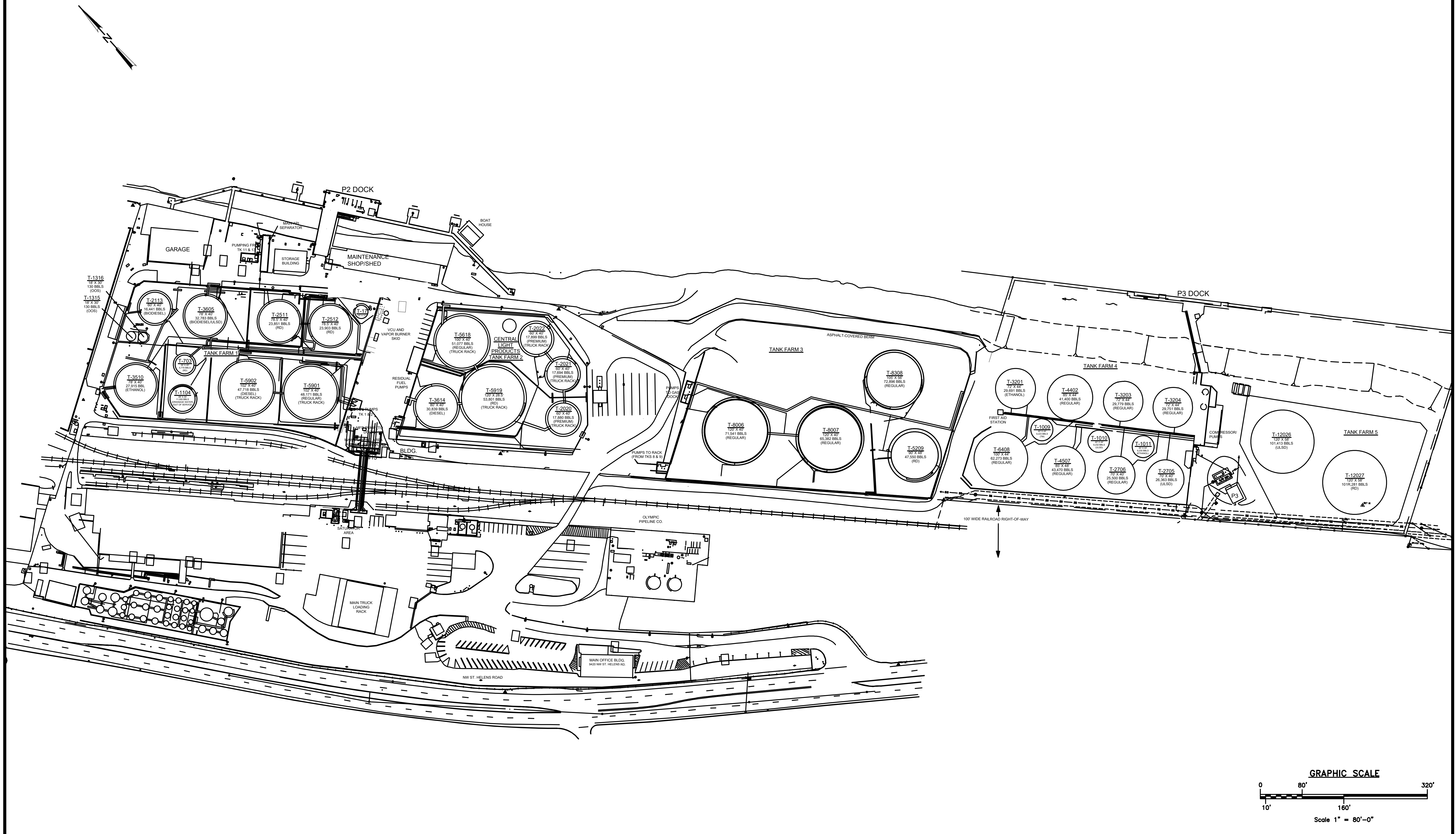
10. “Guidelines for Seismic Evaluation and Design of Petrochemical Facilities,” Task Committee on Seismic Evaluation and Design of Petrochemical Facilities, American Society of Civil Engineers (ASCE), 1997, 2nd Edition, 2011, and 3rd Edition, 2020.

11. STI Standard SP001 “Standard for the Inspection of Aboveground Storage Tanks, Steel Tank Institute (STI),” 6th Edition, 2018.

12. Zhang, G., Robertson, P.K., and Brachman, R.W.I., 2004, Estimating Liquefaction-Induced Lateral Displacements Using the Standard Penetration Test or Cone Penetration Test, Journal of Geotechnical and Geoenvironmental Engineering, Volume 130, No. 8, 2004.

Appendix A

Site Plan & Inventory



NOTES:
1) CONTRACTOR TO FIELD VERIFY ALL DIMENSIONS AND ELEVATIONS PRIOR TO FABRICATION AND INSTALLATION.

REFERENCE DRAWINGS:

NO.	REVISION	BY	DATE	APR	
1	REVISED TANK DESCRIPTIONS	JAV	5/24/24	TF	JAV
0	UPDATED PER PROJECT	KLV	09/27/23	NSD	JAV

NuStar
Energy L.P.

19003 IH-10 WEST
San Antonio, Texas 78257
Tel: (210) 918-2000
1-800-866-9060

PROJECT LOCATION: Portland, Oregon	
DRAWN BY: KLV	**DATE:** 09/27/23
CHECKED:	**DATE:**
APPROVED:	**DATE:**
SCALE: 1" = 80'-0"	

PORTLAND TERMINAL PLOT PLAN

ORIGINAL PROJECT NO.

DRAWING NO. PORT-21-001 **REV.** 1

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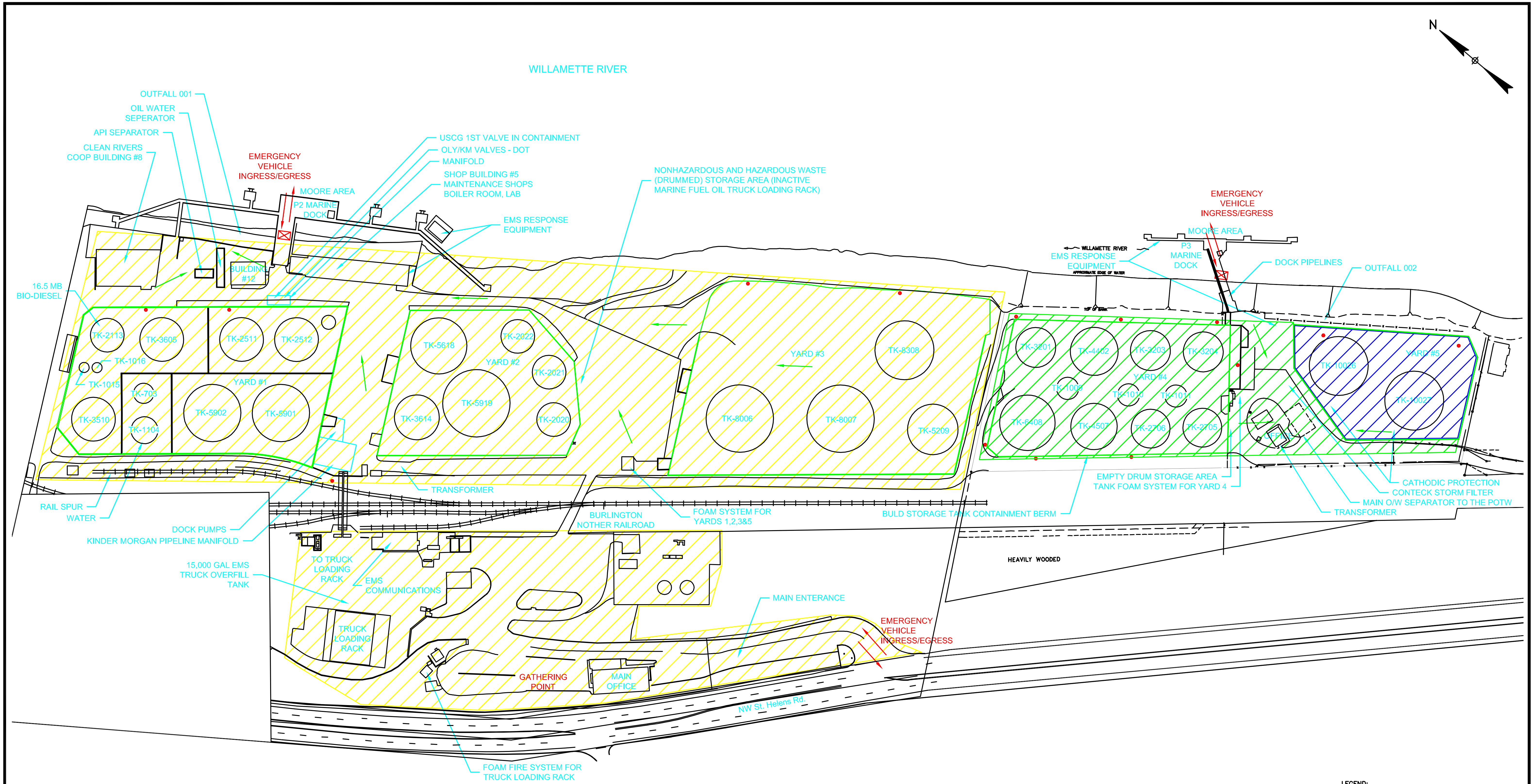


LEGEND:

—	CONTAINMENT BERM/WALL
●	FIRE HYDRANT OR WATER SYSTEM
☒	EMERGENCY SHUT-OFF VALVE
←	DIRECTION OF DISCHARGE
▨	YARD 5 TO CONTECH
▨	P2 DRAINAGE AREA
▨	P3 DRAINAGE AREA

NOTES:
1) CONTRACTOR TO FIELD VERIFY ALL DIMENSIONS AND ELEVATIONS PRIOR TO FABRICATION AND INSTALLATION.

REFERENCE DRAWINGS:

NO.	REVISION	BY	DATE		APR
0	ISSUED	BJZ	06/07/16	CC	CC

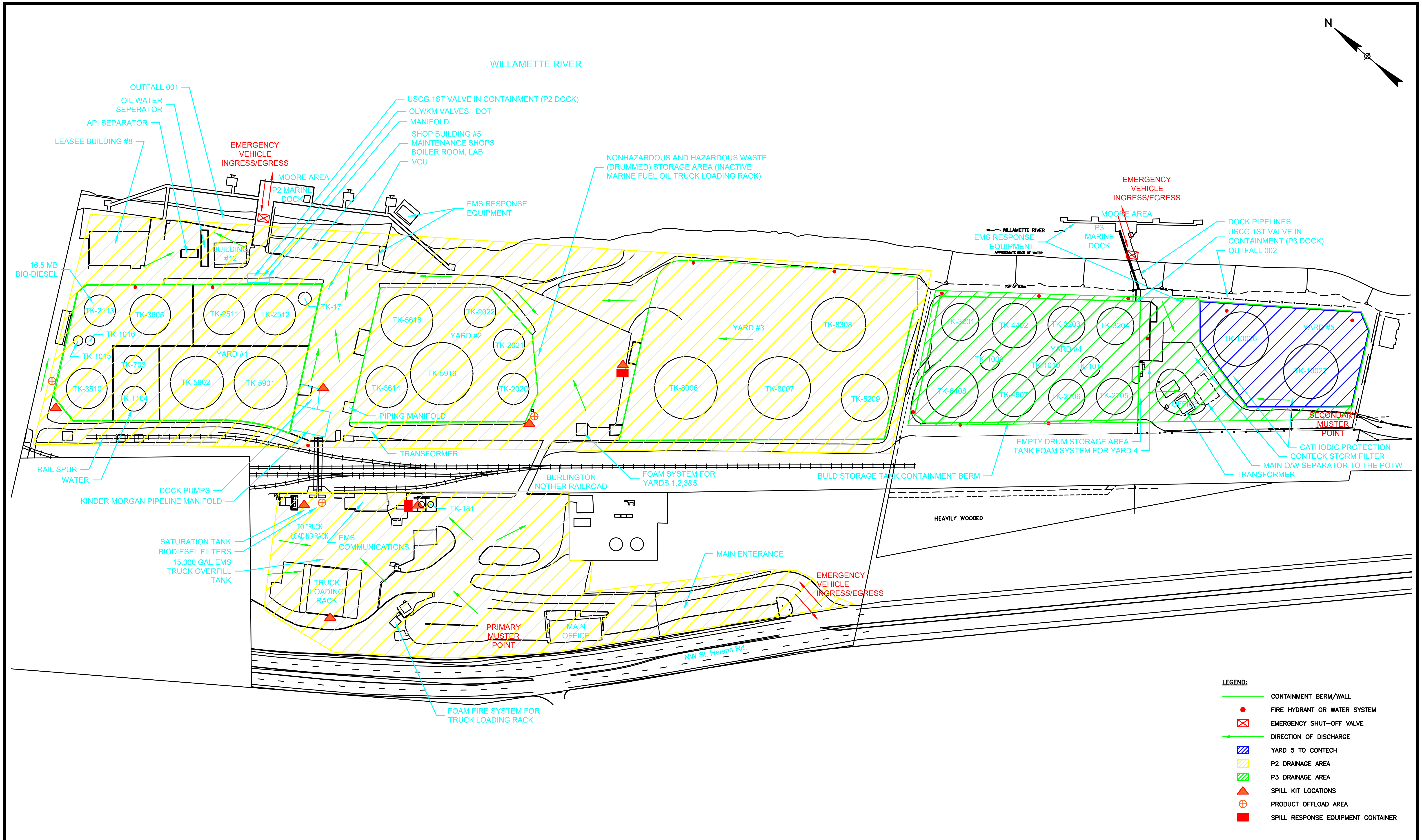
NuStar Energy L.P.

19003 IH-10 West
San Antonio, Texas 78257
Tel: (210) 918-2000
1-800-866-9060

PROJECT LOCATION: Portland, OR
DRAWN BY: B. ZEPPIERI DATE: 06/07/16
CHECKED: C. CHAN DATE: 06/07/16
APPROVED: C. CHAN DATE: 06/07/16
SCALE: NONE

PORTLAND TERMINAL SITE DRAINAGE PLAN

ORIGINAL PROJECT NO.
DRAWING NO. PORT-09-004
REV. 0



NOTES:
1) CONTRACTOR TO FIELD VERIFY ALL DIMENSIONS AND ELEVATIONS PRIOR TO FABRICATION AND INSTALLATION.

REFERENCE DRAWINGS:

NO.	REVISION	BY	DATE	APR	
2	UPDATED CALLOUTS	BJZ	07/27/22		
1	UPDATED VALVE CONTAINMENT P2/P3 DOCK	BJZ	05/28/19	CC	CC
0	ISSUED	BJZ	06/07/16	CC	CC

NuStar
Energy L.P.

19003 IH-10 West
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PROJECT LOCATION: Portland, OR
DRAWN BY: B. ZEPPIERI DATE: 06/07/16
CHECKED: C. CHAN DATE: 06/07/16
APPROVED: C. CHAN DATE: 06/07/16
SCALE: NONE

PORTLAND TERMINAL
HS&E (HEALTH, SAFETY & ENVIRONMENT)
SITE DRAINAGE PLAN

ORIGINAL PROJECT NO.
DRAWING NO. PORT-09-004
REV. 2

LEGEND

—	CONTAINMENT BERM/WALL
—	EVACUATION ROUTE
●	FIRE HYDRANT OR WATER SYSTEM
⊠	EMERGENCY SHUT-OFF VALVE

N

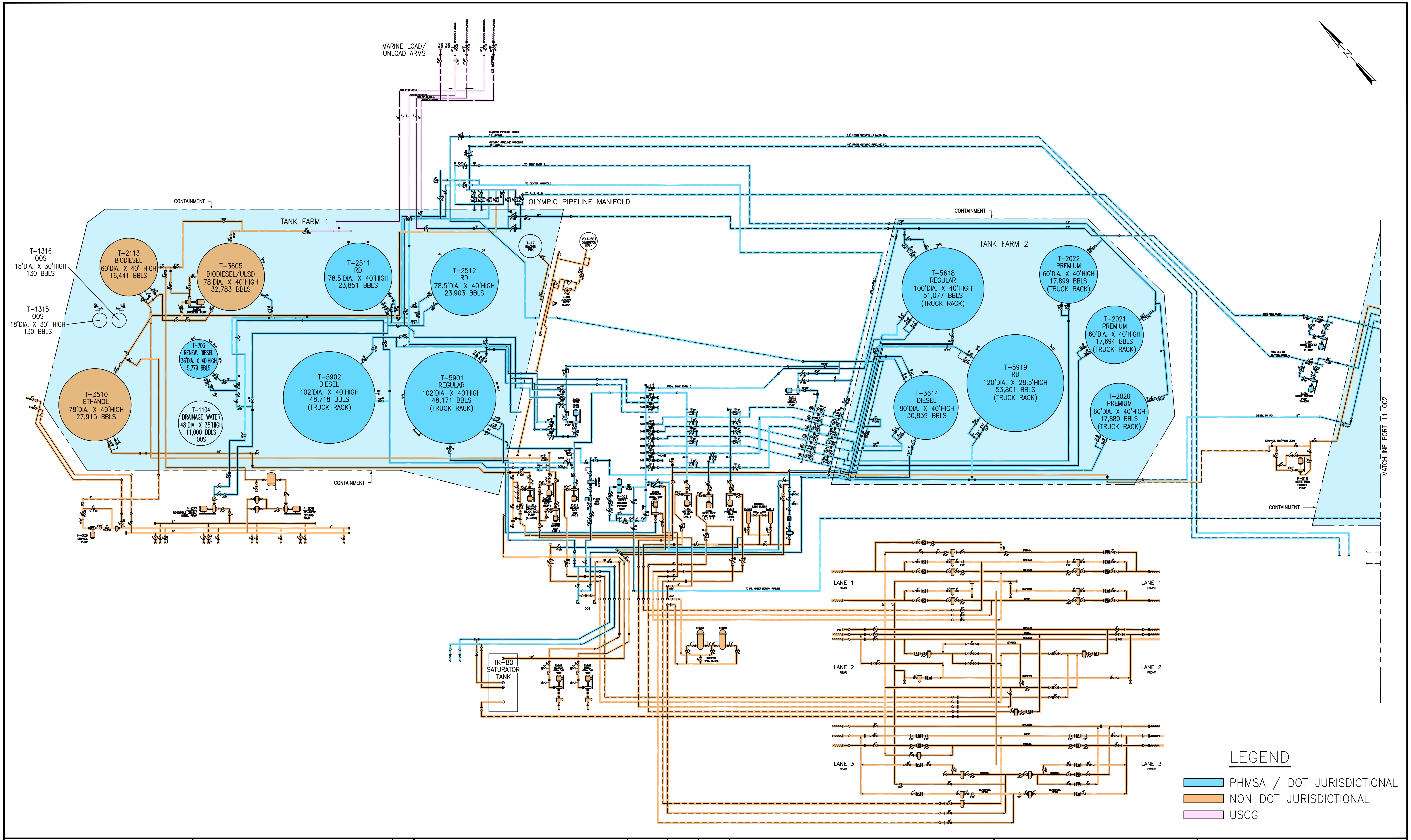
NOT TO SCALE

Site Evacuation Plan

Spill Prevention Control and Countermeasure Plan
Shore Terminals, LLC Portland Facility
Portland, Oregon

Cascadia
Associates, LLC

Project Number	0060-003-002	Figure
August 2019		C-8



LEGEND

🟦 PHMSA / DOT JURISDICTIONAL

🟧 NON DOT JURISDICTIONAL

🟪 USCG

NOTES:

REFERENCE DRAWINGS:
PORT-21-007

NO.	REVISION	BY	DATE	APR	
11	AS-BUILT	SLC	08/02/19	APR	RLM
10	UPDATED PER FIELD PROJECTS	BJZ	02/15/18	KP	
14	UPDATED PUMP NUMBERS	BJZ	08/17/22	KV	
13	MOVED SPEC BLIND AT T-2113 AREA	BJZ	07/19/22	KP	JV
12	UPDATED PER VCU PROJECT	BJZ	04/06/22	KV	

NuStar
Energy L.P.

19003 IH-10 WEST
San Antonio, Texas 78257
Tel: (210) 918-2000
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PROJECT LOCATION: 9420 NW ST. HELENS RD., PORTLAND, OR 97231

DRAWN BY:	WHR	**DATE:**	3/22/10
CHECKED:	KDL	**DATE:**	4/9/10
APPROVED:		**DATE:**	
SCALE:	NOT TO SCALE		

PORTLAND TERMINAL
PROCESS FLOW DIAGRAM
1 OF 2

ORIGINAL PROJECT NO.

DRAWING NO. PORT-11-001

REV. 14

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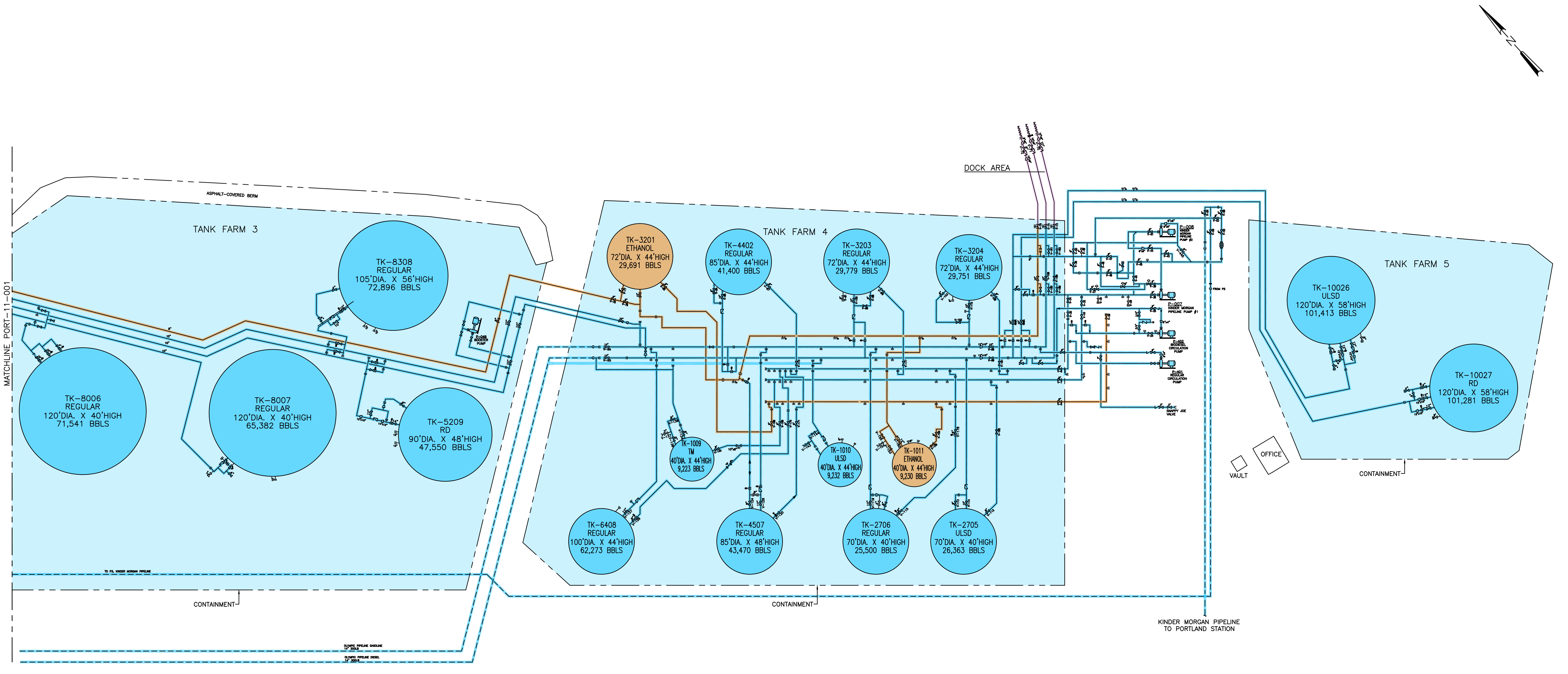

LEGEND

- ▭ PHMSA / DOT JURISDICTIONAL
- ▭ NON DOT JURISDICTIONAL
- ▭ USCG

NOTES:

REFERENCE DRAWINGS:
PORT-21-007

NO.	REVISION	BY	DATE	APR	
14	UPDATED PER FIELD WALKDOWN	BJZ	08/30/18	KP	
18	UPDATED PUMP NUMBERS	BJZ	08/17/22	KV	
17	AS-BUILT	SLC	08/02/19	SLC	RLM
16	UPDATED P3 VALVES MOVED INTO CONTAINMENT	BJZ	12/05/18	KP	DJ
15	UPDATED PER FIELD MARK-UP ADDED VALVE, REMOVED OOS PIPING	BJZ	11/19/18	KP	KP

NuStar
Energy L.P.

19003 IH-10 WEST
San Antonio, Texas 78257
Tel: (210) 918-2000
1-800-866-9060

PROJECT LOCATION: 9420 NW ST. HELENS RD., PORTLAND, OR 97231
DRAWN BY: WHR **DATE:** 3/22/10
CHECKED: KDL **DATE:** 4/9/10
APPROVED: **DATE:**
SCALE: NOT TO SCALE

PORTLAND TERMINAL PROCESS FLOW DIAGRAM
2 OF 2

ORIGINAL PROJECT NO.
DRAWING NO. PORT-11-002
REV. 18

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Appendix B

Geotechnical Assessment

155 Grand Avenue
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gannettfleming.com

Revised May 30, 2024

SGH Project No. 237367.00-NPSV / Gannett Fleming Project No. 077741

Julie A. Galbraith
Senior Project Manager
Simpson Gumpertz & Heger Inc.
1999 Harrison Street, Suite 2400
Oakland, CA 94612

Re: **Preliminary Geotechnical Report**
Shore Terminals Portland Terminal – Seismic Vulnerability Assessment
Portland, Oregon

Dear Ms. Galbraith:

At your request, Gannett Fleming, Inc. (Gannett Fleming) has prepared this report summarizing our preliminary geotechnical investigation in support of the Seismic Vulnerability Assessment of the Shore Terminals LLC (Shore Terminals) Portland Terminal located at 9420 NW St. Helens Road in Portland, Oregon. We performed our assessment in general accordance with the scope of services per our agreement with Simpson Gumpertz & Heger Inc. (SGH) dated January 26, 2024. The following provides a summary of our findings, analysis results, and conclusions.

PROJECT BACKGROUND

The primary improvements at the terminal consist of 31 storage tanks, secondary containment structures, two piers, product transfer pipelines, a truck loading rack, and associated facilities. Several previous geotechnical investigations and assessments have been completed for the site. A Seismic Vulnerability Assessment of the terminal will be required in accordance with the State of Oregon Department of Environmental Quality (DEQ) Division 300 Fuel Tank Seismic Stability Rules, Oregon Administrative Rules 340-300-0000 (Rules). The Rules require a Seismic Vulnerability Assessment be performed to evaluate the risk of seismically-induced impacts including liquefaction, settlement, lateral spreading, and ground failures. The objective of such an assessment is to identify any risk mitigation measures that may be necessary. SGH is leading the Seismic Vulnerability Assessment with geotechnical input provided by Gannett Fleming.

OBJECTIVE AND SCOPE OF SERVICES

The purpose of our geotechnical assessment is to provide input in support of the Seismic Vulnerability Assessment. In accordance with our agreement with SGH dated January 26, 2024, our assessment considers data from our geotechnical investigation, existing site-specific geotechnical information, and other existing data. The scope of our services included the following.

- Review of existing information and subsurface characterization considering geotechnical data for the site.
- Field exploration comprised of three Seismic Cone Penetration Tests (SCPTs).
- Characterization of static soil parameters and cyclic/post-cyclic behavior of each significant geologic stratum.
- Preliminary seismic hazards evaluation including the following.
  - Development of design earthquake ground motion values for geotechnical analysis.
  - Performance of liquefaction triggering/cyclic degradation analysis based on CPT data and data specific to Willamette River Silt.
  - Performance of decoupled analyses to evaluate mechanisms potentially contributing to ground surface vertical and lateral deformations using simplified methods and post-liquefaction slope stability analysis.
- Development of seismically-induced lateral and vertical ground deformation estimates based on the results of the preliminary seismic hazards evaluation.
- Qualitative evaluation of the potential effects of ground deformations on fuel storage tanks, piers, and associated facilities.
- Consultation with SGH regarding geotechnical input to support structural evaluations.
- Preparation of this report.

SITE CONDITIONS

The terminal is located on the east side of NW St. Helens Road just east of the foothills of the Tualatin Mountains and west of the Willamette River as shown in Figure 1. Facilities at the terminal are comprised of five tank yards (Tank Yards 1 through 5) with 31 steel liquid products storage tanks about 40 to 120 feet in diameter and secondary containment walls (including a containment berm on the east side of Tank Yard 3), two piers (Pier P2 and Pier P3), a three-bay truck loading rack, a five-bay rail siding, pumps, pipelines, and associated facilities. An aerial image of the terminal is presented in Figure 2. Liquid products are transferred out of and into the terminal via several modes including the rail siding, truck loading rack, and piers. The rail lines bisect the site and are aligned roughly parallel to the shoreline. Data from a bathymetric survey completed by AKS Engineering & Forestry, LLC (AKS) at the site in 2023 indicate the waterfront slope is roughly 70 feet high (AKS, 2023). The tank yards are located between the waterfront and existing rail lines, with a ground surface at roughly elevation 35 feet (NAVD88). The ground surface west of the tank yards slopes up gently to about elevation 45 feet at the location of the truck loading rack, with higher ground and steeper slopes near the truck loading rack on the west boundary of the site adjacent to NW St. Helens Road.

A bulkhead wall and concrete revetment were previously constructed along a portion of the waterfront slope from the north boundary of the terminal to roughly 600 feet south of this location. The bulkhead wall is about 8 ½ feet tall and founded on a shallow footing as indicated on a drawing prepared by General Petroleum Corporation (GPC) dated 1943 (GPC, 1943). The wall supports fill placed to facilitate construction of adjacent buildings and infrastructure adjacent to Pier P2. In addition, concrete revetment was constructed on the waterfront slope up to roughly 7 feet (measured vertically) below the toe of the wall prior to the retaining wall construction as indicated on drawings prepared by GPC dated 1935 (GPC, 1935) and 1943 (GPC, 1943).

In addition, a subsurface slurry cutoff wall was previously constructed parallel to the shoreline near the top of the waterfront slope adjacent to Tank Yard 2 and Tank Yard 3 in support of groundwater remediation. Slurry wall details are shown on drawings prepared by Sverdrup & Parcel and Associates, Inc. (SPA) dated 1981 (SPA, 1981). Further, we understand that ground improvement elements have been installed at Tank Yard 5 to address seismic stability impacts on the tanks and facilities at this location. Drawings or as-built plans for the ground improvement are not available.

EXISTING DATA

A previous geotechnical investigation was performed at the site in 2019 as described in a report by Landau Associates, Inc. (Landau) dated July 22, 2019 (Landau, 2019). The investigation included Cone Penetration Tests (CPTs) at three locations, two test pits, and one hand boring. Shear wave velocity measurements were collected in one of the CPTs. The hand boring was advanced to a depth of about 6 feet and test pits were excavated to depths of about 2 to 4 ½ feet.

CH2M HILL, Inc. (CH2M Hill) completed a geotechnical investigation at the site in 2006 as summarized in a Geotechnical Data Report dated June 2006 (CH2M Hill, 2006). The investigation included two CPTs and five soil borings. The CPTs were advanced with a truck-mounted rig to depths of about 26 feet (shallow refusal) to 43 ½ feet. The soil borings were drilled using mud rotary methods to depths of about 30 ½ to 60 feet. Soil samples from the borings were primarily collected using a Standard Penetration Test (SPT) sampler advanced under the impact of an automatic 140-pound hammer free-falling 30 inches. Soil samples were also collected using Shelby tubes, with rock samples collected using an NX-sized core drill bit.

The approximate locations of the previous explorations are shown in Figure 2. Data from these previous geotechnical investigations including exploration logs and laboratory test results presented in Appendix A were considered as part of our geotechnical assessment.

FIELD EXPLORATION

To supplement the existing data, we performed a field exploration included three Seismic CPTs (SCPTs). The SCPTs were located at Tank Yard 3, between Tank Yard 2 and Tank Yard 3, and southeast of the truck loading rack at the approximate locations shown in Figure 2. We provide a summary of the field exploration below.

Cone Penetration Tests

Three SCPTs were performed by ConeTec, Inc. on February 22, 2024. The SCPTs were advanced to refusal encountered at depths of about 20 to 50 feet. Considering the depth to bedrock encountered in previous explorations at the site, we anticipate the SCPTs encountered refusal at or near the top of bedrock. The SCPT soundings were performed using an electronic cone penetrometer in general accordance with the current ASTM Standards (ASTM D5778 and ASTM D7400). The SCPT equipment consisted of a cone penetrometer assembly mounted at the end of a series of hollow sounding rods. The cone penetrometer assembly consisted of a conical tip with a 60-degree apex angle and a projected cross-sectional area of 2.33 in^2 (15 cm^2) and a cylindrical friction sleeve with a surface area of 34.88 in^2 (225 cm^2). The interior of the cone penetrometer is instrumented with strain gauges that allow simultaneous measurements of cone tip and friction sleeve resistance during penetration. The cone penetrometer assembly is continuously pushed into the soil by a set of hydraulic rams at a standard rate of 0.79 inch per second (2 cm per second) while the cone tip resistance and sleeve friction resistance are recorded at fixed depth increments, generally every 1 inch (2.5 cm) and stored in digital form. A specially designed 30-ton truck provides the required reaction weight for pushing the cone assembly and is also used to transport and house the test equipment. The computer-generated graphical logs include cone resistance, friction resistance, and friction ratio. Soil behavior type interpretations are based on guidelines by Robertson (2009 and 2010). The CPT data are included in Appendix B.

Seismic shear wave tests were performed by stopping the penetration of the cone and the rods and decoupling them from the rig. A sledge hammer is used to manually trigger a shear wave into the soil. The distance from the source to the cone is calculated based on the total depth of the cone and the horizontal offset distance between the source and the cone. An interval velocity is calculated using a minimum of two tests performed at two different depths. The shear wave velocity data are included in Appendix B.

SUBSURFACE CONDITIONS

The site is underlain by various amounts of fill materials placed during site development. Regional geologic mapping indicates the fill is underlain by young Quaternary alluvium comprised of river and stream deposits of silt, sand, clay, and peat of present floodplains (Schlicker, H.G., et al., 1967). The alluvium is largely confined to the ancient incised Willamette River channel, which includes the current channel and the adjacent floodplains. The mapping suggests the alluvium is underlain by Columbia River basalt at depth.

The previous and current exploration indicate subsurface conditions encountered that are generally consistent with site development and regional geology. Subsurface soils are primarily comprised of fill, alluvial deposits, and bedrock. The fill encountered at the site varies in thickness from about 9 to 23 feet, with greater thickness adjacent to the shoreline and decreased thickness on the west side of the site. The fill primarily consists of loose to dense sands with varying amounts of gravel and silt. Alluvial deposits underlying the fill are comprised of fine-grained and coarse-grained soils. The fine-grained alluvium encountered is up to about 30 feet thick and generally consists of soft to very stiff silts and lean clays deposited by successive historic flood events. Coarse-grained alluvium underlying the fine-grained alluvial deposits primarily consist of loose to dense sands interlayered with gravels. The coarse-grained alluvial deposits are up to about 8 feet thick and are underlain by bedrock (Columbia River basalt). Shallow bedrock was encountered in the west portion of the site near NW St. Helens Road at a depth of about 2 to 6 feet southwest of the truck loading rack, with deeper bedrock up to about 64 feet encountered adjacent to the shoreline.

In addition to the primary strata described previously, the 2006 CH2MHill Geotechnical Data Report (CH2M Hill, 2006) indicates silt with varying amounts of gravel encountered at Borings B-2 (depth of about 20 feet), B-3 (depth of about 15 feet), and B-4 (depth of about 25 feet) appears to be colluvium or ancient landslide debris from the hillside west of the site.

Groundwater

Pore pressure dissipation tests (PPDTs) performed during SCPTs completed as part of the current investigation indicate groundwater depths of about 3 ½ to 13 ½ feet, with shallower groundwater encountered on the west side of the site. PPDTs performed during the 2019 Landau investigation indicate groundwater depths of about 10 to 18 ½ feet, with the shallower groundwater apparently influenced by the slurry cutoff wall near the shoreline. Groundwater levels were not measured during the 2006 investigation by CH2M Hill. Fluctuations in groundwater levels likely occur due to variations in the Willamette River water level, rainfall, underground drainage patterns, regional influence, and other factors.

SEISMIC HAZARDS ASSESSMENT

We have evaluated seismic hazards including liquefaction, lateral spreading, and seismic densification. As part of this, we have developed design earthquake ground motions for the purposes of our assessment. A summary of design earthquake ground motions and our conclusions regarding the potential for liquefaction, lateral spreading, and seismic densification is provided below.

Design Earthquake Ground Motions

We developed seismic design parameters in accordance with the 2016 American Society of Civil Engineers (ASCE) Standard 7-16 (ASCE 7-16): Minimum Design Loads for Buildings and Other Structures (ASCE, 2016) for the purposes of evaluating liquefaction potential and lateral spreading. Considering the existing geotechnical data and depth to bedrock, the site can be characterized as Site Class C or D. Using the ASCE 7 Hazard Tool, we calculated a maximum considered earthquake geometric mean (MCE_G) peak ground acceleration adjusted for site class (PGA_M) of 0.49g, corresponding to a moment magnitude (M_w) of 9.3 on the Cascadia Megathrust fault, which governs the seismic hazard at the site. Note that this dominant magnitude is slightly more conservative than the M_w 9.0 scenario noted in Chapter 99 of the Oregon Laws; however, we expect the difference in results of our liquefaction and lateral spreading assessment to not vary significantly given the high magnitude of either event. It should also be noted that consideration of a Site Class C or D results in the same PGA_M for this site.

Liquefaction

Using the empirical procedure developed by Boulanger and Idriss (2014), we evaluated the potential for saturated soil deposits to undergo liquefaction or cyclic softening, which are referred to herein as liquefaction. We primarily considered the CPT data presented in Appendix B. Our analysis accounts for the liquefaction potential of sands and post-cyclic behavior of silt-rich soil with consideration to data from published studies of Willamette River Silt (Dickenson, et al., 2022) as well as the potential for seismic densification (seismic settlement of sands above the groundwater table). We considered a PGA_M of 0.49g and a moment magnitude (M_w) of 9.3.

The results of our evaluation indicate the potential for liquefaction is high considering the design earthquake. Excess pore-water pressures generated during liquefaction will cause ground settlement as the pore pressures dissipate within saturated soils (referred to as reconsolidation). In addition, excess pore pressures will result in strength loss, which can lead to lateral spreading and other effects such as floatation of underground structures. The primary mechanisms of seismically-induced ground settlement are reconsolidation (seismic settlement of soils below the groundwater table), ejecta-induced, and shear-induced deformation. In addition, sands above the groundwater table can undergo seismic densification resulting in ground settlement. We summarize our assessment of seismic densification and the effects of liquefaction and cyclic degradation including ground settlement and floatation of underground structures below, which is followed by our evaluation of lateral spreading in a subsequent section of this report.

Seismic Densification and Reconsolidation Settlement

Considering the generally shallow groundwater conditions at the site, the risk of seismically-induced settlement resulting from the densification of sands above the groundwater table is low. However, a considerable amount of liquefaction-induced settlement from reconsolidation can occur. The seismically-induced ground deformations summarized in a subsequent section of this report are based on the approach developed by Robertson and Lisheng (2010) and Zhang, et al. (2002).

Ejecta-Induced Settlement

Based on our evaluation of the potential for surface effects, we conclude there is a high likelihood of ground surface disruption following liquefaction given the relatively thin non-liquefiable soil (crust) overlying relatively thick liquefiable soil. Surface effects can occur as water is forced to the ground surface when the dissipation of excess pore-water pressures in the liquefied soil exceeds the resistance of the overlying non-liquefiable crust. This can lead to sediment ejecta and settlement from ground loss as the expelled pore-water carries sand particles to the ground surface through volcano-like vents (referred to as sand boils). Ground surface disruption associated with lateral spreading tends to increase the likelihood of sediment ejecta. Our assessment of ejecta-induced settlement considers a review of case histories, such as those summarized by Mijic, et al. (2002), and professional experience including post-earthquake observations.

Shear-Induced Settlement

In addition to settlement from reconsolidation and sediment ejecta, liquefaction-induced foundation settlement can occur when shear-induced deformations driven by cyclic loading occur due to ratcheting and bearing capacity types of movement caused by soil structure interaction (SSI). The amount of foundation settlement in response to the design earthquake depends on the seismic bearing pressures imposed by the structure, foundation dimensions, and liquefied soil strengths. We anticipate settlement would be most significant where the thickness of non-liquefiable crust beneath the foundation is the lowest. While shear-induced foundation settlement is difficult to predict and would need to be evaluated on a case-by-case basis, we expect that up to about 1 foot or more of shear-induced foundation settlement could occur.

Floatation of Underground Structures

Underground structures including underground tanks, vaults, and manholes may be susceptible to floatation due to liquefaction. This can occur as the soil liquefies and loses shear resistance against the uplift force from the buoyancy of the underground structure. The magnitude of uplift displacement depends on the depth of the structure as well as the duration and intensity of earthquake ground motions and is difficult to predict. This would need to be further evaluated for specific underground structures if needed.

Lateral Spreading

Lateral spreading is a phenomenon where a soil mass moves laterally on liquefied soil down a gentle slope or toward a free face, such as the adjacent Willamette River channel, due to reduced soil strengths and earthquake-induced forces acting on soils within and above the liquefied layer (seismic inertial loading). The magnitudes of lateral displacement are expected to be significant near the Willamette River shoreline, reducing in magnitude with increasing distance from the waterfront slope. We summarize our assessment of lateral spreading considering seismic slope stability analysis as well as simplified empirical and semiempirical approaches below.

Seismic Slope Stability

We performed slope stability analysis using the software program SLOPE/W, which is a two-dimensional, plane-strain, limit equilibrium analytical tool. To evaluate seismic stability, we used the method of slices developed by Spencer (1967). The analysis considers a two-dimensional idealized cross section, the location of which is shown in Figure 3, and fully-liquefied undrained residual strengths for the potentially liquefiable soils. In addition, we considered a river water level of 14 feet (NAVD88), which represents an average water level considering a historic ordinary low water elevation of 8 feet (NAVD88) and ordinary high water elevation of 20 feet (NAVD88). Further, considering loads from tanks in portions of the site, we considered a surcharge pressure of 2,000 pounds per square foot to represent tank loading in our analysis. As indicated in the stability analysis results presented in Appendix C of this report, post-earthquake static stability safety factors are 1.0 or less for portions of the site within about 210 feet (without tank surcharge) to 230 feet (with tank surcharge) of the top of the waterfront slope. These results suggest unlimited shear strains may develop within this zone, leading to a flow slide or state of unlimited flow (instability), with deformations likely exceeding 5 feet during the design earthquake event. This flow slide zone is depicted in Figure 3 and excludes the tanks at Tank Yard 5 as these are supported on ground improvement, which have an unknown effect on deformations at this time. To assess lateral spreading in areas landward of the flow slide zone, we considered two simplified approaches as summarized below.

Empirical Evaluation

To estimate liquefaction-induced lateral displacements, we used an empirical approach developed by Youd, et al. (2002). The approach uses empirical equations developed from the multilinear regression of a large case history database for the prediction of lateral spread displacement. The inputs include earthquake magnitude and distance; the cumulative thickness, average mean grain size, and average fines content of liquefiable soil layers; and parameters characterizing ground geometry including level ground with a free face. It should be noted that the case history database considered for this approach is largely comprised of earthquakes with magnitudes between 6 and 8, with extrapolation to higher magnitudes resulting in considerable uncertainty given the sparsity of data in this range (Youd, et al., 2002). Accordingly, we considered a magnitude 8 for the purposes of this assessment, which we judge reasonably represents the seismic hazard at the site. Our preliminary estimates of seismically-induced ground deformations associated with lateral spreading are discussed further below.

Semiempirical Evaluation

To supplement our empirically-based estimates, we also estimated liquefaction-induced lateral displacements using a semiempirical approach developed by Zhang, et al. (2004). The approach uses SPT- and CPT-based methods to evaluate liquefaction potential to estimate potential maximum cyclic shear strains for saturated soils under seismic loading. A lateral displacement index is obtained by integrating the maximum cyclic shear strains with depth considering empirical correlations from case history data developed relating actual lateral displacement, lateral displacement index, and parameters characterizing ground geometry including level ground with a free face (Zhang, et al., 2004). We used this approach to obtain preliminary estimates of seismically-induced ground deformations associated with lateral spreading, which is discussed further below.

Lateral Spread Characteristics

During lateral spreading, surface layers commonly break into large blocks, which progressively migrate toward a free face as depicted in Exhibit 1 below. Lateral spreading creates a zone of extension near the head of the spread, which can result in large open ground fissures, with compressional features occurring near the toe. Zones of compression are usually expressed as buckled soil, pavements, or structures. Accordingly, the ground can break into discrete blocks that will move horizontally relative to each other, with the potential for some blocks overriding each other, resulting in heave or settlement. In addition, the development of ground fissures can promote ground loss from sediment ejecta and increase the likelihood of surface effects and associated settlement.

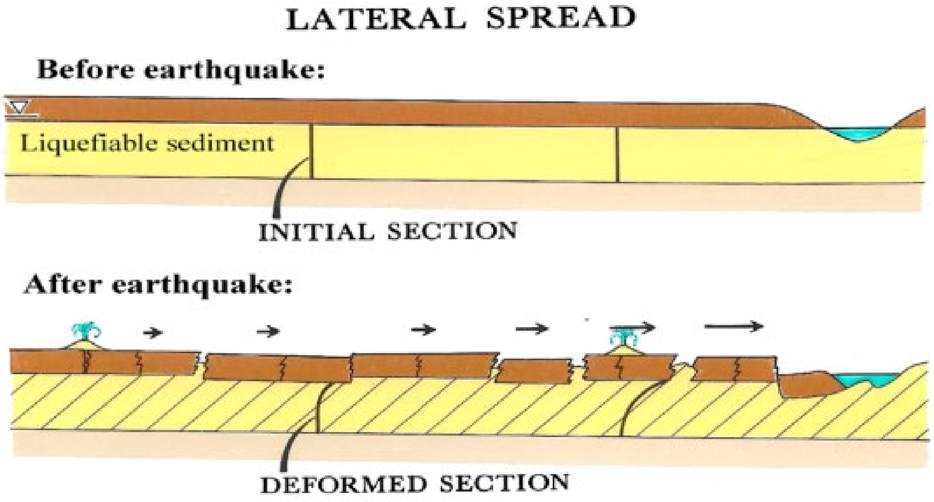

Exhibit 1: Schematic of Lateral Spread Characteristics (Youd 2018)

Lateral spreading will also impose kinematic lateral loads on pile foundations where the soil movements occur relative to the piles. This will primarily impact the piers, and any onshore pile-supported structures, with the impacts being greatest near the shoreline where the liquefiable soils are the thickest and potential deformations are the greatest. Lateral spreading may also impose lateral loads on ground improvement elements, such as exist at Tank Yard 5. While deep foundations and ground improvement may mitigate settlement, it is uncertain if these improvements are sufficient to resist kinematic loads due to lateral spreading.

Seismically-Induced Ground Deformations

We have developed preliminary estimates of vertical and lateral seismically-induced ground deformations to approximate the range of movements expected at the site. These estimates consider the proximity of the site to the free face slope of the waterfront along the Willamette River and a slope height of 70 feet. As indicated previously, the results of seismic slope stability analysis suggest the risk of lateral spreading is greatest within roughly 200 feet of the top of the waterfront slope, which is identified in Exhibit 2 below and Figure 3 as the flow slide zone. In this zone, large masses of ground may travel long distances (likely more than 5 feet) in the form of liquefied flows or blocks of ground riding on liquefied flows.

Our estimates of seismically-induced lateral ground deformations beyond the flow slide zone based on the approach developed by Youd, et al. (2002) and Zhang, et al. (2004), along with an average of the two methods, are presented in Exhibit 2 below. The results indicate a reduction in estimated deformations with greater distances from the shoreline. In addition, the estimates based on Zhang, et al. (2004) are higher than those based on Youd, et al. (2002), with the difference in deformations between the two methods within about 10 to 25 percent. The average estimate of lateral spread deformations based on the two simplified methods is shown on an aerial image of the site in the form of deformation contours presented in Figure 3. As shown in Figure 3, estimated lateral spread deformations outside the flow slide zone range from about nil (west of the truck loading rack where liquefiable soils are absent and bedrock is shallow) to about 3 feet. It should be noted that there is considerable uncertainty in deformation estimates using the approach developed by Youd, et al. (2002) and Zhang, et al. (2004). Actual deformations may vary significantly.

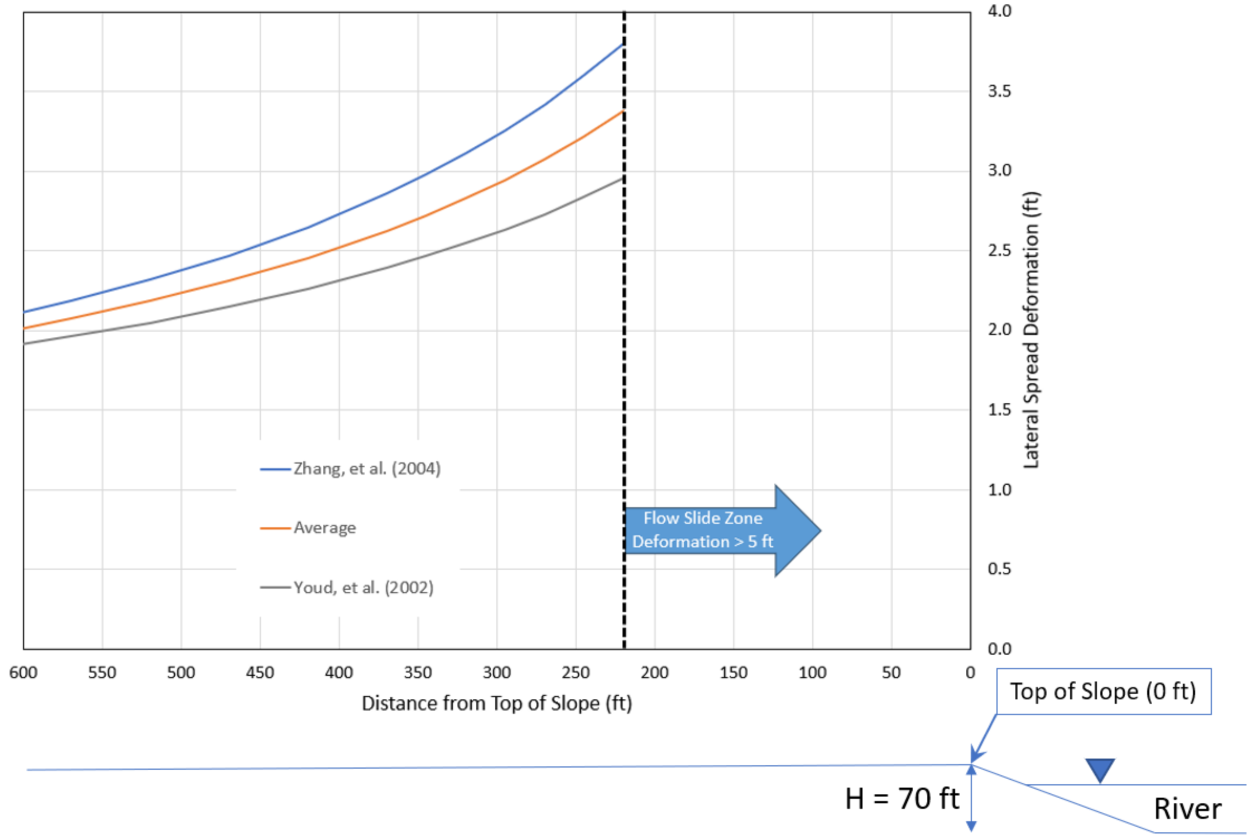

Exhibit 2: Seismically-Induced Lateral Ground Deformation

As indicated previously, the primary mechanisms of liquefaction-induced settlement are reconsolidation, ejecta-induced, and shear-induced deformation. It should be noted that lateral spreading also results in ground settlement, which can be as much as about one-third to one-half of the magnitude of lateral displacement. We summarize our preliminary estimates of vertical settlement from densification, reconsolidation, sediment ejecta, and lateral spreading in Table 1 below. These estimates do not consider shear-induced foundation settlements discussed previously.

Table 1: Seismically-Induced Vertical Settlement

Mechanism	Probable Approximate Vertical Settlement Range[1] (inches)
Densification	< ½
Reconsolidation	2 to 6
Ejecta-Induced[2]	Up to 12 (locally near ejecta)
Vertical Component of Lateral Spreading	Nil to > 30
All the Above	2 to > 30

1. The estimated vertical ground deformations consider free-field conditions. Additional settlement of tanks and other structures may occur due to shear-induced foundation settlement as discussed previously.
2. Ground loss from sediment ejecta is highly variable and difficult to estimate.

CONCLUSIONS

As discussed herein, there are various liquefaction-induced mechanisms that could impact the terminal infrastructure. The most significant risk is related to lateral spreading near the shoreline, where the potential for flow slide failure exists, which can result in impacts on the facilities in this area including kinematic loading on piles supporting the piers. The risk of lateral spreading at the site is significantly reduced at greater distances from the shoreline. Where seismically-induced vertical and lateral ground deformations are not acceptable, mitigation measures could be considered. Mitigation of shoreline deformation could consist of the installation of a subsurface buttress and/or bulkhead structure depending on waterfront configuration. The installation of a waterfront/shoreline buttress would not only mitigate the deformations near the shoreline, but also at greater distances from the shoreline. In addition, the potential for lateral spreading on the waterside of a shoreline buttress and potential kinematic load impacts on the existing tanks supported on improved ground and existing piers would need to be assessed. Assuming lateral deformations are acceptable or have been mitigated, settlement and other foundation impacts could be mitigated by structural improvements/strengthening of shallow foundations, deep foundations, and/or ground improvement to make them less susceptible to vertical ground deformations.

Any future investigations should be focused on the collection of data in support of developing remedial measures or further evaluating the performance of specific structures. While additional investigations will provide data for further subsurface characterization and assessment, this information will not likely change conclusions regarding the overall seismic risk.

LIMITATIONS

This report has been prepared for the use of SGH and is specific to the conditions at the site as described herein. The opinions, conclusions, and recommendations contained in this report are based upon information obtained from existing geotechnical data, experience, and engineering judgment, and have been formulated in accordance with generally accepted geotechnical practices at the time this report was prepared; no other warranty is expressed or implied. In addition, the conclusions and recommendations presented in this report are based on interpretations of the subsurface conditions encountered in widely spaced explorations. Actual conditions may vary. If subsurface conditions encountered in the field differ from those described in this report, Gannett Fleming should be consulted to determine if changes to the conclusions presented herein or supplemental recommendations are required.

The opinions presented in this report are valid as of the date of this report. Changes in the condition of a site can occur with the passage of time, whether due to natural processes or the works of man. In addition, changes in applicable standard of practice can occur, whether from legislation or the broadening of knowledge. Accordingly, this report may be invalidated, wholly or partially, by changes outside of Gannett Fleming's control. In any case, this report should not be relied upon after a period of three years without prior review and approval by Gannett Fleming.

CLOSING

We appreciate the opportunity to collaborate with you on this important project. Please contact us if you have any questions.

Sincerely,

Gannett Fleming, Inc.

REGISTERED PROFESSIONAL ENGINEER
104139PE
OREGON
MAY 14, 2024
BENJAMIN SERNA
EXPIRES: 06/30/2026

05/30/2024

Benjamin Serna, PE
Principal Engineer

R. William Rudolph

R. William Rudolph
Senior Consultant

Attachments: Figures
Appendix A – Existing Data
Appendix B – Seismic CPT Data
Appendix C – Slope Stability Analysis

REFERENCES

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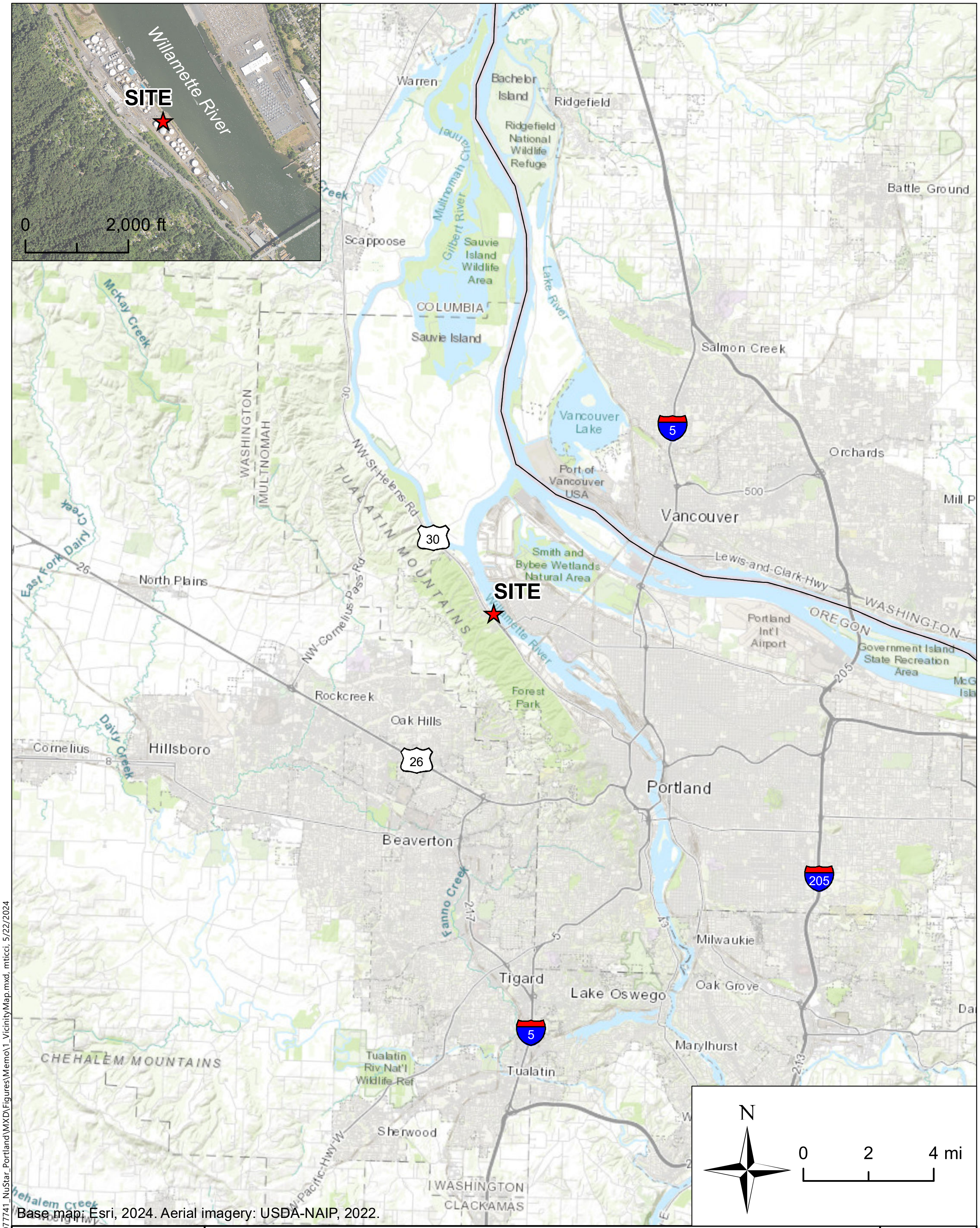
FIGURES



Base map: Esri, 2024. Aerial imagery: USDA-NAIP, 2022.

GANNETT FLEMING

SITE VICINITY PLAN

Shore Terminals Portland Terminal
Preliminary Geotechnical Assessment
Portland, Oregon

FIGURE
1

Date: 5/22/2024	By: M. Ticci	Scale: 1:250,000	Project No: 077741

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Imagery: USDA-NAIP, 2022.

GANNETT FLEMING

EXPLORATION LOCATION PLAN
Shore Terminals Portland Terminal
Preliminary Geotechnical Assessment
Portland, Oregon

FIGURE
2

Date: 5/22/2024	By: M. Ticci	Scale: 1:2,400	Project No: 077741

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Imagery: USDA-NAIP, 2022.

GANNETT FLEMING

PRELIMINARY LATERAL SPREAD DEFORMATIONS

Shore Terminals Portland Terminal
Preliminary Geotechnical Assessment
Portland, Oregon

FIGURE
3

Date: 5/22/2024 | By: M. Ticci | Scale: 1:2,400 | Project No: 077741

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APPENDIX A – EXISTING DATA

APPENDIX A

Soil Boring Logs

CH2MHILL

PROJECT NUMBER:

BORING NUMBER:

SHEET 1 OF 1

BORING LOG EXPLANATION

PROJECT :

LOCATION :

ELEVATION :

DRILLING CONTRACTOR :

DRILLING METHOD AND EQUIPMENT :

WATER LEVELS : ---

START :

END :

LOGGER :

DEPTH BELOW GROUND SURFACE (ft)				STANDARD PENETRATION TEST RESULTS
	INTERVAL (ft)			
		RECOVERY (ft)	#TYPE	6"-6"-6"(-6") (N)
	1.0			
		1.5		
	2.5			
	3.5			
			1-SS	
5	5.0			
10				
15	15.0			
				3-5-6(-4) (11)
	16.5			
20				
25				
30				

SOIL DESCRIPTION

SOIL NAME, USCS GROUP SYMBOL, COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY

Sample Interval: Top/Bottom (ft. bgs)
Amount of Sample Recovered (ft)

Sample Type - Sample Number

(SS)	Standard split-spoon drive sampler, 2.0-inch (51-mm) outside diameter, 1.4-inch (35-mm) inside diameter (without liners)
(M)	Modified split-spoon drive sampler, 2.5-inch (64-mm) outside diameter, 2.0-inch (51-mm) inside diameter (with liners)
(MC)	Modified California split-spoon drive sampler, 3.0-inch (76-mm) outside diameter, 2.4-inch (64-mm) inside diameter (with liners)
(ST)	Thin-walled Shelby tube sampler, 3.0-inch (76-mm) outside diameter, 2.9-inch (74-mm) inside diameter
(G)	Grab sample collected from drill cuttings

Standard Penetration Test Results

Number of blows required to advance driven sampler over three (or four) 6-inch (152-mm) increments. Number in parenthesis is the total number of blows required to advance the sampler 12-inch (305 mm) beyond the first 6-inch (152-mm) interval. Drive samplers advanced using a 140 lb (63.5 kg) Hammer with the 30-inch (762-mm) drop. The blow counts given have not been modified to account for field and/or depth conditions.

General Notes

1) Soil classifications are based on the Unified Soil Classification System. Classifications and descriptions made in the field have been modified based on the results of laboratory testing.

2) Boring logs depict subsurface conditions only at the specific locations and times the boring was made. Logs do not necessarily reflect strata variations that may exist between boring locations.

COMMENTS

DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS, AND INSTRUMENTATION

Comments

Comments and observations regarding drilling or sampling made by the driller or field personnel.

Test

Field and Laboratory tests include the following:

WC	Moisture Content (ASTM D-2216)
UWD	Dry Unit Weight (ASTM D-2937) in pounds per cubic foot (pcf) or kilonewtons per cubic meter (kN/m3)
SA	Seive analysis (ASTM D-1140) See appropriate laboratory data sheets for gradation curve
P200	Percentage of soil particles passing the No.200 sieve (ASTM D-422)
HD	Standard hydrometer analysis
LL PL PI	Atterberg Limits (ASTM D-4318) LL = Liquid Limit, PL = Plastic Limit, PI = Plasticity Index
PP	Unconfined Compressive Strength in tons per square foot (tsf) or kilopascals (kPa) measured using a pocket penetrometer device
TV	Unconfined Compressive Strength in tsf or kPa measured using a torvane device
TX-UU	Unconsolidated Undrained Triaxial Shear Strength in pounds per square foot (psf) or kPa as measured in the laboratory (ASTM D-2850). Confining pressure given in parenthesis
TX-CU	Consolidated Undrained Triaxial Shear Strength in psf or kPa as measured in the laboratory (ASTM D-4767). Confining pressure given in parenthesis
CONSOL	One-Dimensional Consolidation (ASTM D-2435)
PERM	Triaxial, Falling Head Permeability (ASTM D-5084)
OC	Organic content (ASTM D-2974)
CA	Corrosion Analysis

TABLE 1
CRITERIA FOR DESCRIBING MOISTURE CONDITION

Description	Criteria
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp, but no visible water
Wet	Visible free water, usually soil is below water table

TABLE 2
RELATIVE DENSITY OF COARSE-GRAINED SOIL
(Developed from Sowers, 1979)

Blows/Ft	Relative Density	Field Test
0-4	Very loose	Easily penetrated with 1/2-in. steel rod pushed by hand
5-10	Loose	Easily penetrated with 1/2-in. steel rod pushed by hand
11-30	Medium	Easily penetrated with 1/2-in. steel rod driven with 5-lb hammer
31-50	Dense	Penetrated a foot with 1/2-in. steel rod driven with 5-lb hammer
>50	Very Dense	Penetrated only a few inches with 1/2-in. steel rod driven with 5-lb hammer

TABLE 3
CONSISTENCY OF FINE-GRAINED SOIL
(Developed from Sowers, 1979)

Blows/Ft	Consistency	Pocket Penetrometer (TSF)	Torvane (TSF)	Field Test
<2	Very soft	<0.25	<0.12	Easily Penetrated several inches by fist
2-4	Soft	0.25-0.50	<0.12-0.25	Easily Penetrated several inches by thumb
5-8	Firm	0.50-1.0	0.25-0.50	Can be penetrated several inches by thumb with moderate effort
9-15	Stiff	1.0-2.0	0.50-1.0	Readily indented by thumb, but penetrated only with great effort
16-30	Very stiff	2.0-4.0	1.0-2.0	Readily indented by thumbnail
>30	Hard	>4.0	>2.0	Indented with difficulty by thumbnail

USCS CHART

MAJOR DIVISIONS			SYMBOLS		TYPICAL DESCRIPTIONS
			GRAPH	LETTER	
COARSE GRAINED SOILS MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)	░	GW	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
			●	GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)	▥	GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
			▨	GC	CLAYEY GRAVELS, GRAVEL - SAND-CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)	░	SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
			░	SP	POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)	▥	SM	SILTY SANDS, SAND - SILT MIXTURES
			▨	SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50	▥	ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
			▨	CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
			≋	OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50	▥	MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
			▨	CH	INORGANIC CLAYS OF HIGH PLASTICITY
			≋	OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS			≋	PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

CH2MHILL

BORING AND TEST PIT LOG LEGEND:

SAMPLE TYPE:

B - BAG SAMPLE

J - JAR SAMPLE

SS - SPLIT SPOON SAMPLE (ASTM D 1586 UNLESS OTHERWISE NOTED)

W - WASH SAMPLE

UT - UNDISTURBED SAMPLE

NX, NQ, HQ - DIAMOND ROCK CORE SAMPLE

STANDARD PENETRATION TEST:

6"-6"-6"(-6") - THE NUMBER OF BLOWS FOR THREE (OR FOUR) 6-INCH INCREMENTS REQUIRED FROM A 140-LB HAMMER FALLING 30 INCHES TO DRIVE A STANDARD 2-INCH O.D. SPLIT-BARREL SAMPLER (ASTM D 1586)

(N) - THE SUM OF BLOWS FOR THE SECOND AND THIRD 6-INCH INCREMENTS

FIELD TEST:

P.P. - POCKET PENETROMETER
T.V. - TORVANE

NOTES:

1. THE BORING AND/OR TEST PIT LOGS AND RELATED INFORMATION DEPICT SUBSURFACE CONDITIONS ONLY AT THE SPECIFIC LOCATIONS AND TIME INDICATED. SUBSURFACE CONDITIONS AND WATER LEVELS AT OTHER LOCATIONS MAY DIFFER FROM CONDITIONS OCCURRING AT THESE BORING AND/OR TEST PIT LOCATIONS. ALSO, THE PASSAGE OF TIME MAY RESULT IN A CHANGE IN THE CONDITIONS AT THESE LOCATIONS.

2. BORINGS AND/OR TEST PITS WERE LOGGED IN THE FIELD BY A REPRESENTATIVE OF CH2M HILL. SAMPLES WERE EXAMINED AND VISUALLY CLASSIFIED IN APPROXIMATE ACCORDANCE WITH ASTM D 2488.

Boring and Test Pit
Log Legend

CH2MHILL

Rock Quality Classification

RQD (%)	Rock Quality
90-100	Excellent
75-90	Good
50-75	Fair
25-50	Poor
0-25	Very Poor

Joint and Bedding Spacing

Spacing	Joints	Bedding/Foliation
Less than 0.5 in.		Laminated
0.5 in. to 2 in.	Very close	Very thin
2 in. to 1 ft.	Close	Thin
1 ft. to 3 ft.	Moderately close	Medium
3 ft. to 10 ft.	Wide	Thick
More than 10 ft.	Very wide	Very thick (massive)

Texture or Grain Size of Rock

Descriptive Term	Defining Characteristics
Fine-grained	Average grain size up to 0.05 inch
Medium-grained	Average grain size from 0.05 to 0.2 inch
Coarse-grained	Average grain size greater than 0.2 inch

Degree of Weathering

Descriptive Term	Defining Characteristics
Fresh	Rock is unstained and discontinuities are unstained.
Slightly	Discontinuities show some staining on surfaces, but discoloration does not penetrate into the mass.
Moderate	Discontinuity surfaces are stained and discoloration extends into the rock mass.
Highly	Individual rock fragments are thoroughly stained. Feldspars have mostly altered to clays. Rock is beginning to take on soil charateristics.

Hardness of Rock

Descriptive Term	Defining Characteristics
Very hard	Cannot be scratched with a steel knife.
Hard	Difficult to scratch with a steel knife.
Soft	Easily scratched with a steel knife.
Very soft	Can be scratched with fingernail.

Boring and Test Pit Log Legend

CH2MHILL

CH2MHILL

PROJECT NUMBER: 334935.A1.01

BORING NUMBER: B-1

SHEET 1 OF 4

SOIL BORING LOG

PROJECT : Valero LP Tank Farm Expansion Project, Portland, Oregon

LOCATION : Tank Yard 3

ELEVATION :

DRILLING CONTRACTOR : Boart Longyear, Tualatin, Oregon

DRILLING METHOD AND EQUIPMENT : Truck Mounted Mobile B-57 with Automatic Trip Hammer, Mud Rotary and NX-Size Coring

WATER LEVELS : --

START : 10/24/05 10:15

END : 10/24/05 15:00

LOGGER : B. Hoffman

DEPTH BELOW GROUND SURFACE (ft)	INTERVAL (ft)	RECOVERY (ft)	#TYPE	STANDARD PENETRATION TEST RESULTS 6"-6"-6" (N)	SOIL DESCRIPTION: SOIL NAME, USCS GROUP SYMBOL, COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS: DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS, AND INSTRUMENTATION
						Ground surface consists of Poorly graded SAND (SP) fill material. bgs = below ground surface PP = Field Pocket Penetrometer tsf = tons per square foot pcf = pounds per cubic foot
5	5.0 6.5	1.1	1-SS	5-3-3 (6)	**Poorly graded SAND (SP)** Dark brown, moist to wet, loose, fine sand with trace medium sand (FILL)	Driller comments occasional gravels encountered to 4 feet bgs and clean sand below 4 feet bgs.
10	10.0 11.5	1.2	2-SS	4-5-7 (12)	**Poorly graded SAND (SP)** Dark gray, moist to wet, medium dense, fine sand (FILL)	
15	15.0 16.5	1.5	3-SS	13-12-15 (27)	**Poorly graded SAND (SP)** Dark gray, moist to wet, medium dense, fine sand (FILL)	Fuel odor encountered while breaking open split spoon sampler for samples 3-SS and 4-SS.
20	20.0 21.5	1.5	4-SS	11-4-3 (7)	**Lean CLAY (CL)** Gray, wet, soft, low plasticity, 5 to 10 percent fine sand	Soil transitions from Poorly graded SAND (SP) to Lean CLAY (CL) at 21.0 feet bgs within split spoon sampler for sample 4-SS.
25						

CH2MHILL

PROJECT NUMBER:	BORING NUMBER:
334935.A1.01	B-1 SHEET 2 OF 4

SOIL BORING LOG

PROJECT : Valero LP Tank Farm Expansion Project, Portland, Oregon

LOCATION : Tank Yard 3

ELEVATION :

DRILLING CONTRACTOR : Boart Longyear, Tualatin, Oregon

DRILLING METHOD AND EQUIPMENT : Truck Mounted Mobile B-57 with Automatic Trip Hammer, Mud Rotary and NX-Size Coring

WATER LEVELS : —

START : 10/24/05 10:15

END : 10/24/05 15:00

LOGGER : B. Hoffman

DEPTH BELOW GROUND SURFACE (ft)	INTERVAL (ft)	RECOVERY (ft)	#TYPE	STANDARD PENETRATION TEST RESULTS 6"-6"-6" (N)	SOIL DESCRIPTION SOIL NAME, USCS GROUP SYMBOL, COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS, AND INSTRUMENTATION
	25.0 26.5	1.5	5-SS	1-1-2 (3)	**Lean CLAY (CL)** Gray with orange-brown mottling, wet, soft, low plasticity	5-SS: PP = 0.4 tsf WC = 48%
	28.5	2.0	6-ST	push	**Elastic SILT (MH)** Gray with orange-brown mottling, wet, soft, moderate plasticity	6-ST pushed with soft resistance. Poorly graded SAND (SP) observed in top of shelby tube (most likely sluff material). Elastic SILT (MH) observed in bottom of shelby tube. 6-ST: WC = 53% PL = 36%, LL = 63%, PI = 27% Unit Weight = 104 pcf CONSOL
30	30.0	1.5	7-SS	2-2-2 (4)	**Lean CLAY (CL)** Gray, wet, soft, low plasticity	7-SS: WC = 56%
35	35.0					
	36.5	1.5	8-SS	1-1-2 (3)	**SILT (ML)** Brownish gray, wet, soft, low plasticity	8-SS: PP = 0.6 tsf WC = 46% PL = 43%, LL = 46%, PI = 3%
40	40.0					
	41.5	1.0	9-SS	2-2-1 (3)	**SILT with sand (ML)** Gray, wet, soft, non-plastic, 15 to 20 percent fine sand	9-SS: PP = 0 WC = 34% PL, LL, and PI = non-plastic
	43.5	2.0	10-ST	push	**SILT with sand (ML)** Gray, wet, soft, low plasticity, 10 to 15 percent fine sand	10-ST pushed with soft resistance. Poorly graded SAND (SP) observed in top of Shelby tube (most likely sluff material). SILT (ML) observed in bottom of Shelby tube. 10-ST: WC = 38% PL = 35%, LL = 44%, PI = 9% UWD = 115 pcf CONSOL
45	45.0	1.5	11-SS	0-0-0 (0)	**Lean CLAY to SILT (CL-ML)** Gray, wet, very soft, low plasticity	11-SS: PP = 0.1 tsf WC = 28% PL = 22%, LL = 29%, PI = 7%
50						

CH2MHILL

PROJECT NUMBER:	BORING NUMBER:
334935.A1.01	B-1 SHEET 3 OF 4

SOIL BORING LOG

PROJECT : Valero LP Tank Farm Expansion Project, Portland, Oregon LOCATION : Tank Yard 3

ELEVATION : DRILLING CONTRACTOR : Boart Longyear, Tualatin, Oregon

DRILLING METHOD AND EQUIPMENT : Truck Mounted Mobile B-57 with Automatic Trip Hammer, Mud Rotary and NX-Size Coring

WATER LEVELS : — START : 10/24/05 10:15 END : 10/24/05 15:00 LOGGER : B. Hoffman

DEPTH BELOW GROUND SURFACE (ft)	INTERVAL (ft)	RECOVERY (ft)	#TYPE	STANDARD PENETRATION TEST RESULTS 6"-6"-6" (N)	SOIL DESCRIPTION SOIL NAME, USCS GROUP SYMBOL, COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS, AND INSTRUMENTATION
	50.0 51.5	1.2	12-SS	3-3-4 (7)	**Silty SAND (SM)** Gray, wet, loose, fine sand, 18 percent low plasticity silt	12-SS: WC = 42% P200 = 18%
						Driller comments rock encountered at depth of 53 feet bgs.
55	55.0 55.1	0.0	13-SS	50/1" (50/1")	No Recovery Begin Rock Coring at 55.1 ft below ground surface See sheet 4 of 4 for rock core log	
60						
65						
70						
75						

CH2MHILL

PROJECT NUMBER: 334935.A1.01

BORING NUMBER: B-1 SHEET 4 OF 4

ROCK CORE LOG

PROJECT : Valero LP Tank Farm Expansion Project, Portland, Oregon
LOCATION : Tank Yard 3
ELEVATION :
DRILLING CONTRACTOR : Boart Longyear, Tualatin, Oregon
DRILLING METHOD AND EQUIPMENT : Truck Mounted Mobile B-57 with Automatic Trip Hammer, Mud Rotary and NX-Size Coring
WATER LEVELS : —
START : 10/24/05 10:15
END : 10/24/05 15:00
LOGGER : B. Hoffman

DEPTH BELOW SURFACE (ft)	CORE RUN, LENGTH, AND RECOVERY (%)	DISCONTINUITIES			GRAPHIC LOG	LITHOLOGY	COMMENTS
		R Q D (%)	FRACTURES PER FOOT	DESCRIPTION			
				DEPTH, TYPE, ORIENTATION, ROUGHNESS, PLANARITY, INFILLING MATERIAL AND THICKNESS, SURFACE STAINING, AND TIGHTNESS		ROCK TYPE, COLOR, MINERALOGY, TEXTURE, WEATHERING, HARDNESS, AND ROCK MASS CHARACTERISTICS	SIZE AND DEPTH OF CASING, FLUID LOSS, CORING RATE AND SMOOTHNESS, CAVING ROD DROPS, TEST RESULTS, ETC.
55	55.1						
	1-NX 5 ft 86%	18	>10 >10 >10 3 0 0	57.3' 2 Joints, 70 degrees, smooth and planar, some iron-oxide staining 57.6' Joint, 50 degrees, smooth and planar, some iron-oxide staining 57.6' to 58.0' Highly fractured 58.3' Joint, 0 degrees, rough and planar 58.8' 2 Joints, 10 degrees, rough and stepped		**BASALT** Dark gray, fine grained, fresh to slightly weathered, hard to very hard (R4-R5), some vesicles	See Soil Boring Log (Sheets 1 through 3 of 4) for log of 0 to 55.1 feet
60	60.1						
						Bottom of Hole at 60.1 ft below ground surface	End of boring. Boring abandoned according to OAR 690-240. Boring backfilled with bentonite chips to the ground surface.
65							
70							
75							

CH2MHILL

PROJECT NUMBER: 334935.A1.01

BORING NUMBER: B-2 SHEET 1 OF 3

SOIL BORING LOG

PROJECT : Valero LP Tank Farm Expansion Project, Portland, Oregon

LOCATION : Tank Yard 5 (708729.9 N, 7620416.4 E)

ELEVATION : 37.3 ft

DRILLING CONTRACTOR : Boart Longyear, Tualatin, Oregon

DRILLING METHOD AND EQUIPMENT : Truck Mounted Mobile B-57 with Automatic Trip Hammer, Mud Rotary and NX-Size Coring

WATER LEVELS : —

START : 10/25/05 12:00

END : 10/25/05 14:30

LOGGER : B. Hoffman

DEPTH BELOW GROUND SURFACE (ft)	INTERVAL (ft)	RECOVERY (ft)	#TYPE	STANDARD PENETRATION TEST RESULTS 6"-6"-6" (N)	SOIL DESCRIPTION SOIL NAME, USCS GROUP SYMBOL, COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS, AND INSTRUMENTATION
						Ground surface consists of Poorly graded SAND (SP) fill material bgs = below ground surface PP = Field Pocket Penetrometer tsf = tons per square foot
5	5.0 6.5	1.3	1-SS	7-8-10 (18)	**Poorly graded SAND (SP)** Dark brown, moist, medium dense, fine grained sand with trace medium sand (FILL)	
10	10.0 11.5	1.0	2-SS	5-6-8 (14)	**Poorly graded SAND with silt (SP-SM)** Dark brown, moist, medium dense, fine grained sand with trace medium sand, 7 percent low plasticity silt (FILL)	2-SS: WC = 17% SA Gravel = 0% Sand = 93% P200 = 7%
15	15.0 16.5	1.5	3-SS	1-2-2 (4)	**SILT (ML)** Brown, wet, soft, low plasticity, 5 percent fine sand	3-SS: PP = 0.5 tsf WC = 33% PL = 25%, LL = 33%, PI = 8%
20	20.0 21.5	1.5	4-SS	1-4-15 (19)	**SILT with gravel (ML)** Brown and dark gray, wet, very stiff, low plasticity, 20 percent fine to coarse angular gravel	4-SS taken right at transition from SILT (ML) to SILT with gravel (ML). SILT with gravel (ML) material possibly landslide debris or material washed from hillside west of site. 4-SS: WC = 32% Driller comments drilling resistance highly variable below 20 feet bgs
25						

CH2MHILL

PROJECT NUMBER: 334935.A1.01

BORING NUMBER: B-2 SHEET 2 OF 3

SOIL BORING LOG

PROJECT : Valero LP Tank Farm Expansion Project, Portland, Oregon

LOCATION : Tank Yard 5 (708729.9 N, 7620416.4 E)

ELEVATION : 37.3 ft

DRILLING CONTRACTOR : Boart Longyear, Tualatin, Oregon

DRILLING METHOD AND EQUIPMENT : Truck Mounted Mobile B-57 with Automatic Trip Hammer, Mud Rotary and NX-Size Coring

WATER LEVELS : ---

START : 10/25/05 12:00

END : 10/25/05 14:30

LOGGER : B. Hoffman

DEPTH BELOW GROUND SURFACE (ft)	INTERVAL (ft)	RECOVERY (ft)	#TYPE	STANDARD PENETRATION TEST RESULTS 6"-6"-6" (N)	SOIL DESCRIPTION SOIL NAME, USCS GROUP SYMBOL, COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS, AND INSTRUMENTATION
	25.0 26.5	0.3	5-SS	1-4-6 (10)	**SILT with gravel (ML)** Brown and dark gray, wet, stiff, low plasticity, 15 percent fine to coarse angular gravel	
30	30.0 31.5	1.5	6-SS	2-2-4 (6)	**Sandy SILT (ML)** Grayish brown, wet, firm, non-plastic to low plasticity, 47 percent fine sand	6-SS: WC = 37% P200 = 53%
35	35.0 36.5	1.5	7-SS	3-2-5 (7)	**Sandy SILT (ML)** Grayish brown, wet, firm, non-plastic to low plasticity, 15 percent fine sand	7-SS: WC = 40% HD Gravel = 0% Sand = 15% Silt = 73% Clay = 12%
						Driller comments rock encountered at 39 feet bgs
40	40.0 40.1	0.0	8-SS	50/1" (50/1")	No Recovery Begin Rock Coring at 40.1 ft below ground surface See sheet 3 of 3 for rock core log	
45						
50						

CH2MHILL

PROJECT NUMBER: 334935.A1.01

BORING NUMBER: B-2

SHEET 3 OF 3

ROCK CORE LOG

PROJECT : Valero LP Tank Farm Expansion Project, Portland, Oregon

LOCATION : Tank Yard 5 (708729.9 N, 7620416.4 E)

ELEVATION : 37.3 ft

DRILLING CONTRACTOR : Boart Longyear, Tualatin, Oregon

DRILLING METHOD AND EQUIPMENT : Truck Mounted Mobile B-57 with Automatic Trip Hammer, Mud Rotary and NX-Size Coring

WATER LEVELS : —

START : 10/25/05 12:00

END : 10/25/05 14:30

LOGGER : B. Hoffman

DEPTH BELOW SURFACE (ft)	CORE RUN, LENGTH, AND RECOVERY (%)	DISCONTINUITIES			GRAPHIC LOG	LITHOLOGY	COMMENTS
		R Q D (%)	FRACTURES PER FOOT	DESCRIPTION DEPTH, TYPE, ORIENTATION, ROUGHNESS, PLANARITY, INFILLING MATERIAL AND THICKNESS, SURFACE STAINING, AND TIGHTNESS		ROCK TYPE, COLOR, MINERALOGY, TEXTURE, WEATHERING, HARDNESS, AND ROCK MASS CHARACTERISTICS	SIZE AND DEPTH OF CASING, FLUID LOSS, CORING RATE AND SMOOTHNESS, CAVING ROD DROPS, TEST RESULTS, ETC.
30							
35							
40	40.1						
	1-NX 4 ft 100%	33	>10 >10 >10 >10	40.1' to 40.8' Highly fractured 41.0' to 41.3', Highly fractured 41.6' Joint, 0 degrees, rough and planar 41.7' Joint, 15 degrees, rough and planar 41.8' to 42.5' Highly fractured 43.2' Joint, 20 degrees, rough and planar 43.2' to 44.1' Highly fractured		**BASALT** Dark gray, fine grained, fresh to slightly weathered, hard to very hard (R4-R5)	See Soil Boring Log (Sheets 1 and 2 of 3) for log of 0 to 40.1 feet
	44.1		0				
45						Bottom of Hole at 44.1 ft below ground surface	End of boring. Boring abandoned according to OAR 690-240. Boring backfilled with bentonite chips to the ground surface.
50							

CH2MHILL

PROJECT NUMBER:	BORING NUMBER:
334935.A1.01	B-3 SHEET 1 OF 2

SOIL BORING LOG

PROJECT : Valero LP Tank Farm Expansion Project, Portland, Oregon LOCATION : Tank Yard 5 (708755.6 N, 7620295.8 E)

ELEVATION : 37.3 ft DRILLING CONTRACTOR : Boart Longyear, Tualatin, Oregon

DRILLING METHOD AND EQUIPMENT : Truck Mounted Mobile B-57 with Automatic Trip Hammer, Mud Rotary

WATER LEVELS : — START : 10/25/05 09:00 END : 10/25/05 11:40 LOGGER : B. Hoffman

DEPTH BELOW GROUND SURFACE (ft)	INTERVAL (ft)	RECOVERY (ft)	#TYPE	STANDARD PENETRATION TEST RESULTS 6"-6"-6" (N)	SOIL DESCRIPTION SOIL NAME, USCS GROUP SYMBOL, COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS, AND INSTRUMENTATION
						Ground surface consists of Poorly graded SAND (SP) fill material. bgs = below ground surface Driller comments wood debris at 2 feet bgs
5	5.0 6.5	0.8	1-SS	2-3-3 (6)	**Poorly graded SAND (SP)** Dark brown, moist, loose, fine grained sand with trace medium sand (FILL)	1-SS: WC = 22% SA Gravel = 0% Sand = 95% P200 = 5%
						Driller comments gravels at 8 feet bgs
10	10.0 11.5	0.2	2-SS	10-9-6 (15)	**Poorly graded GRAVEL (GP)** Dark gray, moist, medium dense, fine to coarse angular gravel, fine to coarse sand, 5 percent low plasticity silt (FILL)	
15	15.0 16.5	1.5	3-SS	1-2-19 (21)	**Gravelly SILT (ML)** Orange-brown and dark gray, wet, very stiff, low plasticity, 30 percent fine to coarse angular gravel, FeO staining	Driller comments material varying greatly in drilling resistance below 14 feet bgs - gravels, cobbles, and soft silt 3-SS: WC = 24% Gravelly SILT (ML) and SILT with gravel (ML) material possibly landslide debris or material washed from hillside west of site.
20	20.0 21.5	0.0	4-SS	18-12-5 (17)	No Recovery	
25						

CH2MHILL

PROJECT NUMBER: 334935.A1.01

BORING NUMBER: B-3 SHEET 2 OF 2

SOIL BORING LOG

PROJECT : Valero LP Tank Farm Expansion Project, Portland, Oregon
LOCATION : Tank Yard 5 (708755.6 N, 7620295.8 E)
ELEVATION : 37.3 ft
DRILLING CONTRACTOR : Boart Longyear, Tualatin, Oregon
DRILLING METHOD AND EQUIPMENT : Truck Mounted Mobile B-57 with Automatic Trip Hammer, Mud Rotary
WATER LEVELS : —
START : 10/25/05 09:00
END : 10/25/05 11:40
LOGGER : B. Hoffman

DEPTH BELOW GROUND SURFACE (ft)	INTERVAL (ft)	RECOVERY (ft)	#TYPE	STANDARD PENETRATION TEST RESULTS 6"-6"-6" (N)	SOIL DESCRIPTION: SOIL NAME, USCS GROUP SYMBOL, COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS: DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS, AND INSTRUMENTATION
	25.0 26.5	1.0	5-SS	6-1-3 (4)	**SILT with gravel (ML)** Brown, wet, soft, low plasticity, 15 to 20 percent fine to coarse subangular gravel, 10 percent fine sand	5-SS: WC = 28%
						Driller comments dense gravel or weathered rock at 28 feet bgs
30	30.0 30.4	0.4	6-SS	50/5" (50/5")	**Silty GRAVEL (GM)** Brown and gray, wet, very dense, fine to coarse angular gravel, 30 percent low plasticity silt (weathered basalt)	Driller comments solid basalt at 30.4 feet
					Bottom of Hole at 30.4 ft below ground surface	End of boring. Boring abandoned according to OAR 690-240. Boring backfilled with bentonite chips to the ground surface.
35						
40						
45						
50						

CH2MHILL

PROJECT NUMBER: 334935.A1.01

BORING NUMBER: B-4 SHEET 1 OF 3

SOIL BORING LOG

PROJECT : Valero LP Tank Farm Expansion Project, Portland, Oregon

LOCATION : Tank Yard 5 (708936.5 N, 7620381.7 E)

ELEVATION : 34.1 ft

DRILLING CONTRACTOR : Boart Longyear, Tualatin, Oregon

DRILLING METHOD AND EQUIPMENT : Truck Mounted Mobile B-59 with Cathead and 140-lb Hammer, Mud Rotary

WATER LEVELS : —

START : 3/2/06 09:50

END : 3/2/06 16:10

LOGGER : B. Hoffman

DEPTH BELOW GROUND SURFACE (ft)	INTERVAL (ft)	RECOVERY (ft)	#TYPE	STANDARD PENETRATION TEST RESULTS 6"-6"-6" (N)	SOIL DESCRIPTION: SOIL NAME, USCS GROUP SYMBOL, COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS: DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS, AND INSTRUMENTATION
						Ground surface consists of Poorly graded SAND (SP) fill material. Ground surface observed to be approximately 25 to 30 feet above Willamette River. One rope wrap used on cathead due to wet weather conditions bgs = below ground surface
5	5.0 6.5	1.0	1-SS	8-14-15 (29)	**Poorly graded SAND with silt (SP-SM)** Brown, moist to wet, medium dense, fine sand, 5 to 10 percent low plasticity silt, trace fine gravel (FILL)	
10	10.0 11.5	0.5	2-SS	27-28-23 (51)	**Poorly graded SAND (SP)** Brown, moist, very dense, fine sand, less than 5 percent silt (FILL)	Piece of riprap in bore hole at approximately 10 feet bgs - cause of high blow counts for 2-SS
						Piece of riprap in bore hole at approximately 14 feet bgs
15	15.0 16.5	0.8	3-SS	10-12-14 (26)	**Poorly graded SAND (SP)** Brown, moist to wet, medium dense, fine sand, 5 percent low plasticity silt (FILL)	
20	20.0 21.5	1.0	4-SS	8-4-4 (8)	**Sandy SILT (ML)** Brown and gray, wet, firm, low plasticity, 31 percent fine sand	Soil transitions from Poorly graded SAND (SP) to Sandy SILT (ML) at 21.0 feet bgs within split spoon sampler for sample 4-SS. 4-SS: WC = 41% P200 = 69%
25						

CH2MHILL

PROJECT NUMBER:	BORING NUMBER:	
334935.A1.01	B-4	SHEET 2 OF 3

SOIL BORING LOG

PROJECT : Valero LP Tank Farm Expansion Project, Portland, Oregon

LOCATION : Tank Yard 5 (708936.5 N, 7620381.7 E)

ELEVATION : 34.1 ft

DRILLING CONTRACTOR : Boart Longyear, Tualatin, Oregon

DRILLING METHOD AND EQUIPMENT : Truck Mounted Mobile B-59 with Cathead and 140-lb Hammer, Mud Rotary

WATER LEVELS : —

START : 3/2/06 09:50

END : 3/2/06 16:10

LOGGER : B. Hoffman

DEPTH BELOW GROUND SURFACE (ft)	INTERVAL (ft)	RECOVERY (ft)	#TYPE	STANDARD PENETRATION TEST RESULTS 6"-6"-6" (N)	SOIL DESCRIPTION: SOIL NAME, USCS GROUP SYMBOL, COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS: DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS, AND INSTRUMENTATION
	25.0 26.5	1.1	5-SS	3-4-6 (10)	**Sandy SILT with gravel (ML)** Gray with orange-brown mottling, wet, stiff, low to moderate plasticity, 30 percent fine sand, 15 percent fine gravel	Driller comments high variability in resistance to drilling from 25 to 35 feet bgs - sand, silt, gravel, and occasional cobbles
30	30.0 31.5	0.7	6-SS	10-3-3 (6)	**Silty GRAVEL with sand (GM)** Gray and brown, wet, loose, 35 percent fine to coarse gravel, 20 percent fine to coarse sand, 45 percent low plasticity silt	6-SS: WC = 25% PL = 24%, LL = 28%, PI = 4% SA Gravel = 35% Sand = 20% P200 = 45%
35	35.0 36.5	0.0	7-SS	5-10-12 (22)	No Recovery	
40	40.0 41.5	1.5	8-SS	3-3-4 (7)	**SILT with sand (ML)** Brownish gray, wet, firm, low plasticity silt, 28 percent fine sand	8-SS: WC = 40% P200 = 72%
45	45.0 46.5	0.0	9-SS	10-14-11 (25)	No Recovery	
50						

CH2MHILL

PROJECT NUMBER: 334935.A1.01

BORING NUMBER: B-4 SHEET 3 OF 3

SOIL BORING LOG

PROJECT : Valero LP Tank Farm Expansion Project, Portland, Oregon
LOCATION : Tank Yard 5 (708936.5 N. 7620381.7 E)
ELEVATION : 34.1 ft
DRILLING CONTRACTOR : Boart Longyear, Tualatin, Oregon
DRILLING METHOD AND EQUIPMENT : Truck Mounted Mobile B-59 with Cathead and 140-lb Hammer, Mud Rotary
WATER LEVELS : —
START : 3/2/06 09:50
END : 3/2/06 16:10
LOGGER : B. Hoffman

DEPTH BELOW GROUND SURFACE (ft)	INTERVAL (ft)	RECOVERY (ft)	#TYPE	STANDARD PENETRATION TEST RESULTS 6"-6"-6" (N)	SOIL DESCRIPTION SOIL NAME, USCS GROUP SYMBOL, COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS, AND INSTRUMENTATION
	50.0 51.5	0.9	10-SS	5-8-18 (26)	Silty SAND (SM) Brown, moist to wet, medium dense, fine sand, 18 percent low plasticity silt	10-SS: WC = 27% P200 = 18%
	53.0	0.0	11-SS	50/0" (50/0")	No Recovery Bottom of Hole at 53.0 ft below ground surface	Driller comments rock encountered at 53 feet bgs End of boring. Boring abandoned according to OAR 690-240. Boring backfilled with bentonite chips to the ground surface.
55						
60						
65						
70						
75						

CH2MHILL

PROJECT NUMBER: 334935.A1.01

BORING NUMBER: B-5 SHEET 1 OF 4

SOIL BORING LOG

PROJECT : Valero LP Tank Farm Expansion Project, Portland, Oregon

LOCATION : Tank Yard 5 (708749.3 N, 7620496.6 E)

ELEVATION : 37.0 ft

DRILLING CONTRACTOR : Boart Longyear, Tualatin, Oregon

DRILLING METHOD AND EQUIPMENT : Truck Mounted Mobile B-59 with Cathead and 140-lb Hammer, Mud Rotary and NX-Size Coring

WATER LEVELS : —

START : 3/3/06 08:20

END : 3/3/06 12:00

LOGGER : B. Hoffman

DEPTH BELOW GROUND SURFACE (ft)	INTERVAL (ft)	RECOVERY (ft)	#TYPE	STANDARD PENETRATION TEST RESULTS 6"-6"-6" (N)	SOIL DESCRIPTION SOIL NAME, USCS GROUP SYMBOL, COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS, AND INSTRUMENTATION
						Ground surface consists of Poorly graded SAND (SP) fill material. One rope wrap used on cathead due to wet weather conditions bgs = below ground surface PP = field pocket penetrometer tsf = tons per square foot
5	5.0 6.5	0.8	1-SS	8-12-11 (23)	**Poorly graded SAND (SP)** Brown, moist, medium dense, fine sand, less than 5 percent silt (FILL)	
10	10.0 11.5	0.6	2-SS	8-7-10 (17)	**Poorly graded SAND (SP)** Brown, moist, medium dense, fine sand with trace medium to coarse sand, less than 5 percent silt (FILL)	
15	15.0 16.5	0.5	3-SS	6-7-9 (16)	**Poorly graded SAND (SP)** Brown, wet, medium dense, fine sand with some medium to coarse sand and trace fine gravel, less than 5 percent silt (FILL)	
20	20.0 21.5	0.4	4-SS	8-7-7 (14)	**Poorly graded SAND (SP)** Brown, wet, medium dense, fine sand with some medium to coarse sand and trace fine gravel, less than 5 percent silt (FILL)	
25						

CH2MHILL

PROJECT NUMBER:	BORING NUMBER:
334935.A1.01	B-5 SHEET 2 OF 4

SOIL BORING LOG

PROJECT : Valero LP Tank Farm Expansion Project, Portland, Oregon LOCATION : Tank Yard 5 (708749.3 N, 7620496.6 E)

ELEVATION : 37.0 ft DRILLING CONTRACTOR : Boart Longyear, Tualatin, Oregon

DRILLING METHOD AND EQUIPMENT : Truck Mounted Mobile B-59 with Cathead and 140-lb Hammer, Mud Rotary and NX-Size Coring

WATER LEVELS : — START : 3/3/06 08:20 END : 3/3/06 12:00 LOGGER : B. Hoffman

DEPTH BELOW GROUND SURFACE (ft)	INTERVAL (ft)	RECOVERY (ft)	#TYPE	STANDARD PENETRATION TEST RESULTS 6"-6"-6" (N)	SOIL DESCRIPTION SOIL NAME, USCS GROUP SYMBOL, COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS, AND INSTRUMENTATION
	25.0 26.5	1.5	5-SS	1-0-2 (2)	**Sandy SILT (ML)** Gray with orange-brown mottling, wet, soft, low plasticity, 36 percent fine sand, trace organics	5-SS: WC = 49% PL = 30%, LL = 31%, PI = 1% P200 = 64%
30	30.0 31.5	1.5	6-SS	3-5-5 (10)	**Lean CLAY or SILT (CL-ML)** Brown, wet, stiff, low plasticity, 10 percent fine sand	6-SS: PP = 1 tsf WC = 35% PL = 24%, LL = 36%, PI = 12%
35	35.0 36.5	1.5	7-SS	3-3-3 (6)	**SILT (ML)** Mottled brown and gray, wet, firm, low plasticity, 13 percent fine sand	7-SS: PP = 0.5 tsf WC = 41% PL = 26%, LL = 31%, PI = 5% P200 = 87% Sand content increasing near bottom of sample 7-SS
	38.5	0.0	8-ST	push	No Recovery	
40	40.0 41.5	1.5	9-SS	2-1-2 (3)	**SILT with sand (ML)** Brown and gray, wet, soft, low plasticity to non-plastic, 26 percent fine sand	9-SS: WC = 37% P200 = 74%
	42.5 44.0	0.6	10-SS	6-10-10 (20)	**Silty GRAVEL with sand (GM)** Gray with orange-brown staining, wet, medium dense, fine to coarse subangular gravel, 32 percent fine to coarse sand, 30 percent low plasticity silt, FeO staining	Driller comments soil changes to sand and gravel at approximately 42.5 feet bgs 10-SS: WC = 21% SA Gravel = 38% Sand = 32% P200 = 30%
45	45.0 46.5	1.5	11-SS	15-17-33 (50)	**Poorly graded SAND with silt (SP-SM)** Dark gray and brown, moist to wet, very dense, fine sand, 10 percent low plasticity silt	
50						

CH2MHILL

PROJECT NUMBER: 334935.A1.01

BORING NUMBER: B-5 SHEET 3 OF 4

SOIL BORING LOG

PROJECT : Valero LP Tank Farm Expansion Project, Portland, Oregon

LOCATION : Tank Yard 5 (708749.3 N, 7620496.6 E)

ELEVATION : 37.0 ft

DRILLING CONTRACTOR : Boart Longyear, Tualatin, Oregon

DRILLING METHOD AND EQUIPMENT : Truck Mounted Mobile B-59 with Cathead and 140-lb Hammer, Mud Rotary and NX-Size Coring

WATER LEVELS : ---

START : 3/3/06 08:20

END : 3/3/06 12:00

LOGGER : B. Hoffman

DEPTH BELOW GROUND SURFACE (ft)	INTERVAL (ft)	RECOVERY (ft)	#TYPE	STANDARD PENETRATION TEST RESULTS 6"-6"-6" (N)	SOIL DESCRIPTION: SOIL NAME, USCS GROUP SYMBOL, COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY	COMMENTS: DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS, AND INSTRUMENTATION
	50.0 50.9	0.9	12-SS	10-50/5" (50/5")	**Silty SAND (SM)** Gray, wet, very dense, fine sand, 20 percent low plasticity silt	Basalt encountered at 50.9 feet bgs
					Begin Rock Coring at 50.9 ft below ground surface See sheet 4 of 4 for rock core log	
55						
60						
65						
70						
75						

CH2MHILL

PROJECT NUMBER: 334935.A1.01

BORING NUMBER: B-5

SHEET 4 OF 4

ROCK CORE LOG

PROJECT : Valero LP Tank Farm Expansion Project, Portland, Oregon

LOCATION : Tank Yard 5 (708749.3 N, 7620496.6 E)

ELEVATION : 37.0 ft

DRILLING CONTRACTOR : Boart Longyear, Tualatin, Oregon

DRILLING METHOD AND EQUIPMENT : Truck Mounted Mobile B-59 with Cathead and 140-lb Hammer, Mud Rotary and NX-Size Coring

WATER LEVELS : —

START : 3/3/06 08:20

END : 3/3/06 12:00

LOGGER : B. Hoffman

DEPTH BELOW SURFACE (ft)	CORE RUN, LENGTH, AND RECOVERY (%)	R Q D (%)	FRACTURES PER FOOT	DISCONTINUITIES DESCRIPTION DEPTH, TYPE, ORIENTATION, ROUGHNESS, PLANARITY, INFILLING MATERIAL AND THICKNESS, SURFACE STAINING, AND TIGHTNESS	GRAPHIC LOG	LITHOLOGY ROCK TYPE, COLOR, MINERALOGY, TEXTURE, WEATHERING, HARDNESS, AND ROCK MASS CHARACTERISTICS	COMMENTS SIZE AND DEPTH OF CASING, FLUID LOSS, CORING RATE AND SMOOTHNESS, CAVING ROD DROPS, TEST RESULTS, ETC.
	50.9						
	1-NX 5 ft 6%	0	▨			**BASALT** Dark gray, fine grained, fractured, hard to very hard (R4-R5)	See Soil Boring Log (Sheets 1 through 3 of 4) for log of 0 to 50.9 feet Driller comments core bit ground by the end of run 1-NX. Lost most of rock core back into bore hole.
55							
	55.9						
						Bottom of Hole at 55.9 ft below ground surface	End of boring. Boring abandoned according to OAR 690-240. Boring backfilled with bentonite chips to the ground surface.
60							
65							
70							
75							

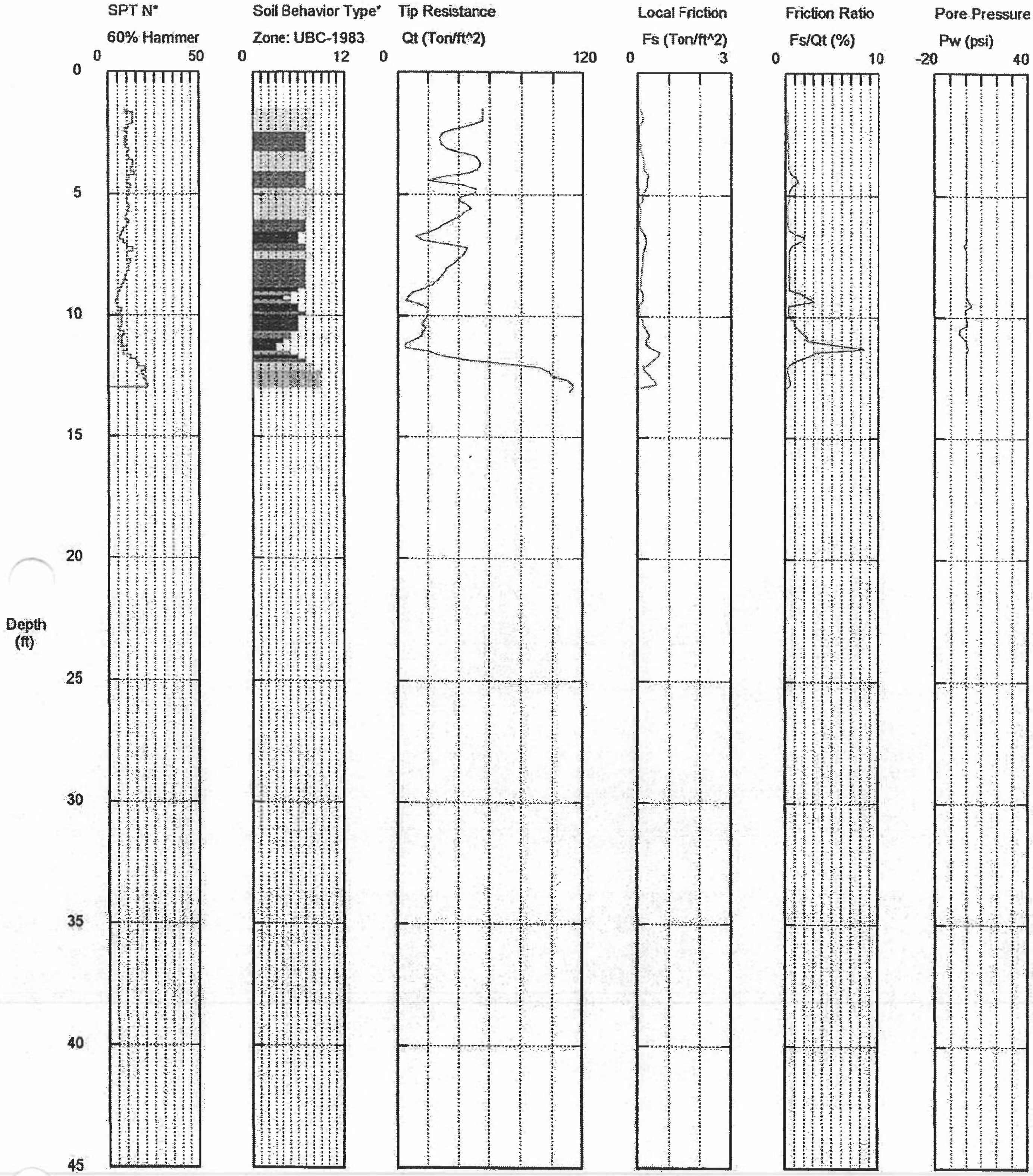
APPENDIX B

Cone Penetrometer Data

CH2M / P-1 / 9420 SW H30 PD

Operator: KDV/SVAN/GEOTECH
Sounding: FILL30
Cone Used: 4CH

CPT Date/Time: 10-24-05 10:16
Location: P-1 9420SW H30PD
Job Number: CH2M334935.A1.01



Maximum Depth = 13.12 feet

Depth Increment = 0.164 feet

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

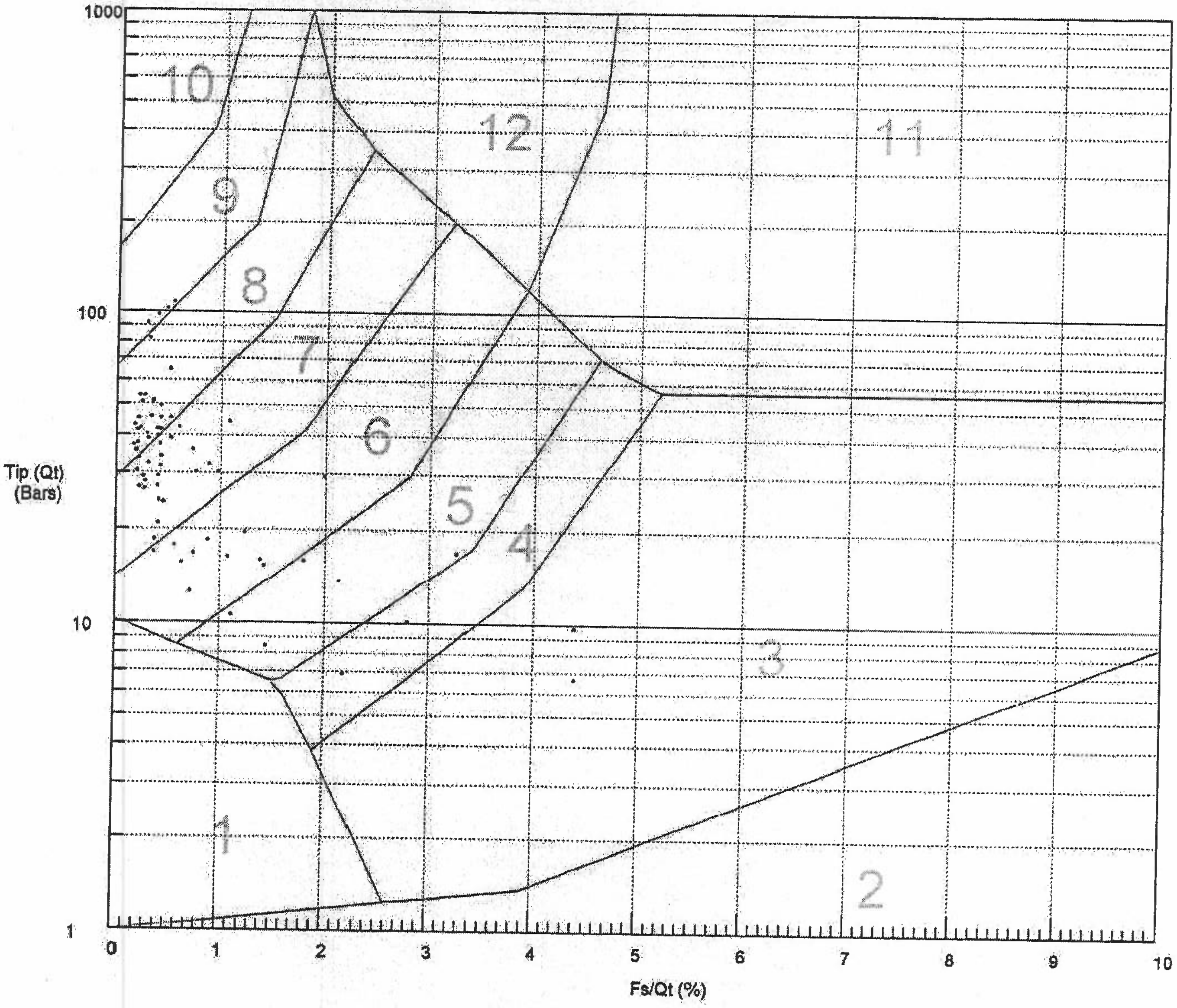
*Soil behavior type and SPT based on data from UBC-1983

CH2M / P-1 9420 SW H30 PD

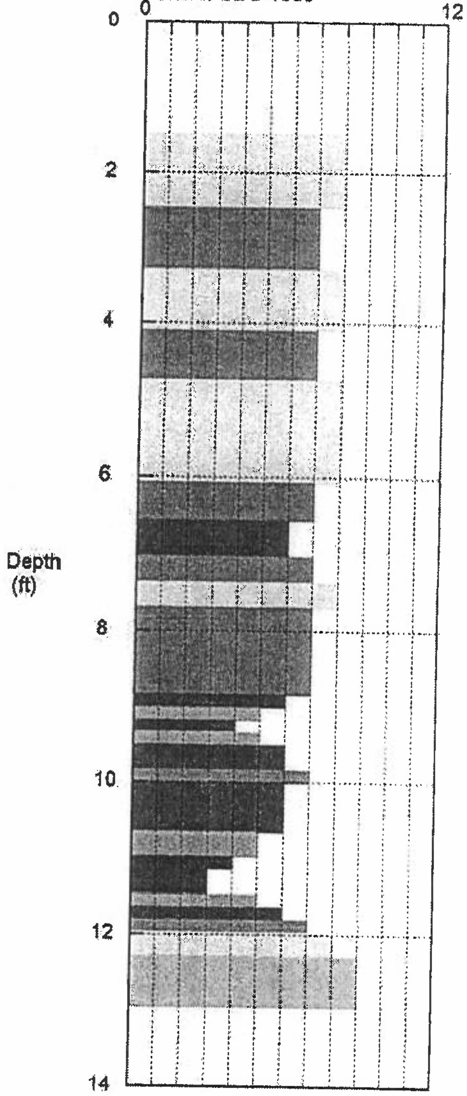
Operator: KDV/SVAN/GEOTECH
Sounding: FILL30
Cone Used: 4CH

CPT Date/Time: 10-24-05 10:16
Location: P-1 9420SW H30PD
Job Number: CH2M334935.A1.01

Classification Data:
Robertson and Campanella UBC-1983



Soil Behavior Type*
Zone: UBC-1983

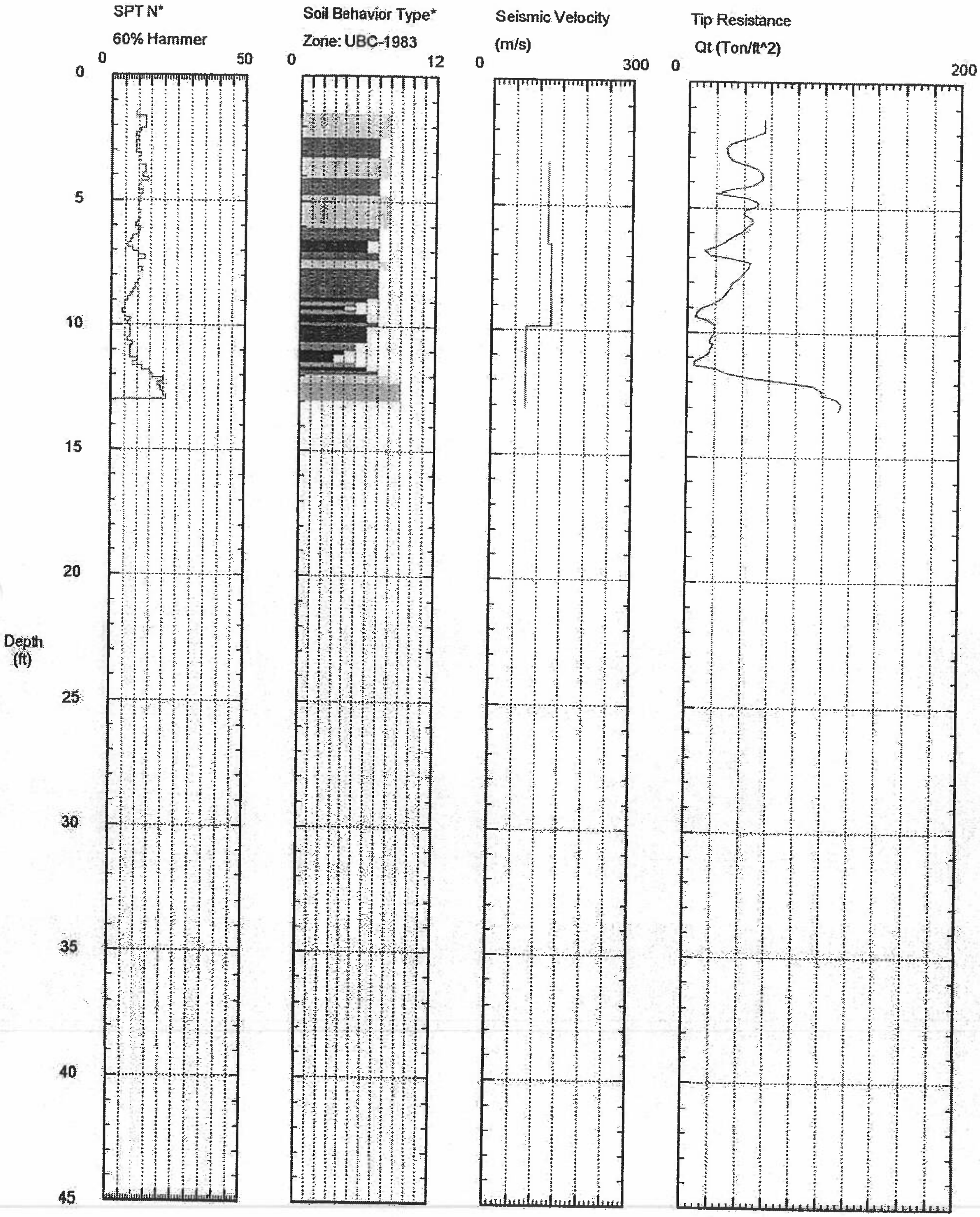


▨ 1	sensitive fine grained
▨ 2	organic material
▨ 3	clay
▨ 4	silty clay to clay
▨ 5	clayey silt to silty clay
▨ 6	sandy silt to clayey silt
▨ 7	silty sand to sandy silt
▨ 8	sand to silty sand
▨ 9	sand
▨ 10	gravelly sand to sand
▨ 11	very stiff fine grained (*)
▨ 12	sand to clayey sand (*)

CH2M / P-1 / 9420 SW H30 PD

Operator: KDV/SVAN/GEOTECH
Sounding: FILL30
Cone Used: 4CH

CPT Date/Time: 10-24-05 10:16
Location: P-1 9420SW H30PD
Job Number: CH2M334935.A1.01



Maximum Depth = 13.12 feet

Depth Increment = 0.164 feet

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

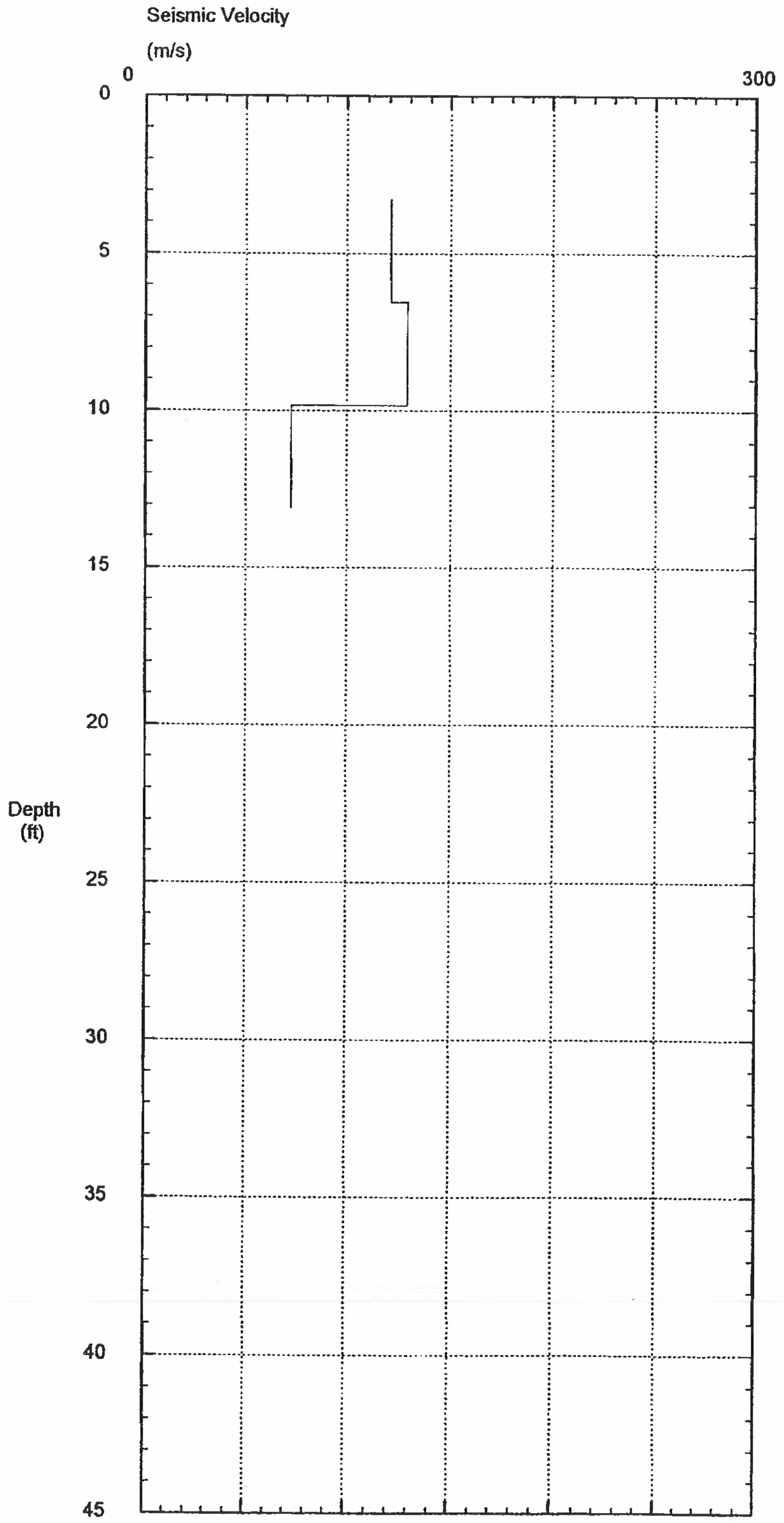
THE SHEAR WAVE PICKS ARE COMPUTER GENERATED VALUES, REQUIRING REVIEW BY YOUR STAFF.

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

CH2M / P-1 / 9420 SW H30 PD

Operator: KDV/SVAN/GEOTECH
Sounding: FILL30
Cone Used: 4CH

CPT Date/Time: 10-24-05 10:16
Location: P-1 9420SW H30PD
Job Number: CH2M334935.A1.01



Maximum Depth = 13.12 feet

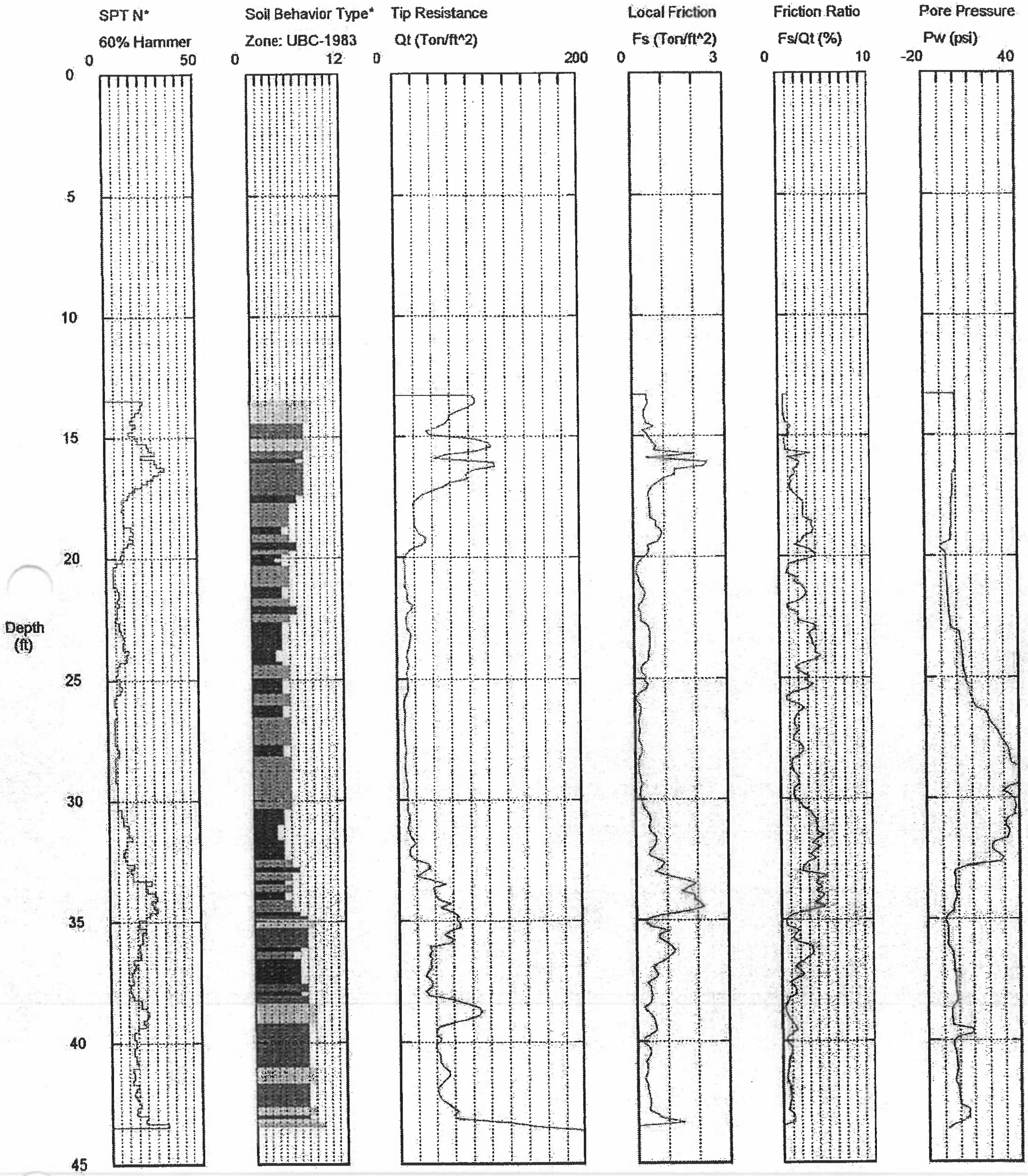
Depth Increment = 0.164 feet

THE SHEAR WAVE PICKS ARE COMPUTER GENERATED VALUES, REQUIRING REVIEW BY YOUR STAFF.

CH2M / P-1 / 9420 SW H30 PD

Operator: KDV/SVAN/GEOTECH
Sounding: FILL33
Cone Used: 4CH

CPT Date/Time: 10-24-05 11:23
Location: P-1 9420SW H30PD
Job Number: CH2M334935.A1.01



Maximum Depth = 43.64 feet

Depth Increment = 0.164 feet

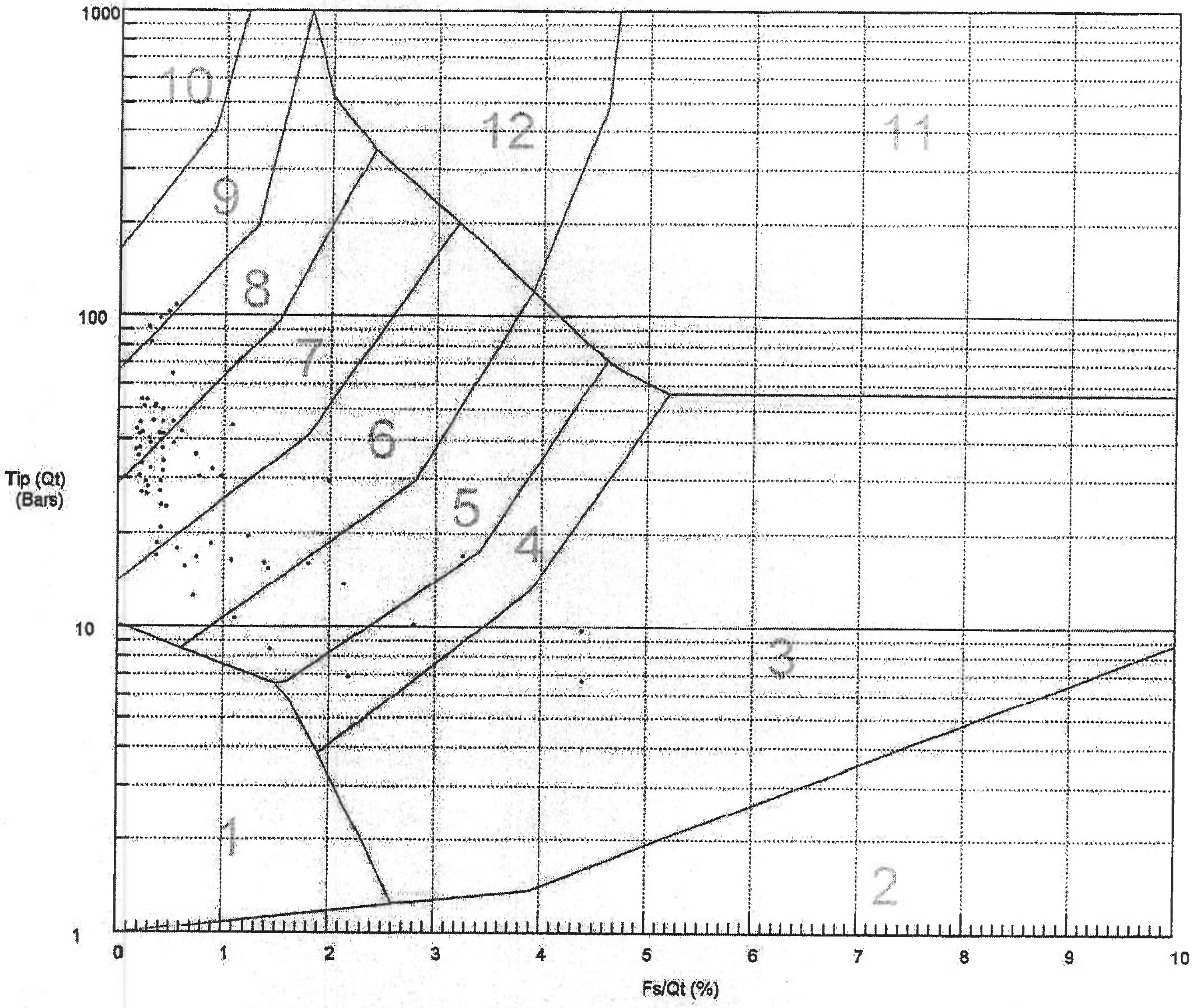
- ▨ 1 sensitive fine grained
- ▨ 2 organic material
- ■ 3 clay
- ■ 4 silty clay to clay
- ▨ 5 clayey silt to silty clay
- ■ 6 sandy silt to clayey silt
- ▨ 7 silty sand to sandy silt
- ▨ 8 sand to silty sand
- ▨ 9 sand
- ▨ 10 gravelly sand to sand
- ▨ 11 very stiff fine grained (*)
- ■ 12 sand to clayey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

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

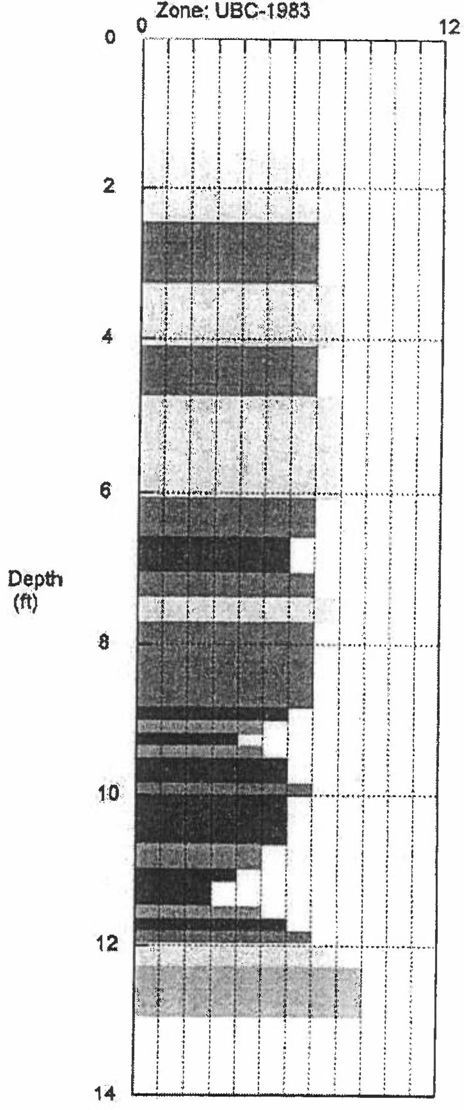
CH2M / P-1 9420 SW H30 PD

Operator:	KDV/SVAN/GEOTECH	CPT Date/Time:	10-24-05 10:16
Sounding:	FILL30	Location:	P-1 9420SW H30PD
Cone Used:	4CH	Job Number:	CH2M334935.A1.01

Classification Data:
Robertson and Campanella UBC-1983



Soil Behavior Type*
Zone: UBC-1983

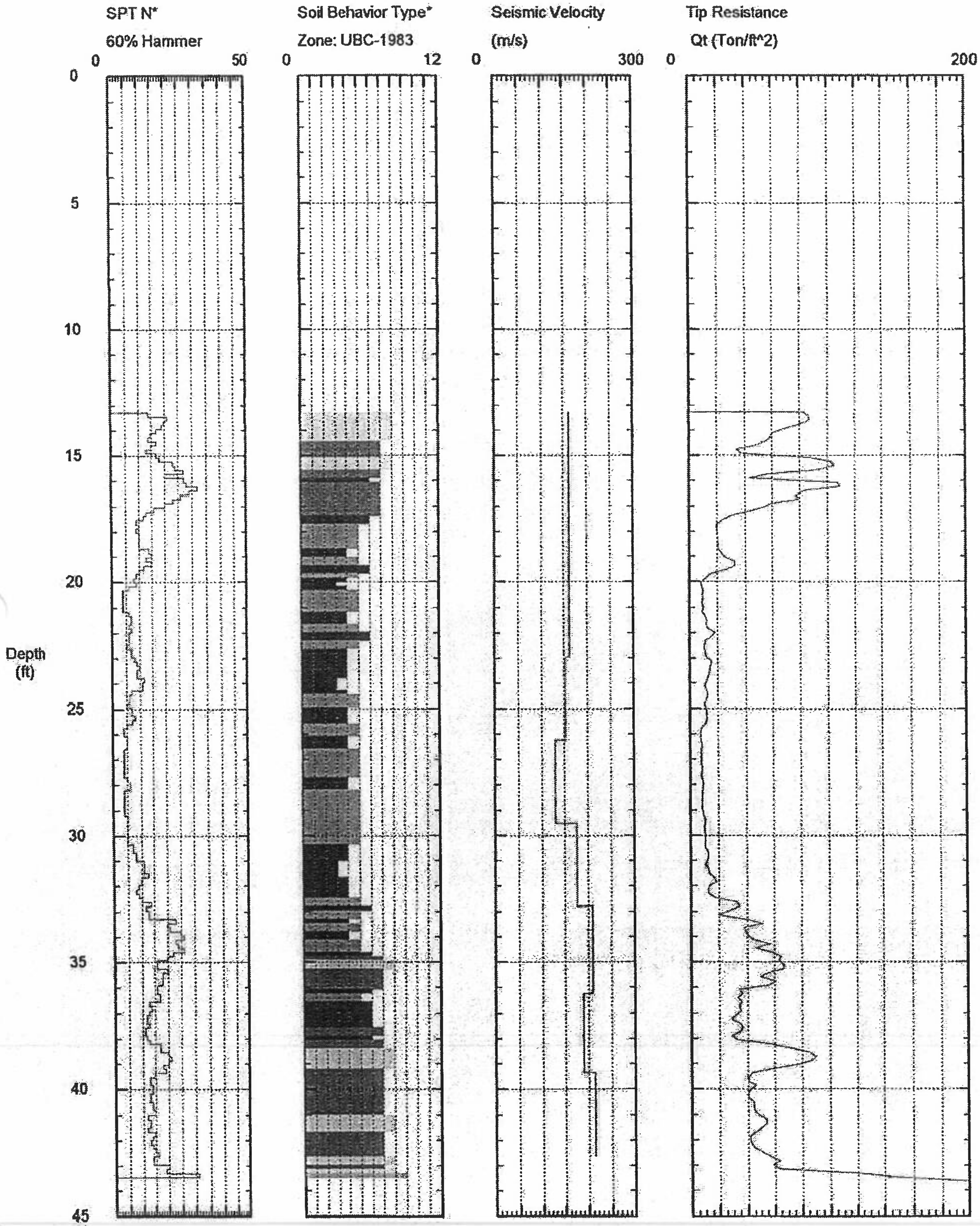


▨ 1 sensitive fine grained	■ 4 silty clay to clay	▨ 7 silty sand to sandy silt	■ 10 gravelly sand to sand
▨ 2 organic material	▨ 5 clayey silt to silty clay	▨ 8 sand to silty sand	▨ 11 very stiff fine grained (*)
■ 3 clay	■ 6 sandy silt to clayey silt	▨ 9 sand	■ 12 sand to clayey sand (*)

CH2M / P-1 / 9420 SW H30 PD

Operator: KDV/SVAN/GEOTECH
Sounding: FILL33
Cone Used: 4CH

CPT Date/Time: 10-24-05 11:23
Location: P-1 9420SW H30PD
Job Number: CH2M334935.A1.01



Maximum Depth = 43.64 feet

Depth Increment = 0.164 feet

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

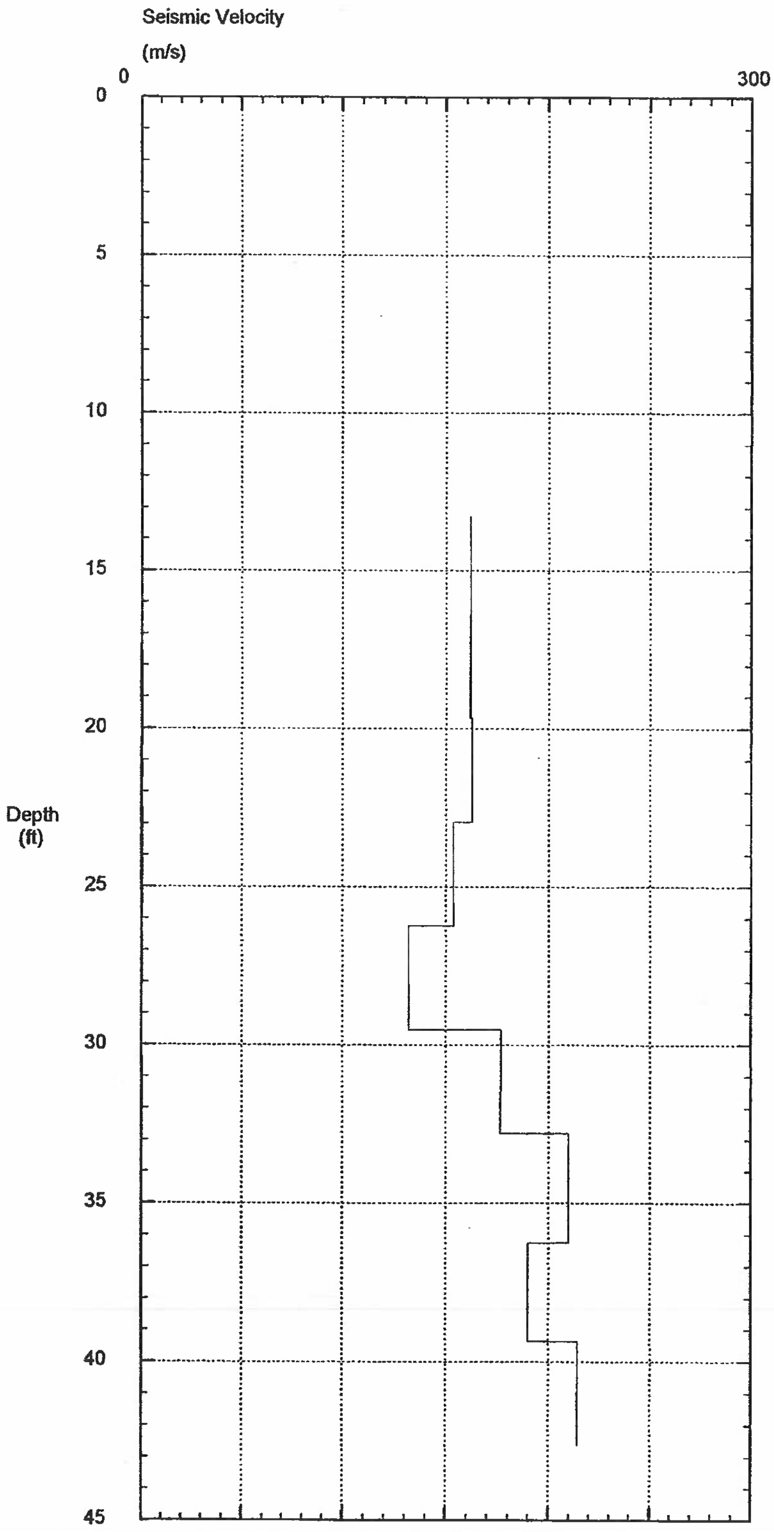
THE SHEAR WAVE PICKS ARE COMPUTER GENERATED VALUES, REQUIRING REVIEW BY YOUR STAFF.

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

CH2M / P-1 / 9420 SW H30 PD

Operator: KDV/SVAN/GEOTECH
Sounding: FILL33
Cone Used: 4CH

CPT Date/Time: 10-24-05 11:23
Location: P-1 9420SW H30PD
Job Number: CH2M334935.A1.01

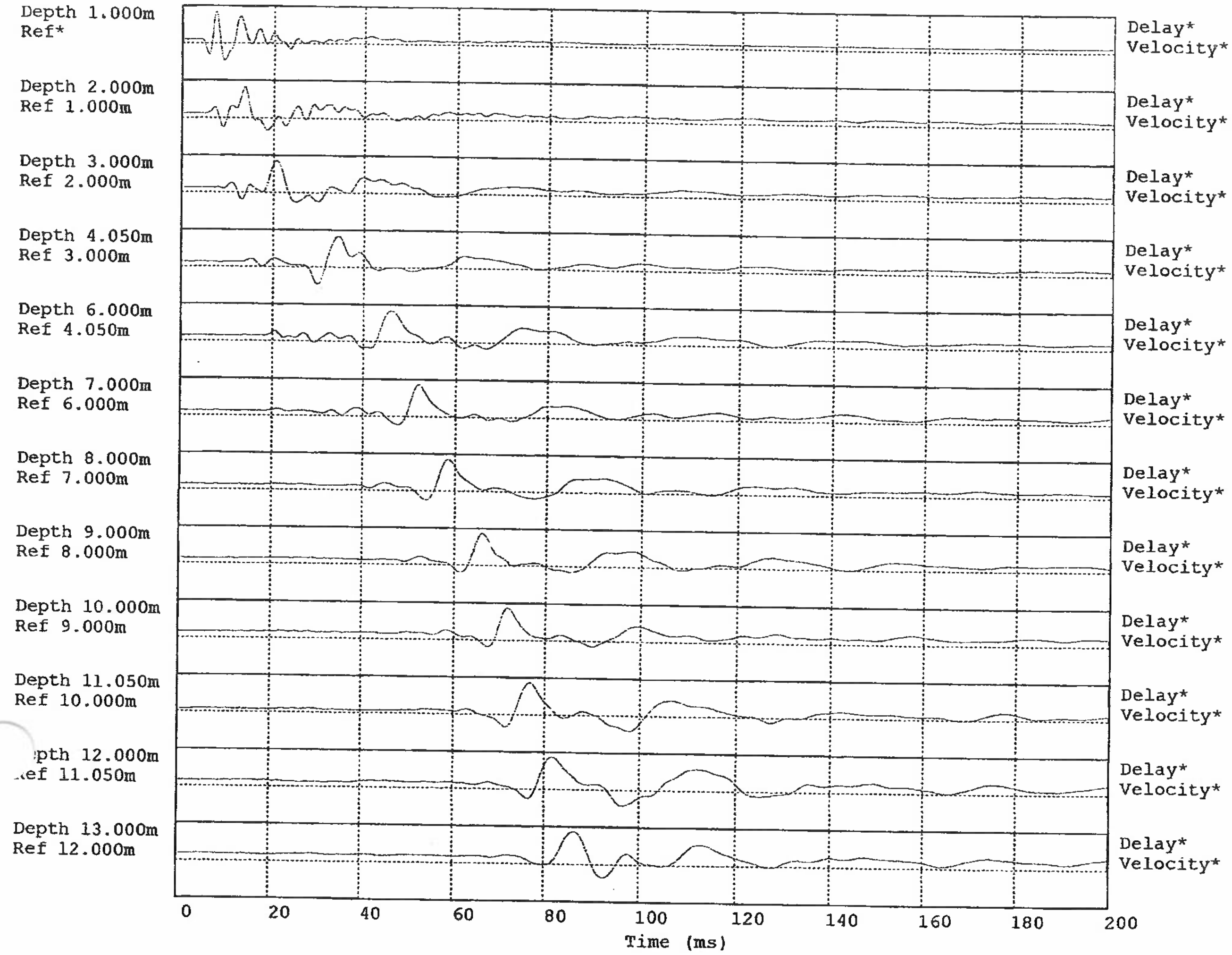


Maximum Depth = 43.64 feet

Depth Increment = 0.164 feet

THE SHEAR WAVE PICKS ARE COMPUTER GENERATED VALUES, REQUIRING REVIEW BY YOUR STAFF.

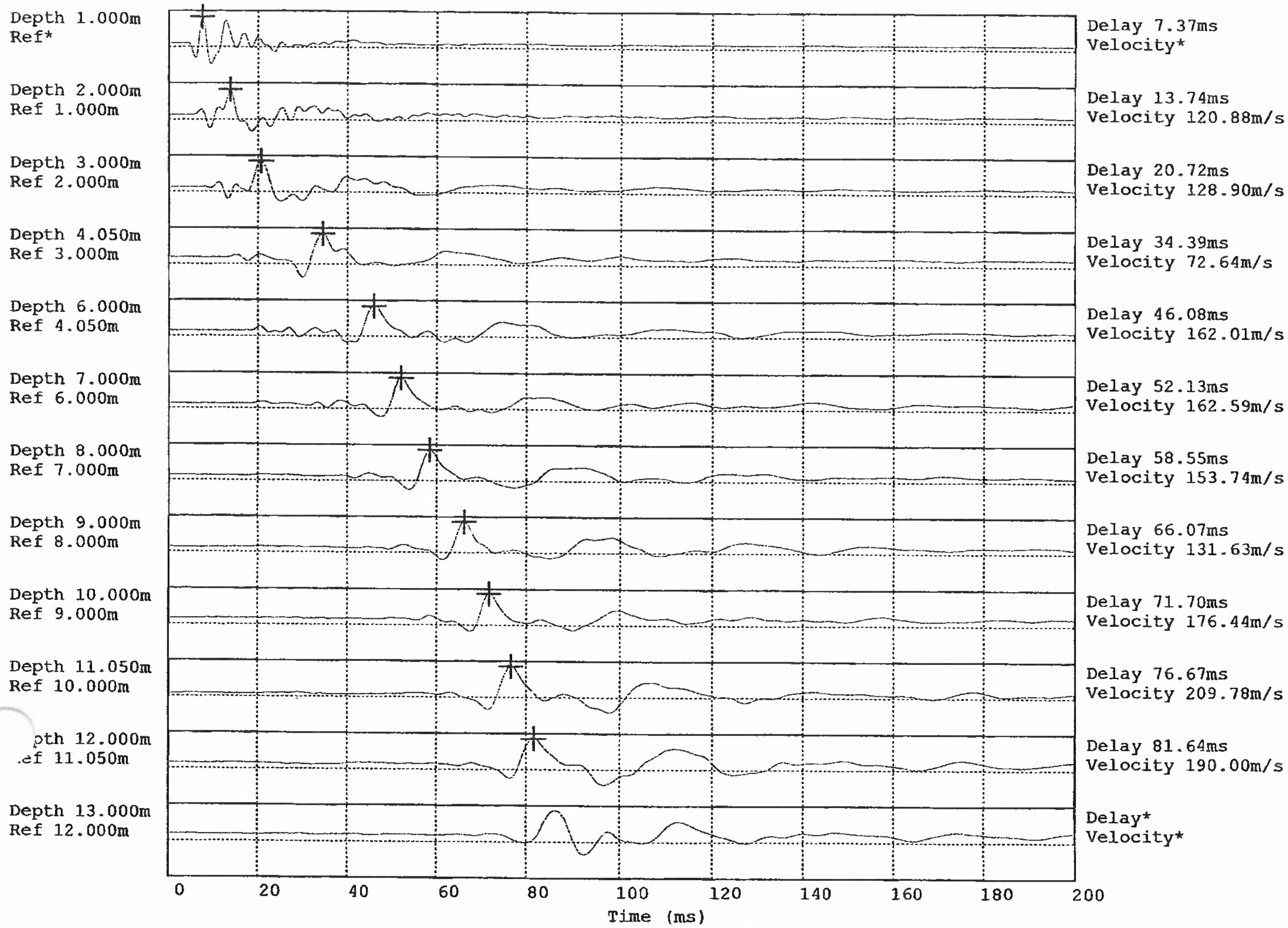
CH2M / P-1 / 9420 SW H30, PD.



Hammer to Rod String Distance 1.2 (m)
* = Not Determined

THE SHEAR WAVE PICKS ARE COMPUTER GENERATED VALUES, REQUIRING REVIEW BY YOUR STAFF.

CH2M / P-1 / 9420 SW H30, PD.



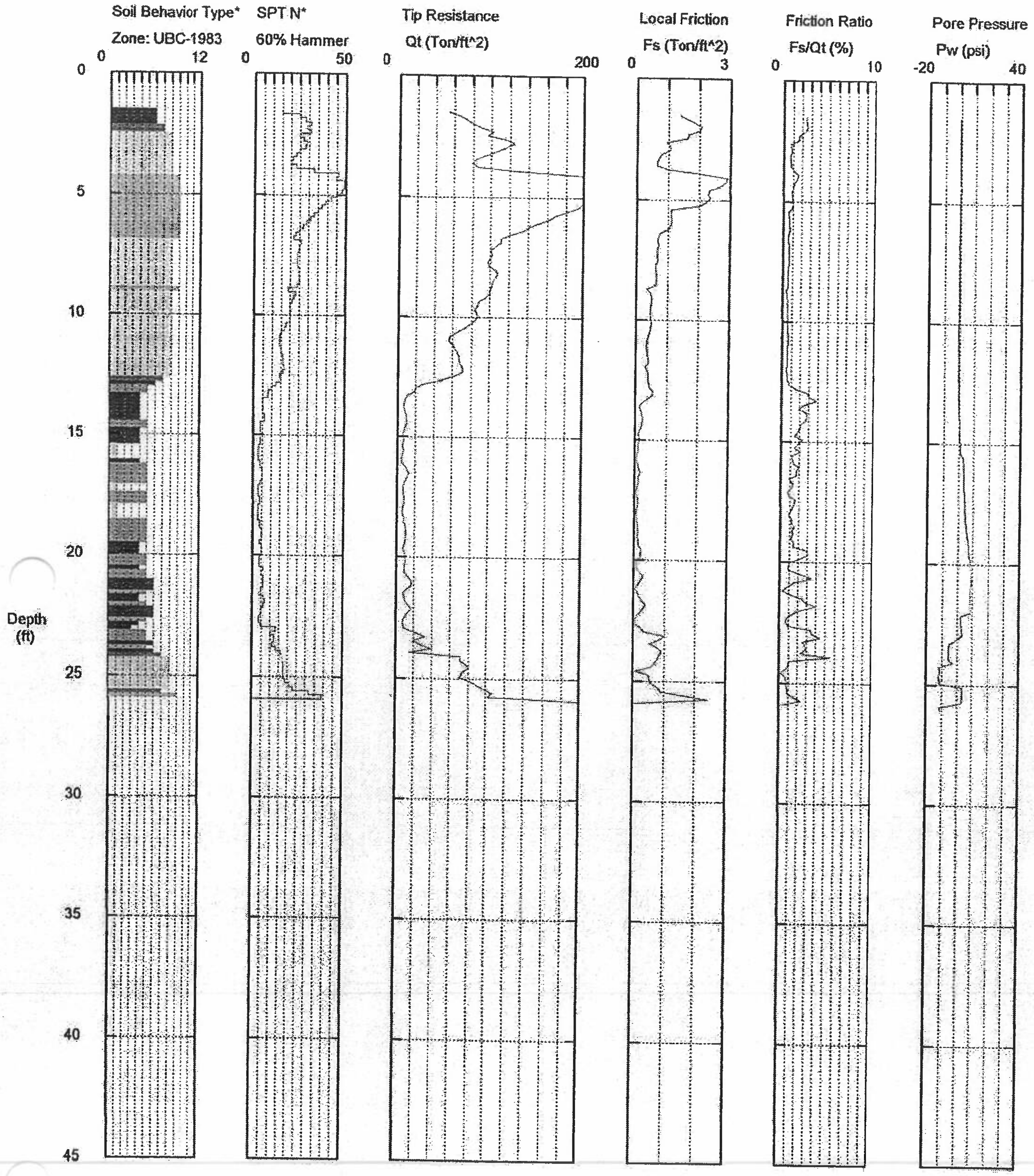
Hammer to Rod String Distance 1.2 (m)
* = Not Determined

THE SHEAR WAVE PICKS ARE COMPUTER GENERATED VALUES, REQUIRING REVIEW BY YOUR STAFF.

CH2M / P-2 / 9420 SW H30 PD

Operator: KDV/SVAN/GEOTECH
Sounding: FILL34
Cone Used: 4CH

CPT Date/Time: 10-24-05 13:43
Location: P-2 9420SW H30PD
Job Number: CH2M334935.A1.01



Maximum Depth = 26.08 feet

Depth Increment = 0.164 feet

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

THE SHEAR WAVE PICKS ARE COMPUTER GENERATED VALUES, REQUIRING REVIEW BY YOUR STAFF.

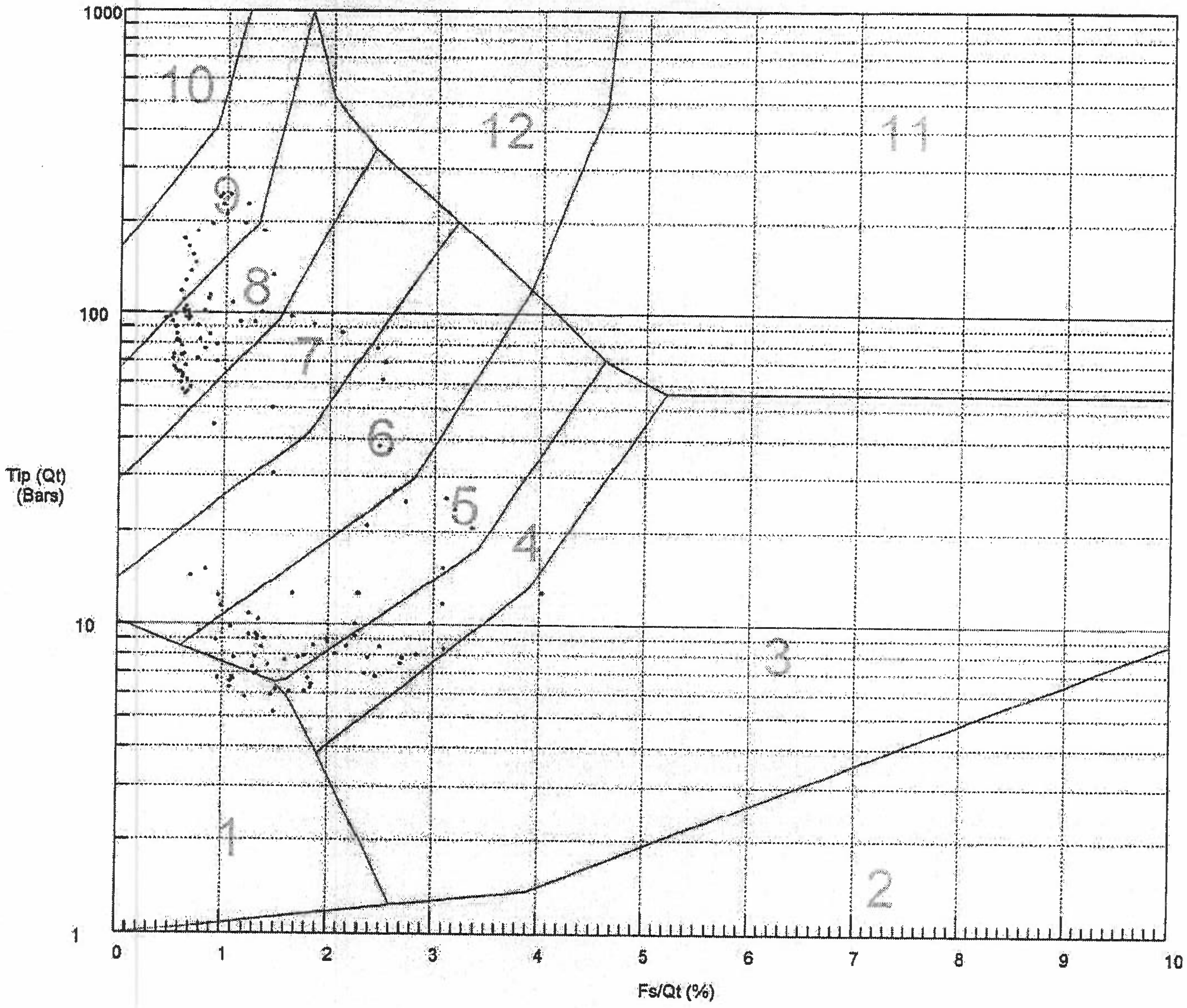
*Soil behavior type and SPT based on data from UBC-1983

CH2M / P-2 , 9420 SW H30 PD

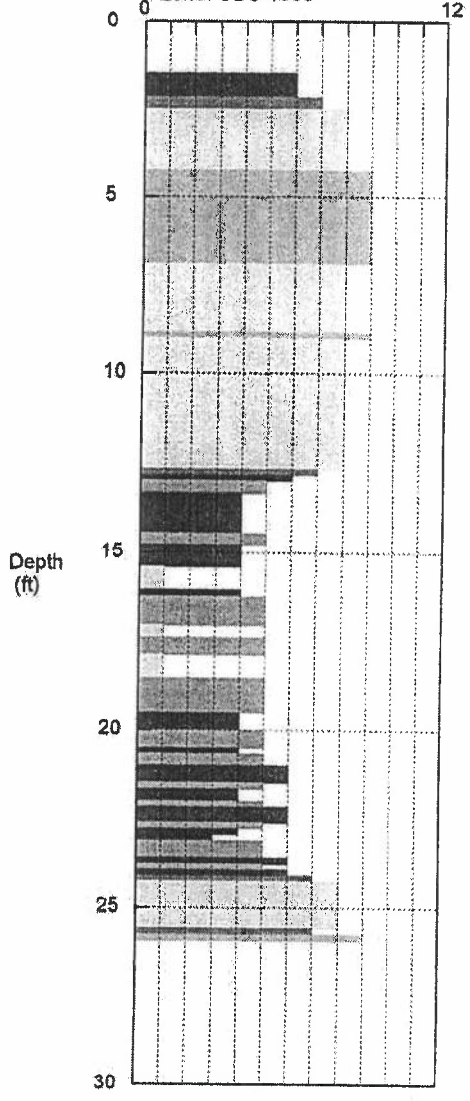
Operator: KDV/SVAN/GEOTECH
Sounding: FILL34
Cone Used: 4CH

CPT Date/Time: 10-24-05 13:43
Location: P-2 9420SW H30PD
Job Number: CH2M334935.A1.01

Classification Data:
Robertson and Campanella UBC-1983



Soil Behavior Type*
Zone: UBC-1983

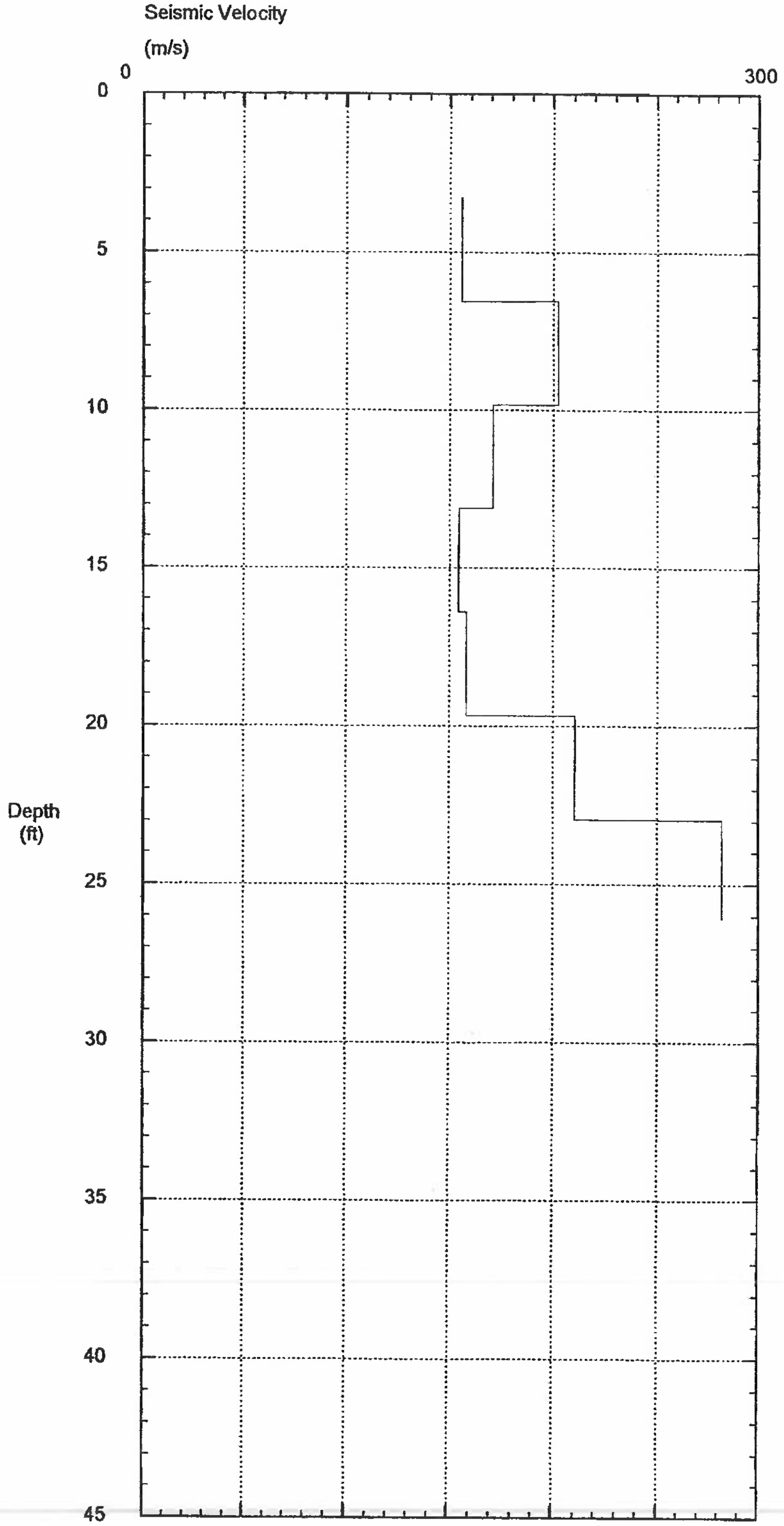


▨ 1 sensitive fine grained
▨ 2 organic material
■ 3 clay
■ 4 silty clay to clay
▨ 5 clayey silt to silty clay
■ 6 sandy silt to clayey silt
▨ 7 silty sand to sandy silt
▨ 8 sand to silty sand
▨ 9 sand
▨ 10 gravelly sand to sand
▨ 11 very stiff fine grained (*)
■ 12 sand to clayey sand (*)

CH2M / P-2 / 9420 SW H30 PD

Operator: KDV/SVAN/GEOTECH
Sounding: FILL34
Cone Used: 4CH

CPT Date/Time: 10-24-05 13:43
Location: P-2 9420SW H30PD
Job Number: CH2M334935.A1.01

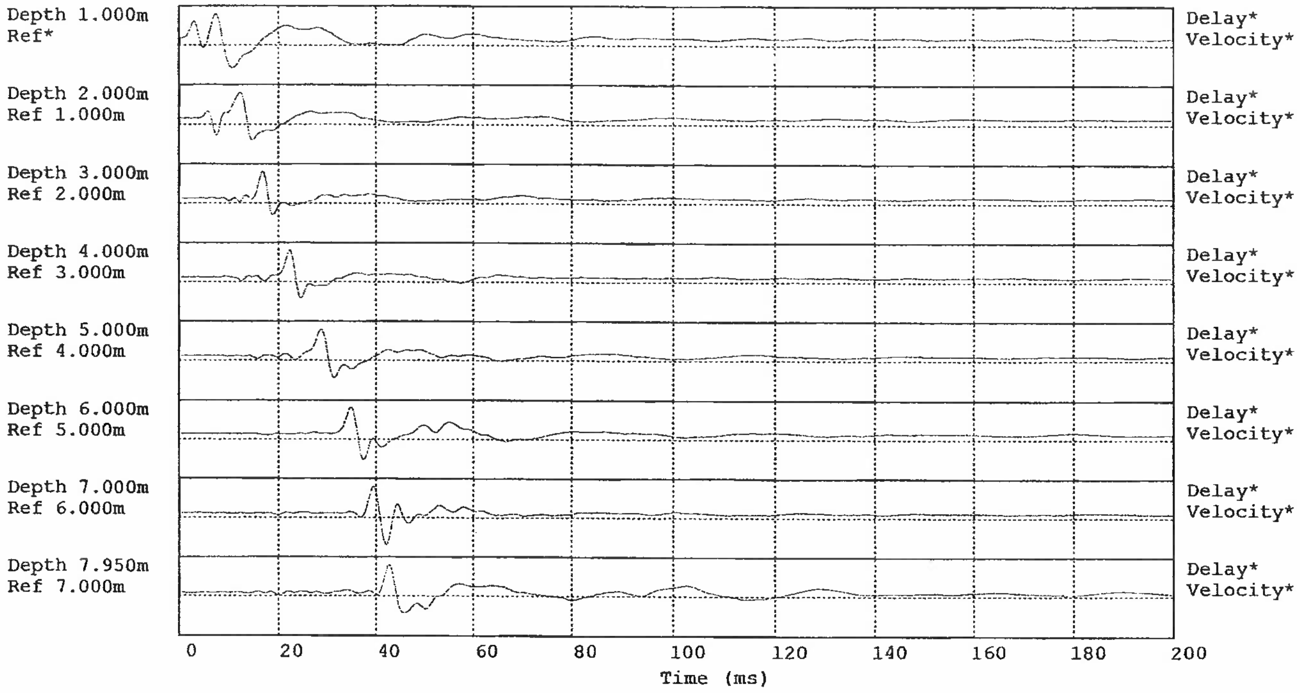


Maximum Depth = 26.08 feet

Depth Increment = 0.164 feet

THE SHEAR WAVE PICKS ARE COMPUTER GENERATED VALUES, REQUIRING REVIEW BY YOUR STAFF.

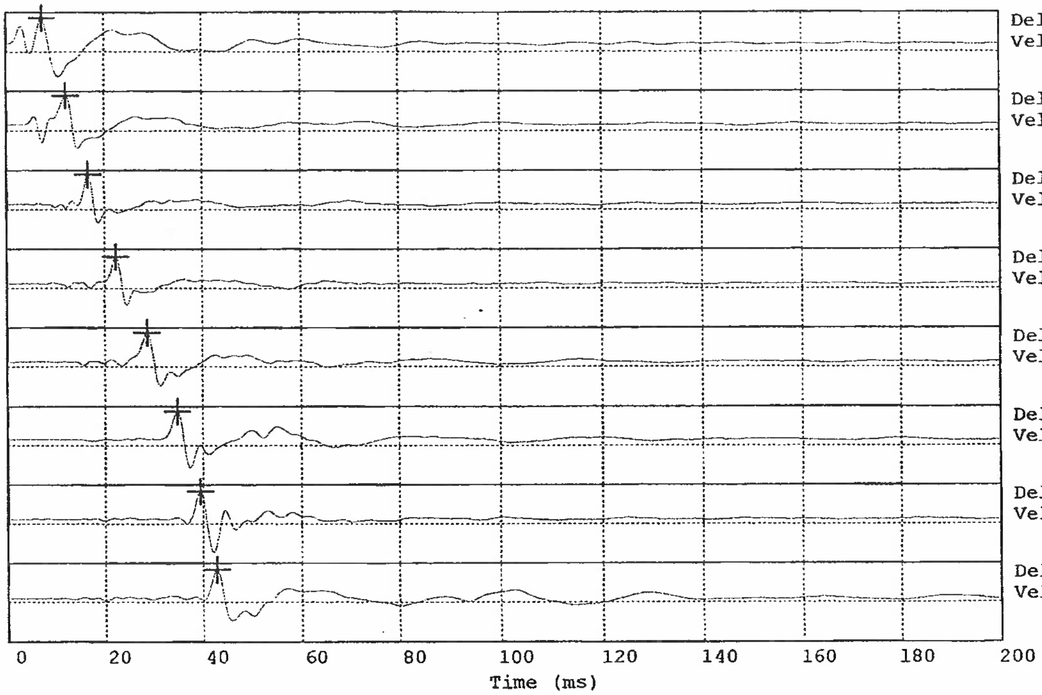
CH2M / P-2 / 9420 SW H30, PD.



Hammer to Rod String Distance 1.2 (m)
* = Not Determined

CH2M / P-2 / 9420 SW H30, PD.

Depth 1.000m	Ref*	Delay 7.45ms	Velocity*
Depth 2.000m	Ref 1.000m	Delay 12.40ms	Velocity 155.68m/s
Depth 3.000m	Ref 2.000m	Delay 16.84ms	Velocity 202.04m/s
Depth 4.000m	Ref 3.000m	Delay 22.37ms	Velocity 171.11m/s
Depth 5.000m	Ref 4.000m	Delay 28.61ms	Velocity 154.60m/s
Depth 6.000m	Ref 5.000m	Delay 34.79ms	Velocity 158.26m/s
Depth 7.000m	Ref 6.000m	Delay 39.46ms	Velocity 210.41m/s
Depth 7.950m	Ref 7.000m	Delay 42.78ms	Velocity 282.20m/s



Hammer to Rod String Distance 1.2 (m)
* = Not Determined

THE SHEAR WAVE PICKS ARE COMPUTER GENERATED VALUES, REQUIRING REVIEW BY YOUR STAFF.

APPENDIX C

Laboratory Test Results

Laboratory Tests Results From Samples Taken From Borings B-1, B-2, and B-3

FEI Testing & Inspection, Inc.
Valero LP
Project 2056007-510

Table 1. Bulk Densities

Sample Number*	Sample Depth (feet)	Water Content (percent)	Moist Bulk Density (pcf)	Dry Density (pcf)
B-1/SH-6	26.5 – 28.5	53.4	103.6	67.5
B-1/SH-10	41.5 – 43.5	37.8	115.0	83.5

Table 2. Percent Fines & Water Content

Sample Number*	Sample Depth (feet)	Percent Fines	Water Content (percent)
B-1/SS-12	50.0 – 51.5	17.9	41.8
B-2/SS-6	30.0 – 31.5	53.2	36.6

Table 3. Natural Water Contents

Sample Number*	Sample Depth (feet)	Natural Water Content (percent)
B-1/SS-5	25.0 – 26.5	48.3
B-1/SS-7	28.5 – 30.0	55.6
B-1/SS-11	43.5 – 45.0	27.8
B-2/SS-4	20.0 – 21.5	32.3
B-3/SS-3	15.0 – 16.5	24.0
B-3/SS-5	25.0 – 26.5	28.2

*FEI Sample No. 3073

Table 4. Sieve Analysis

Sieve Size	Percent Passing		
	B-2/SS-2	B-2/SS-7	B-3/SS-1
1/2"	100.0		100.0
#4	99.7		99.7
#10	99.4	100.0	99.2
#20	96.0	99.8	95.9
#40	56.5	99.3	49.5
#60	18.5	98.9	11.7
#100	10.6	97.3	5.8
#200	6.8	85.0	3.6

*FEI Sample No. 3073

FEI
TESTING & INSPECTION, INC.

SHELBY TUBE DESCRIPTIONS

BOREHOLE NO.: B-1

PROJECT NAME: Valero LP

PROJECT NO.: 2056007-510

SAMPLE: SH-10

DEPTH: 41.5 - 43.5

DATE SAMPLED:

DATE PUSHED: 11-9-05

COMMENTS: FEI sample # 3073

Sample Description	*Sample Testing*
No recovery	
SAND, soft to medium stiff moist, non plastic	
Brown sandy SILT, medium stiff moist	
Brown - Grey Clayey SILT Stiff, Damp to moist	Consolidation, Atterberg, & Bulk density samples obtained

FEI
TESTING & INSPECTION, INC.

SHELBY TUBE DESCRIPTIONS

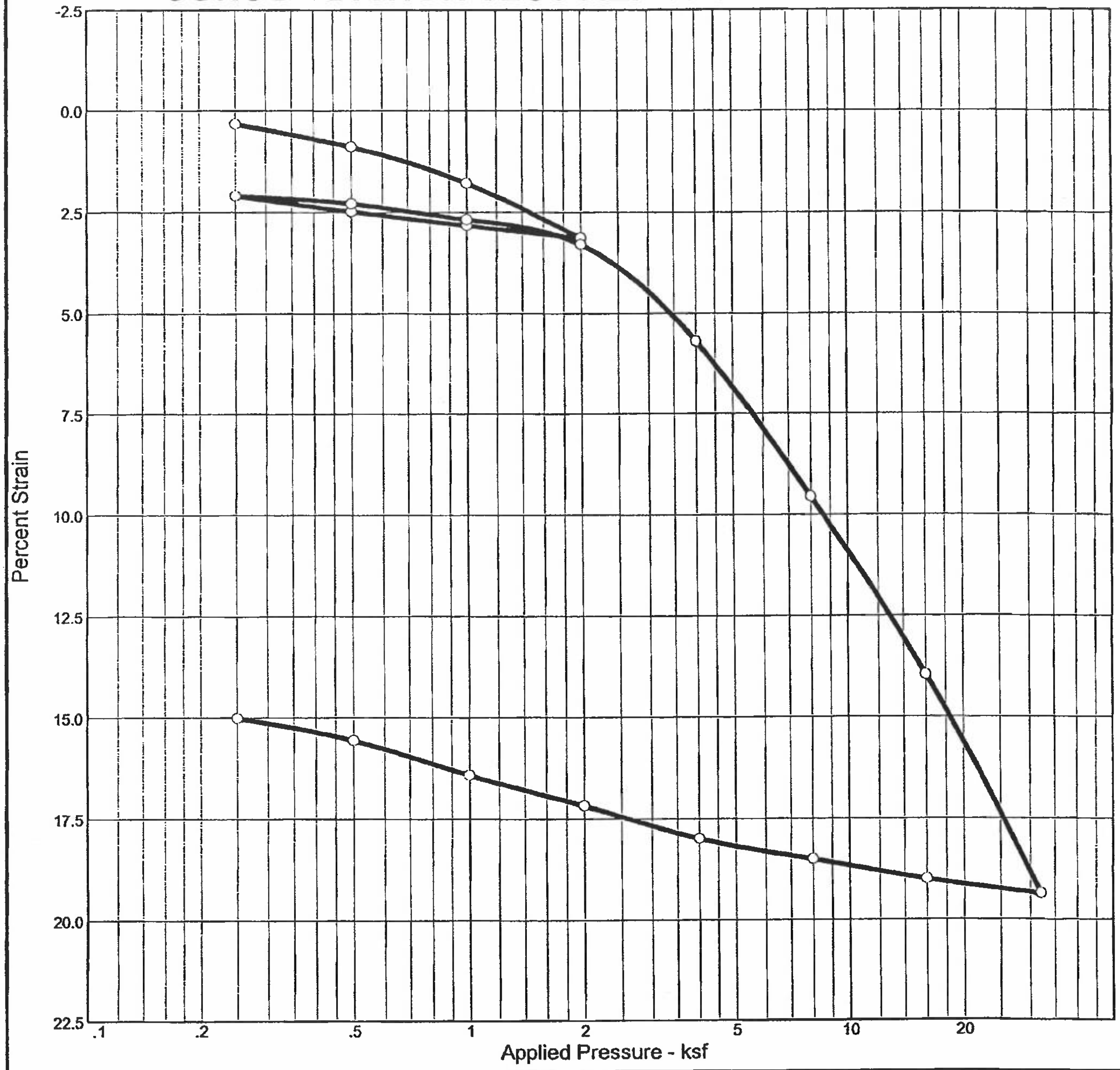
BOREHOLE NO.: B-1

PROJECT NAME: Valero LP
SAMPLE: SH-6
DATE SAMPLED:
COMMENTS: FEI sample # 3073

PROJECT NO.: 2056007-S10
DEPTH: 26.5 - 28.5
DATE PUSHED: 11-9-05

Sample Description			Sample Testing
No Recovery			
SAND, Trace clayey silt soft to medium stiff Moist			
Grey clayey SILT medium stiff Damp to moist			
			Consolidation, Atterberg & Bulk density samples obtained

CONSOLIDATION TEST REPORT - ASTM D2435



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (ksf)	P_c (ksf)	C_c	C_r	Swell Press. (ksf)	Swell %	e_o
Sat.	Moist.											
86.8 %	37.8 %	76.8	44	9	2.65		4.74	0.39	0.04			1.153

MATERIAL DESCRIPTION	USCS	AASHTO
Brown - Grey clayey SILT	ML	

Project No. 334935.A1.01 **Client:** CH2M Hill
Project: Valero L.P.

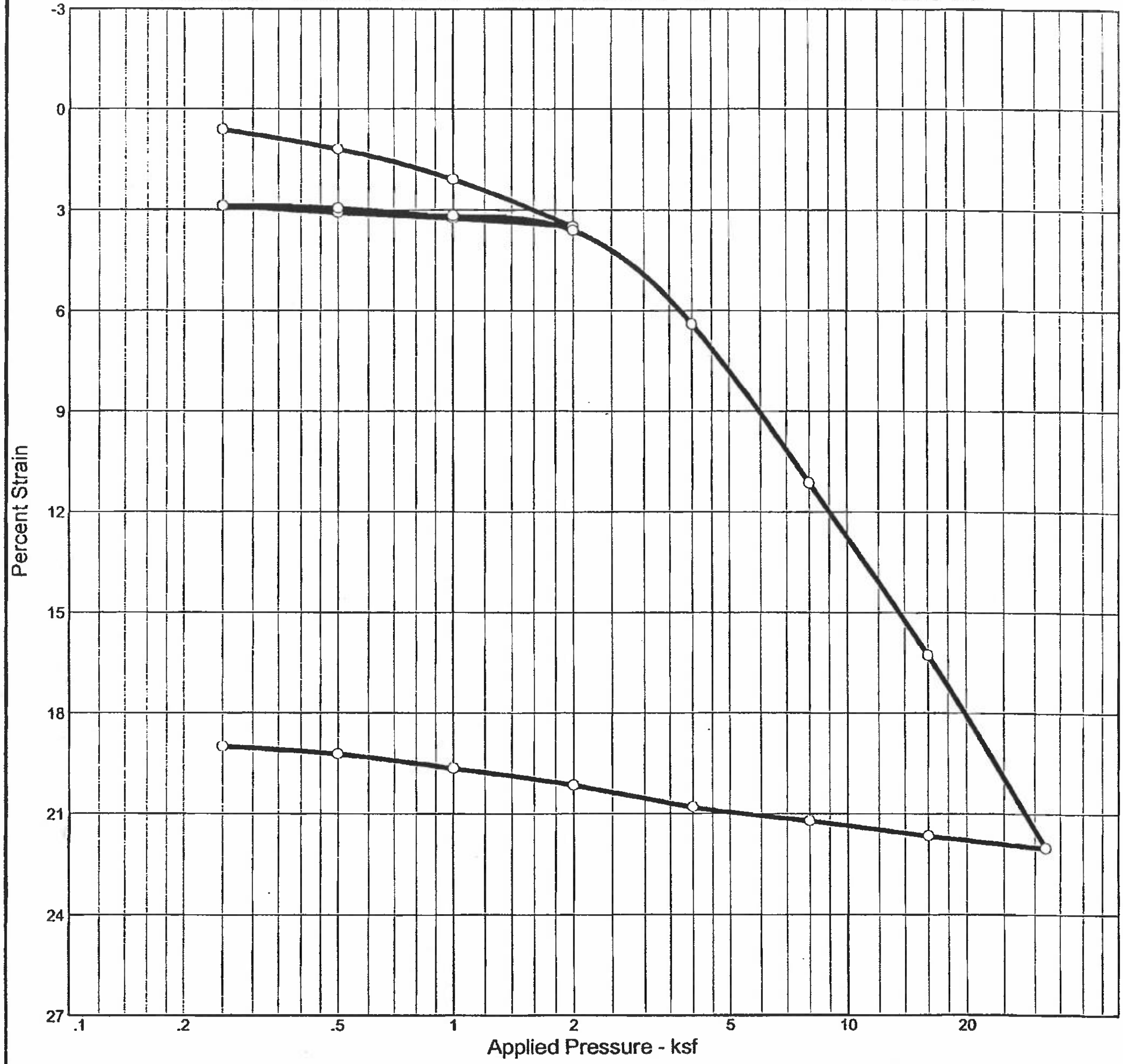
Source: 3073 **Sample No.:** B-1/SH-10 **Elev./Depth:** 41.5-43.5

Remarks:

CONSOLIDATION TEST REPORT - ASTM D2435
FEI Testing & Inspection, Inc.
Corvallis, OR

Figure 1

CONSOLIDATION TEST REPORT - ASTM D2435



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (ksf)	P_c (ksf)	C_c	C_r	Swell Press. (ksf)	Swell %	e_o
Sat.	Moist.											
104.3 %	53.4 %	69.4	63	27	2.65		4.72	0.45	0.03			1.356

MATERIAL DESCRIPTION	USCS	AASHTO
Grey clayey SILT	MH	

Project No. 334935.A1.01 **Client:** CH2M Hill

Project: Valero L.P.

Source: 3073 **Sample No.:** B-1/SH-6 **Elev./Depth:** 26.5-28.5

Remarks:

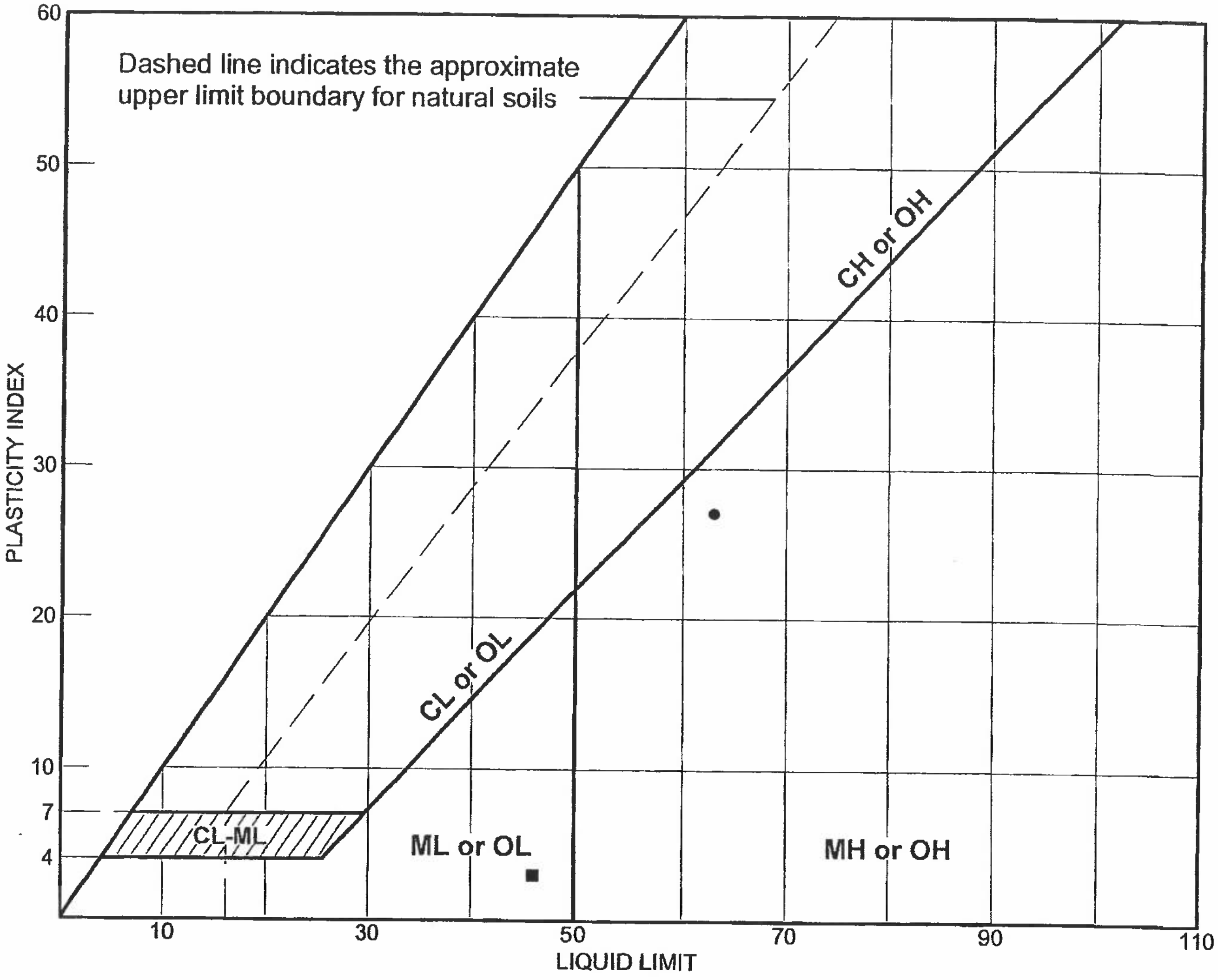
CONSOLIDATION TEST REPORT - ASTM D2435

FEI Testing & Inspection, Inc.

Corvallis, OR

Figure 2

ATTERBERG LIMITS REPORT - ASTM D4318



SOIL DATA

SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●	3073	B-1/SH-6	26.5-28.5	53.4	36	63	27	MH
■	3073	B-1/SS-8	35.0-36.5	46.5	43	46	3	ML
▲	3073	B-1/SS-9	40.0-41.5	33.66	NP	NV	NP	NP

ATTERBERG LIMITS REPORT - ASTM D4318

FEI Testing & Inspection, Inc.

Corvallis, OR

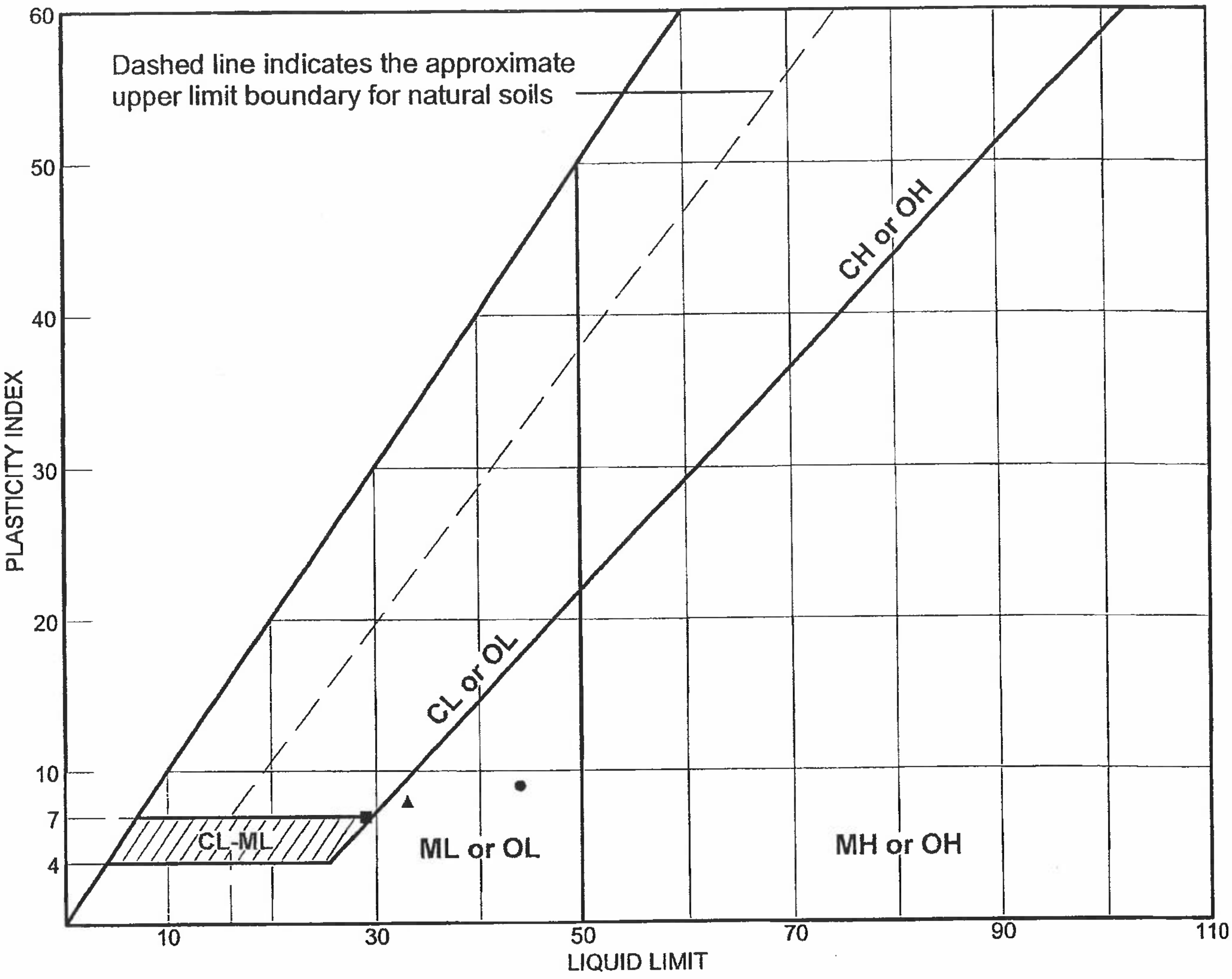
Client: CH2M Hill

Project: Valero L.P.

Project No.: 334935.A1.01

Figure 3

ATTERBERG LIMITS REPORT - ASTM D4318



SOIL DATA

SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●	3073	B-1/SH-10	41.5-43.5	37.8	35	44	9	ML
■	3073	B-1/SS-11	53.5-45.0	27.8	22	29	7	CL-ML
▲	3073	B-2/SS-3	15.0-16.5	32.8	25	33	8	ML

ATTERBERG LIMITS REPORT - ASTM D4318

FEI Testing & Inspection, Inc.

Corvallis, OR

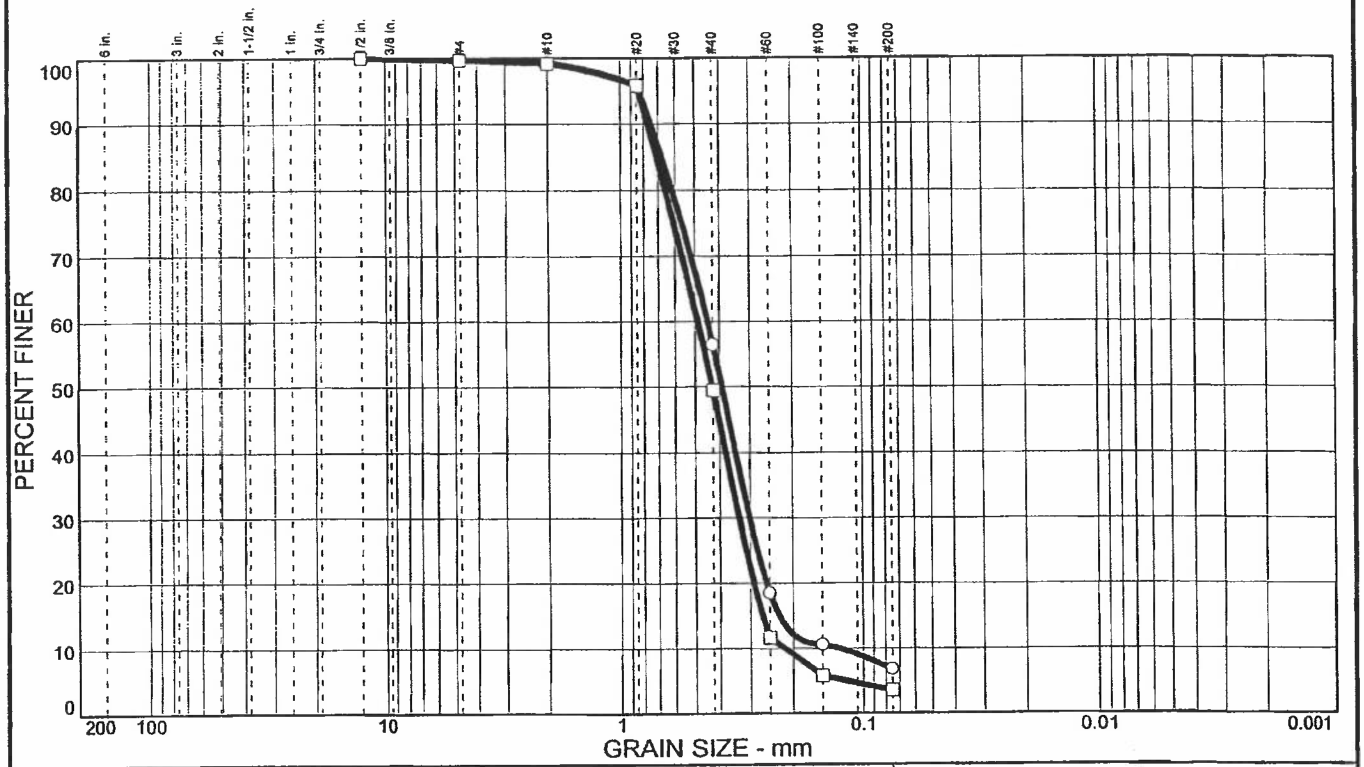
Client: CH2M Hill

Project: Valero L.P.

Project No.: 334935.A1.01

Figure 4

Particle Size Distribution Report



	% COBBLES	% GRAVEL		% SAND			% FINES	
		CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
○	0.0	0.0	0.3	0.3	42.9	49.7	6.8	
□	0.0	0.0	0.3	0.5	49.7	45.9	3.6	

☒	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○			0.678	0.446	0.391	0.303	0.228	0.125	1.64	3.55
□			0.711	0.488	0.428	0.334	0.268	0.216	1.06	2.26

MATERIAL DESCRIPTION	USCS	AASHTO
○ Poorly graded sand with silt	SP-SM	A-3
□ Poorly graded sand	SP	A-1-b

Project No. 334935.A1.01 **Client:** CH2M Hill

Project: Valero L.P.

○ **Source:** 3073 **Sample No.:** B-2/SS-2 **Elev./Depth:** 10.0-11.5

□ **Source:** 3073 **Sample No.:** B-3/SS-1 **Elev./Depth:** 5.0-6.0

Remarks:

○ Water content 17.3%

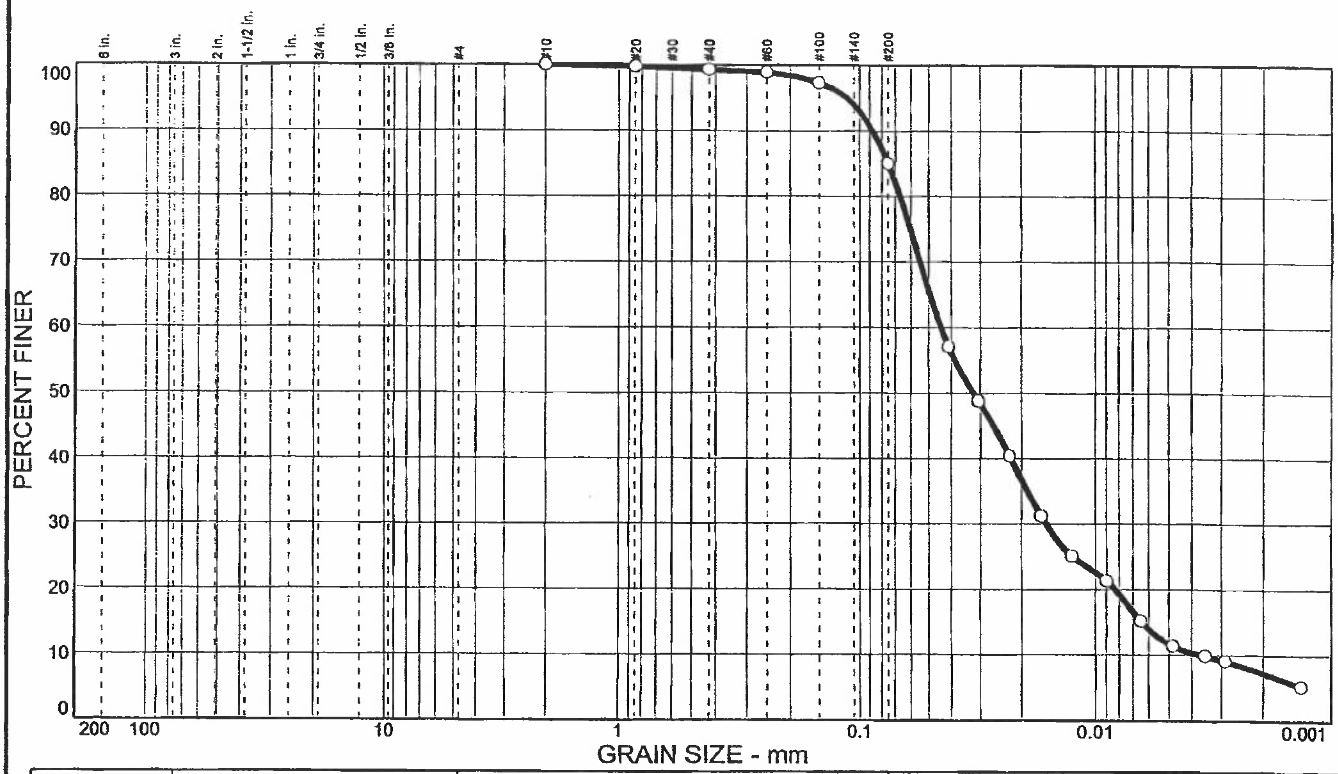
□ Water content 22.0%

Particle Size Distribution Report

FEI Testing & Inspection, Inc.
Corvallis, OR

Figure 5

Particle Size Distribution Report



	% COBBLES	% GRAVEL		% SAND			% FINES	
		CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
○	0.0	0.0	0.0	0.0	0.7	14.3	73.3	11.7

☒	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○			0.0750	0.0443	0.0320	0.0159	0.0064	0.0036	1.58	12.24

MATERIAL DESCRIPTION	USCS	AASHTO
○ Silt	ML	A-4(0)

Project No. 334935.A1.01 **Client:** CH2M Hill

Project: Valero L.P.

○ **Source:** 3073 **Sample No.:** B-2/SS-7 **Elev./Depth:** 35.0-36.5

Remarks:

○ Water content 39.6%

Particle Size Distribution Report

FEI Testing & Inspection, Inc.

Corvallis, OR

Figure 6

Laboratory Tests Results From Samples Taken From Borings B-4 and B-5

Table 1. Percent Fines & Water Content

Sample Number*	Sample Depth (feet)	Percent Fines	Water Content (percent)
B-4/SS-4	20.0 – 21.5	69.1	40.9
B-4/SS-8	40.0 – 41.5	72.4	39.9
B-4/SS-10	50.0 – 51.5	17.5	26.6
B-5/SS-5	25.0 – 26.5	64.5	48.6
B-5/SS-7	35.0 – 36.5	86.6	41.4
B-5/SS-9	40.0 – 41.5	74.5	36.8

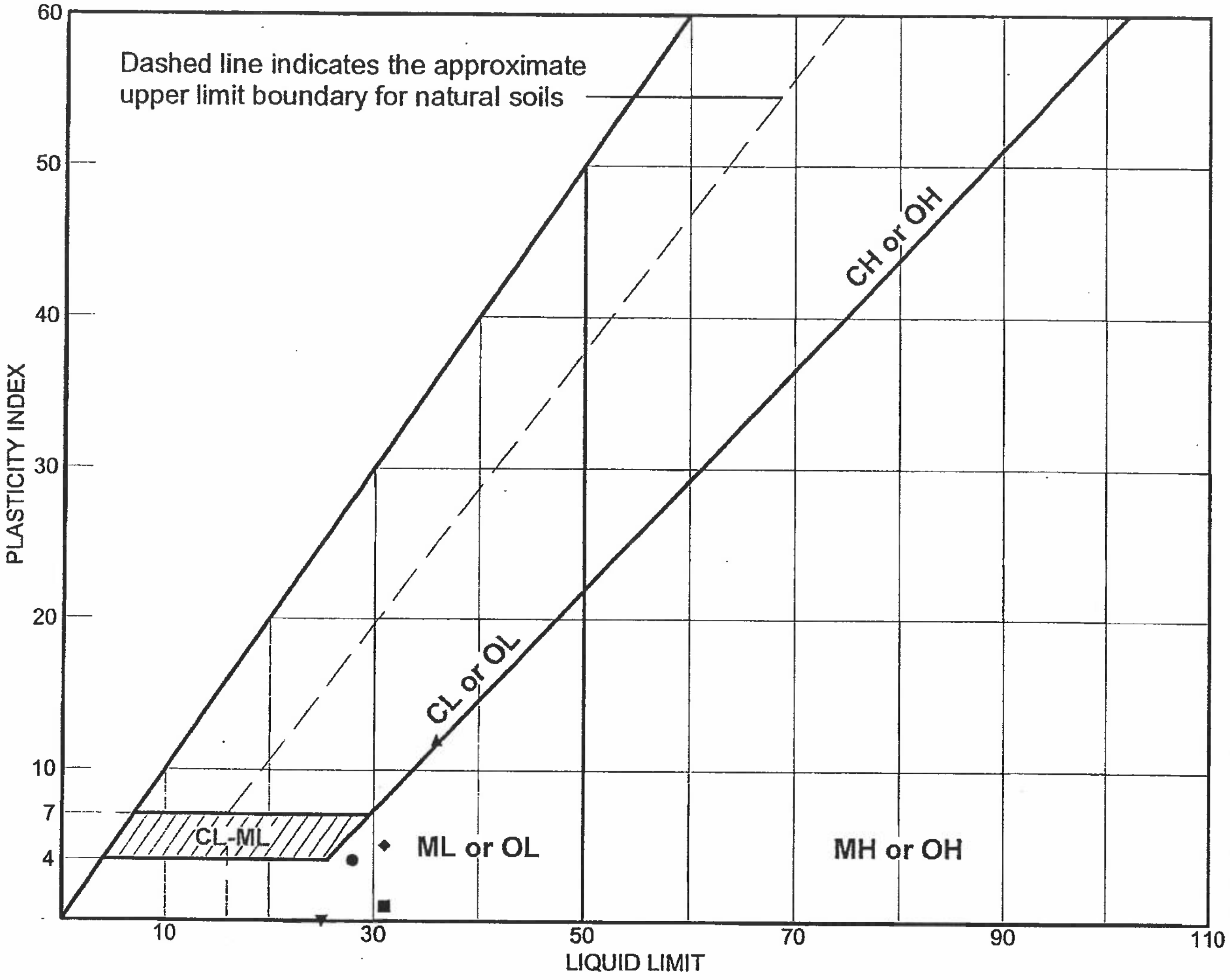
Table 2. Sieve Analysis

Sieve Size	Percent Passing	
	B-4/SS-6*	B-5/SS-10*
1 1/2"		100.0
1"	100.0	74.9
3/4"	89.6	69.2
1/2"	76.5	68.3
#4	64.8	61.9
#10	59.2	58.5
#20	55.1	55.8
#40	52.4	53.6
#60	50.5	47.7
#100	48.9	37.4
#200	45.3	29.6

---

*FEI Sample No. 3142

ATTERBERG LIMITS REPORT - ASTM D4318



SOIL DATA								
SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●	3142	B-4/SS-6	30.0-31.5	24.7	24	28	4	ML
■	3142	B-5/SS-5	25.0-26.58	48.6	30	31	1	ML
▲	3142	B-5/SS-6	30.0-31.5	34.8	24	36	12	CL-ML
◆	3142	B-5/SS-7	35.0-36.5	41.4	26	31	5	ML
▼	3142	B-5/SS-9	40.0-41.5	36.8	26	25	NP	NP

ATTERBERG LIMITS REPORT - ASTM D4318

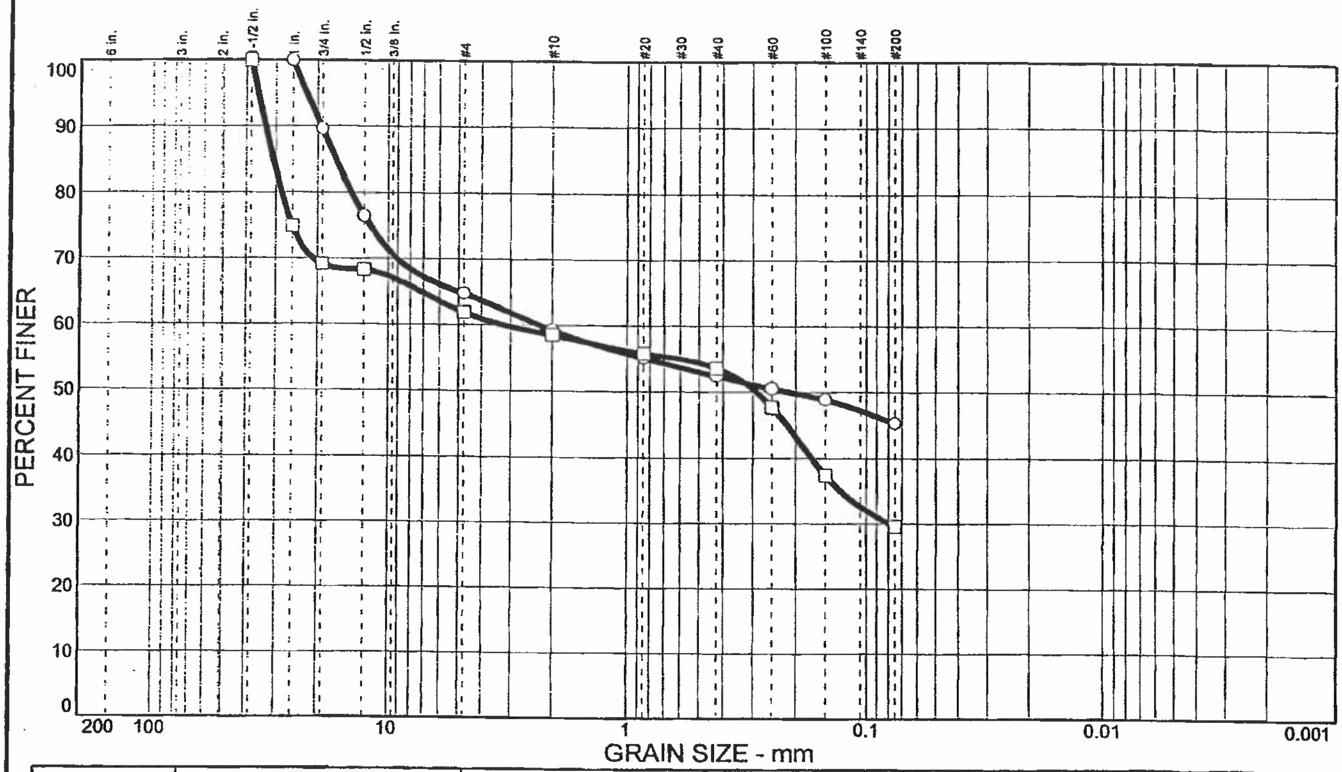
FEI Testing & Inspection, Inc.
Corvallis, OR

Client: CH2M Hill
Project: Valero L.P.

Project No.: 334935.A1.01

Figure 1

Particle Size Distribution Report



	% COBBLES	% GRAVEL		% SAND			% FINES	
		CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
○	0.0	10.4	24.8	5.6	6.8	7.1	45.3	
□	0.0	30.8	7.3	3.4	4.9	24.0	29.6	

☒	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○			16.7	2.28	0.211					
□			30.8	3.23	0.290	0.0789				

MATERIAL DESCRIPTION	USCS	AASHTO
○ Silty gravel with sand	GM	A-4(0)
□ Silty gravel with sand	GM	A-2-4(0)

Project No. 334935.A1.01 **Client:** CH2M Hill

Project: Valero L.P.

○ **Source:** 3142 **Sample No.:** B-4/SS-6 **Elev./Depth:** 30.0-31.5

□ **Source:** 3142 **Sample No.:** B-5/SS-10 **Elev./Depth:** 42.5-44.0 feet

Remarks:

○ Water Content:24.7%

□ Water Content:21.1%

Particle Size Distribution Report

FEI Testing & Inspection, Inc.

Corvallis, OR

Fig. No: 2C

ATTACHMENT 1

Field Explorations

ATTACHMENT 1
FIELD EXPLORATIONS

Site subsurface conditions were explored on March 20 and 21, 2019. The exploration program included four cone penetration test (CPT) soundings (CPT-1, CPT-1a, CPT-2, and CPT-3), one hand-auger boring (HB-1), and two test pits (TP-1 and TP-2). The explorations were advanced at the approximate locations shown on Figure 2. CPT-1 was advanced 31 feet (ft) below ground surface (bgs), CPT-1a 30 ft bgs, CPT-2 47 ft bgs, and CPT-3 65 ft bgs. Boring HB-1 was advanced 6 ft bgs, and test pits TP-1 and TP-2 were advanced 2 ft bgs and 4.5 ft bgs respectively. The exploration locations were selected using existing infrastructure. Ground surface elevations at the exploration locations were not determined.

The CPT soundings were advanced by Oregon Geotechnical Explorations of Keizer, Oregon, subcontracted by Landau Associates, Inc. (LAI). The hand auger exploration was advanced by LAI personnel, and the test pits were excavated by Howard's Construction and Excavating of Olympia, Washington, subcontracted by LAI.

Before CPT soundings were advanced, the exploration locations were pre-cored/excavated by Penhall of Portland, Oregon, subcontracted by LAI. Vacuum extraction was then used to pre-excavate the exploration locations to approximately 7 ft bgs. An excavator, contracted by Norwest Engineering, completed the extraction.

The field exploration program was coordinated and monitored by LAI personnel, who also obtained representative soil samples, maintained a detailed record of the subsurface soil and groundwater conditions observed, and described the soil encountered by visual and textural examination. Each representative soil type was described using the soil classification system shown on Figure 1-1, in general accordance with ASTM International (ASTM) standard test method D2488, *Standard Recommended Practice for Description of Soils (Visual-Manual Procedure).*

Summary logs of the hand-auger boring and test pits are presented on Figures 1-2, 1-3, and 1-4. These logs represent LAI's interpretation of the subsurface conditions identified during the field exploration program. The stratigraphic contacts shown on the summary logs represent the approximate boundaries between soil types; actual transitions may be more gradual. The soil and groundwater conditions depicted are for the specific dates and locations reported, and are not necessarily representative of other locations and times. The results of CPT soundings are provided in Attachment 3.

LAI personnel used the grab method to collect disturbed soil samples from the boring and test pits. Samples collected in this manner were taken to LAI's soils laboratory for further examination and testing. A discussion of laboratory test procedures and the laboratory test results are included in Attachment 2.

Upon completion of drilling and sampling, the excavations were decommissioned in general accordance with local requirements.

Soil Classification System

MAJOR DIVISIONS			GRAPHIC SYMBOL	USCS LETTER SYMBOL(1)	TYPICAL DESCRIPTIONS (2)(3)
COARSE-GRAINED SOIL (More than 50% of material is larger than No. 200 sieve size)	GRAVEL AND GRAVELLY SOIL (More than 50% of coarse fraction retained on No. 4 sieve)	CLEAN GRAVEL (Little or no fines)	▨	**GW**	Well-graded gravel; gravel/sand mixture(s); little or no fines
			▨	**GP**	Poorly graded gravel; gravel/sand mixture(s); little or no fines
		GRAVEL WITH FINES (Appreciable amount of fines)	▨	**GM**	Silty gravel; gravel/sand/silt mixture(s)
			▨	**GC**	Clayey gravel; gravel/sand/clay mixture(s)
	SAND AND SANDY SOIL (More than 50% of coarse fraction passed through No. 4 sieve)	CLEAN SAND (Little or no fines)	▨	**SW**	Well-graded sand; gravelly sand; little or no fines
			▨	**SP**	Poorly graded sand; gravelly sand; little or no fines
		SAND WITH FINES (Appreciable amount of fines)	▨	**SM**	Silty sand; sand/silt mixture(s)
			▨	**SC**	Clayey sand; sand/clay mixture(s)
FINE-GRAINED SOIL (More than 50% of material is smaller than No. 200 sieve size)	SILT AND CLAY (Liquid limit less than 50)		▨	**ML**	Inorganic silt and very fine sand; rock flour; silty or clayey fine sand or clayey silt with slight plasticity
			▨	**CL**	Inorganic clay of low to medium plasticity; gravelly clay; sandy clay; silty clay; lean clay
			▨	**OL**	Organic silt; organic, silty clay of low plasticity
	SILT AND CLAY (Liquid limit greater than 50)		▨	**MH**	Inorganic silt; micaceous or diatomaceous fine sand
			▨	**CH**	Inorganic clay of high plasticity; fat clay
			▨	**OH**	Organic clay of medium to high plasticity; organic silt
	HIGHLY ORGANIC SOIL		▨	**PT**	Peat; humus; swamp soil with high organic content

OTHER MATERIALS	GRAPHIC SYMBOL	LETTER SYMBOL	TYPICAL DESCRIPTIONS
PAVEMENT	■ ▨	**AC** or **PC**	Asphalt concrete pavement or Portland cement pavement
ROCK	▨	**RK**	Rock (See Rock Classification)
WOOD	▨	**WD**	Wood, lumber, wood chips
DEBRIS	▨	**DB**	Construction debris, garbage

Notes:
1. USCS letter symbols correspond to symbols used by the Unified Soil Classification System and ASTM classification methods. Dual letter symbols (e.g., SP-SM for sand or gravel) indicate soil with an estimated 5-15% fines. Multiple letter symbols (e.g., ML/CL) indicate borderline or multiple soil classifications.
2. Soil descriptions are based on the general approach presented in the Standard Practice for Description and Identification of Soils (Visual-Manual Procedure), outlined in ASTM D 2488. Where laboratory index testing has been conducted, soil classifications are based on the Standard Test Method for Classification of Soils for Engineering Purposes, as outlined in ASTM D 2487.
3. Soil description terminology is based on visual estimates (in the absence of laboratory test data) of the percentages of each soil type and is defined as follows:

   Primary Constituent: > 50% - "GRAVEL," "SAND," "SILT," "CLAY," etc.
   Secondary Constituents: > 30% and ≤ 50% - "very gravelly," "very sandy," "very silty," etc.
   > 15% and ≤ 30% - "gravelly," "sandy," "silty," etc.
   Additional Constituents: > 5% and ≤ 15% - "with gravel," "with sand," "with silt," etc.
   ≤ 5% - "with trace gravel," "with trace sand," "with trace silt," etc., or not noted.

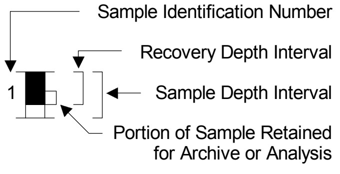
4. Soil density or consistency descriptions are based on judgement using a combination of sampler penetration blow counts, drilling or excavating conditions, field tests, and laboratory tests, as appropriate.

Drilling and Sampling Key

SAMPLER TYPE

Code	Description
a	3.25-inch O.D., 2.42-inch I.D. Split Spoon
b	2.00-inch O.D., 1.50-inch I.D. Split Spoon
c	Shelby Tube
d	Grab Sample
e	Single-Tube Core Barrel
f	Double-Tube Core Barrel
g	2.50-inch O.D., 2.00-inch I.D. WSDOT
h	3.00-inch O.D., 2.375-inch I.D. Mod. California
i	Other - See text if applicable
1	300-lb Hammer, 30-inch Drop
2	140-lb Hammer, 30-inch Drop
3	Pushed
4	Vibrocore (Rotosonic/Geoprobe)
5	Other - See text if applicable

SAMPLE NUMBER & INTERVAL



Field and Lab Test Data

Code	Description
PP = 1.0	Pocket Penetrometer, tsf
TV = 0.5	Torvane, tsf
PID = 100	Photoionization Detector VOC screening, ppm
W = 10	Moisture Content, %
D = 120	Dry Density, pcf
-200 = 60	Material smaller than No. 200 sieve, %
GS	Grain Size - See separate figure for data
AL	Atterberg Limits - See separate figure for data
GT	Other Geotechnical Testing
CA	Chemical Analysis

Groundwater

▽	Approximate water level at time of drilling (ATD)
▼	Approximate water level at time after drilling/excavation/well

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NuStar Energy LP Terminal
Improvements
Portland, OR

Soil Classification System and Key

Figure
1-1

HB-1

LAI Project No: 1374013.010.012

SAMPLE DATA

SOIL PROFILE

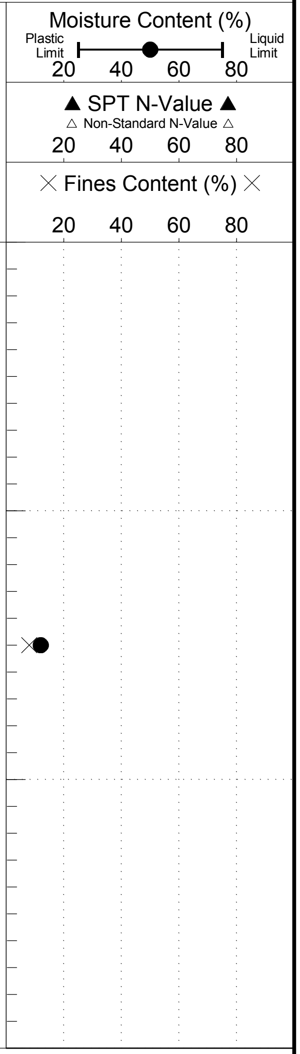
Drilling Method: Hand Auger

Ground Elevation (ft): Not Determined

Drilled By: LAI

Logged By: JSN Date: 03/20/19

Depth (ft)	Elevation (ft)	Sample Number & Interval	Sampler Type	Blows/Foot	Test Data	Graphic Symbol	USCS Symbol		Groundwater
0							SP-SM	Brown, fine to coarse SAND with gravel and silt (loose, damp) **(FILL)**	Groundwater Not Encountered
		S-1	d5		W = 12 GS				
		S-2	d5						
6									



Boring Completed 03/20/19
Total Depth of Boring = 6.0 ft.

Notes:
1. Stratigraphic contacts are based on field interpretations and are approximate.
2. Reference to the text of this report is necessary for a proper understanding of subsurface conditions.
3. Refer to "Soil Classification System and Key" figure for explanation of graphics and symbols.

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NuStar Energy LP Terminal
Improvements
Portland, OR

Log of Boring HB-1

Figure
1-2

TP-1

LAI Project No: 1374013.010.012

SAMPLE DATA

SOIL PROFILE

Drilling Method: Tracked Excavator

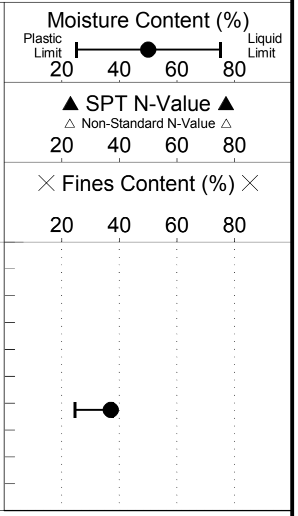
Ground Elevation (ft): Not Determined

Drilled By: Howard's Construction & Excvtg

Logged By: JSN Date: 03/21/19

Depth (ft)	Elevation (ft)	Sample Number & Interval	Sampler Type	Blows/Foot	Test Data	Graphic Symbol	USCS Symbol	Description	Groundwater
0						▨	CL	Brown, gravelly CLAY with sand (loose, damp) **(FILL)**	Groundwater Not Encountered
		S-1	d5		W = 37 AL	▨			
2		S-2	d5				BST	**(BEDROCK)**	

Boring Completed 03/21/19
Total Depth of Boring = 2.0 ft.

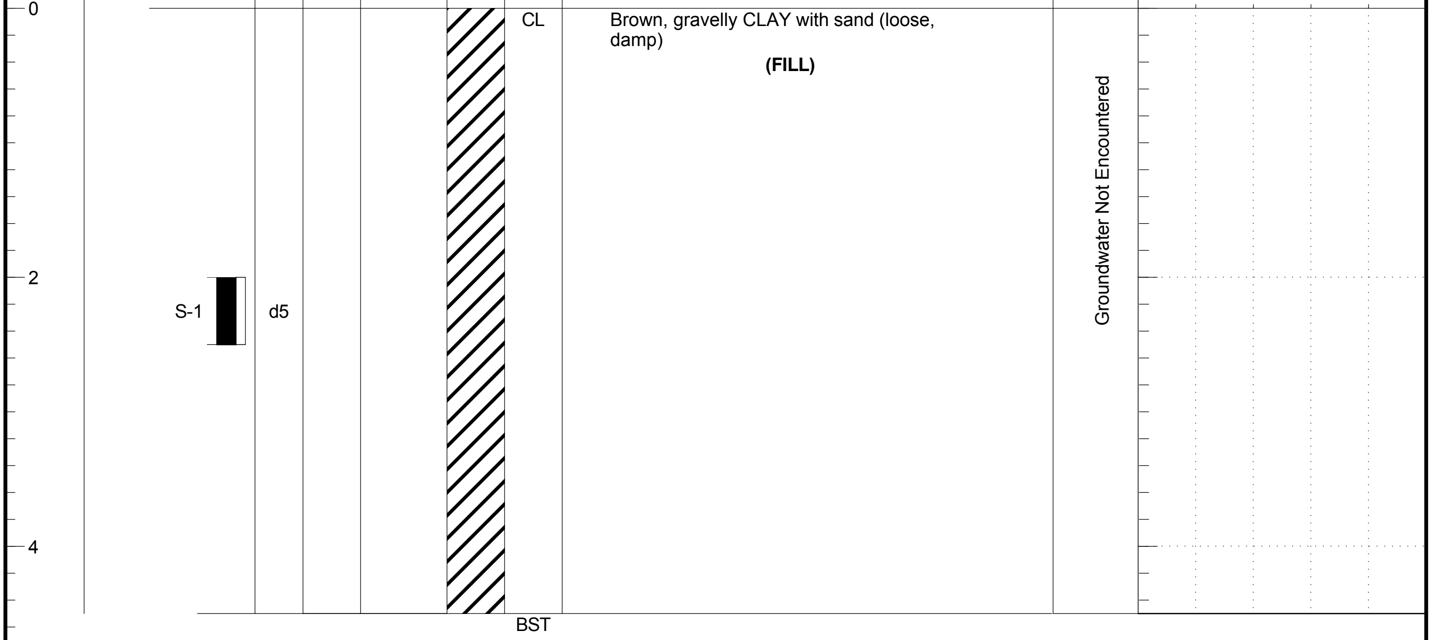


Notes:
1. Stratigraphic contacts are based on field interpretations and are approximate.
2. Reference to the text of this report is necessary for a proper understanding of subsurface conditions.
3. Refer to "Soil Classification System and Key" figure for explanation of graphics and symbols.

TP-2

LAI Project No: 1374013.010.012

SAMPLE DATA						SOIL PROFILE				Groundwater	Moisture Content (%) / SPT N-Value / Fines Content (%)
Depth (ft)	Elevation (ft)	Sample Number & Interval	Sampler Type	Blows/Foot	Test Data	Graphic Symbol	USCS Symbol	Drilling Method: Tracked Excavator Ground Elevation (ft): Not Determined Drilled By: Howard's Construction & Excvtg Logged By: JSN	Date: 03/21/19		Plastic Limit – Liquid Limit 20 40 60 80 ▲ SPT N-Value ▲ △ Non-Standard N-Value △ 20 40 60 80 × Fines Content (%) × 20 40 60 80



CL Brown, gravelly CLAY with sand (loose, damp)

(FILL)

BST **(BEDROCK)**

Boring Completed 03/21/19
Total Depth of Boring = 4.5 ft.

Notes:
1. Stratigraphic contacts are based on field interpretations and are approximate.
2. Reference to the text of this report is necessary for a proper understanding of subsurface conditions.
3. Refer to "Soil Classification System and Key" figure for explanation of graphics and symbols.

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NuStar Energy LP Terminal Improvements
Portland, OR

Log of Boring TP-2

Figure
1-4

ATTACHMENT 2

Laboratory Soil Testing

ATTACHMENT 2
LABORATORY SOIL TESTING

Samples obtained from the explorations were taken to LAI's soils laboratory for further examination and testing. Laboratory tests were performed on representative samples to characterize engineering and index properties of site soils. The laboratory testing program was performed in general accordance with the ASTM International (ASTM) standard test methods described below.

Natural Moisture Content

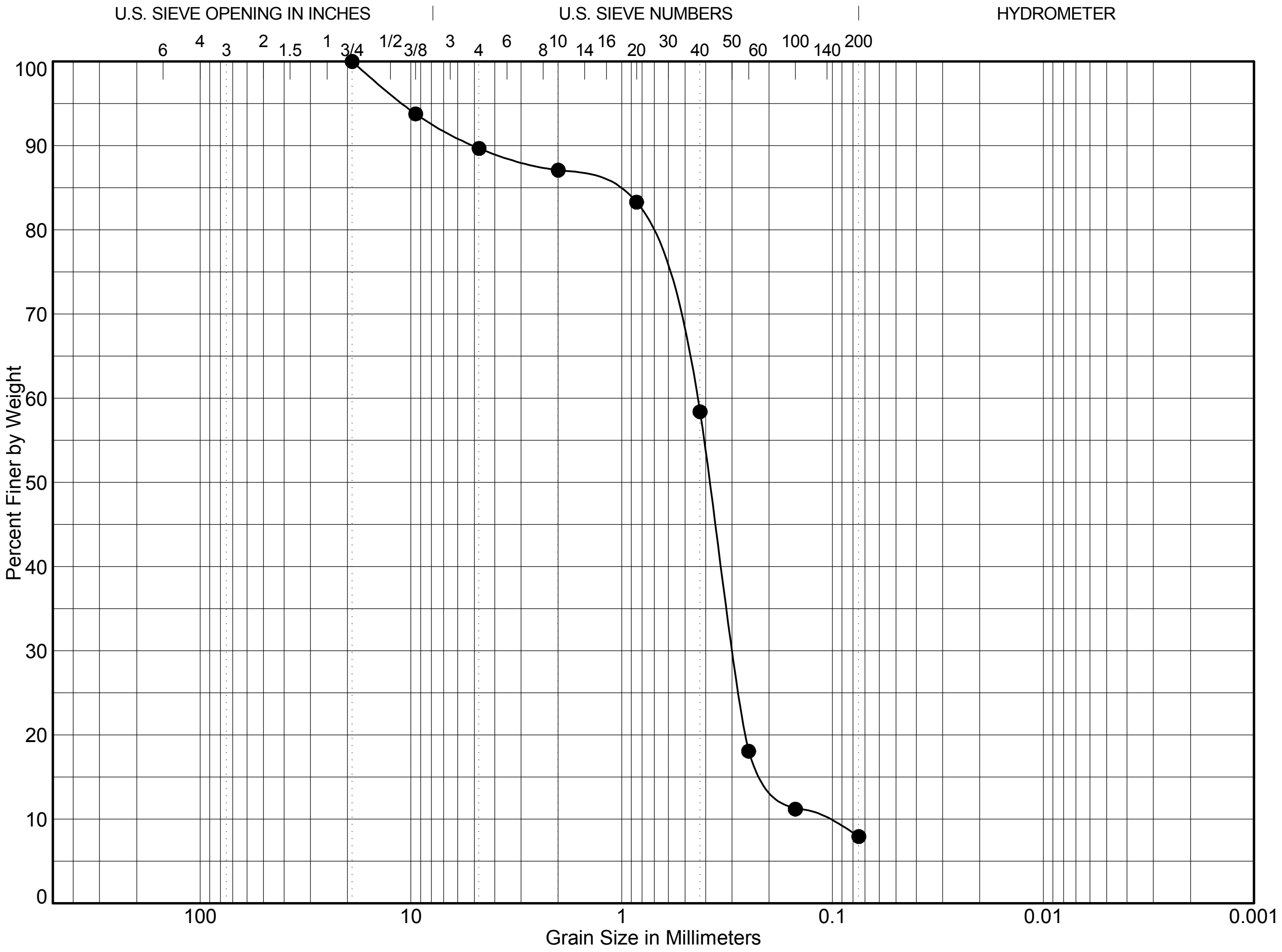
Natural moisture content determinations were performed on select soil samples in general accordance with ASTM test method D2216. The natural moisture content is shown as W = xx (i.e., percent of dry weight) in the column labeled "Test Data" on the summary boring logs in Attachment 1.

Grain Size Analysis

To provide an indication of the grain size distribution of site soils, grain size analyses were performed in accordance with ASTM test method D422. Samples selected for grain size analysis are designated with a "GS" in the column labeled "Test Data" on the summary boring logs in Attachment 1. The results of the grain size analyses are presented in the form of grain size distribution curves on Figure 2-1.

Atterberg Limit Determination

To assess the plasticity of fine-grained site soils, Atterberg limit tests were performed in general accordance with ASTM test method D4318. Samples selected for Atterberg limit tests are designated with an "AL" in the column labeled "Test Data" on the summary boring logs in Attachment 1. The results of the Atterberg limit tests are presented in graphical and tabular form on Figure 2-2.



Cobbles	Gravel		Sand			Silt or Clay
	coarse	fine	coarse	medium	fine	

Point	Depth	Classification	LL	PL	PI	C_c	C_u
● HB-1	3.0	Fine to coarse SAND with gravel and silt (SP-SM)				1.65	3.81

Point	Depth	D_{100}	D_{60}	D_{50}	D_{30}	D_{10}	%Coarse Gravel	% Fine Gravel	% Coarse Sand	% Medium Sand	% Fine Sand	% Fines
● HB-1	3.0	19	0.444	0.381	0.293	0.117	0.0	10.3	2.6	28.7	50.5	7.9

$C_c = D_{30}^2/(D_{60}* D_{10})$

$C_u = D_{60}/D_{10}$

To be well graded: $1 < C_c < 3$ and $C_u > 4$ for GW or $C_u > 6$ for SW

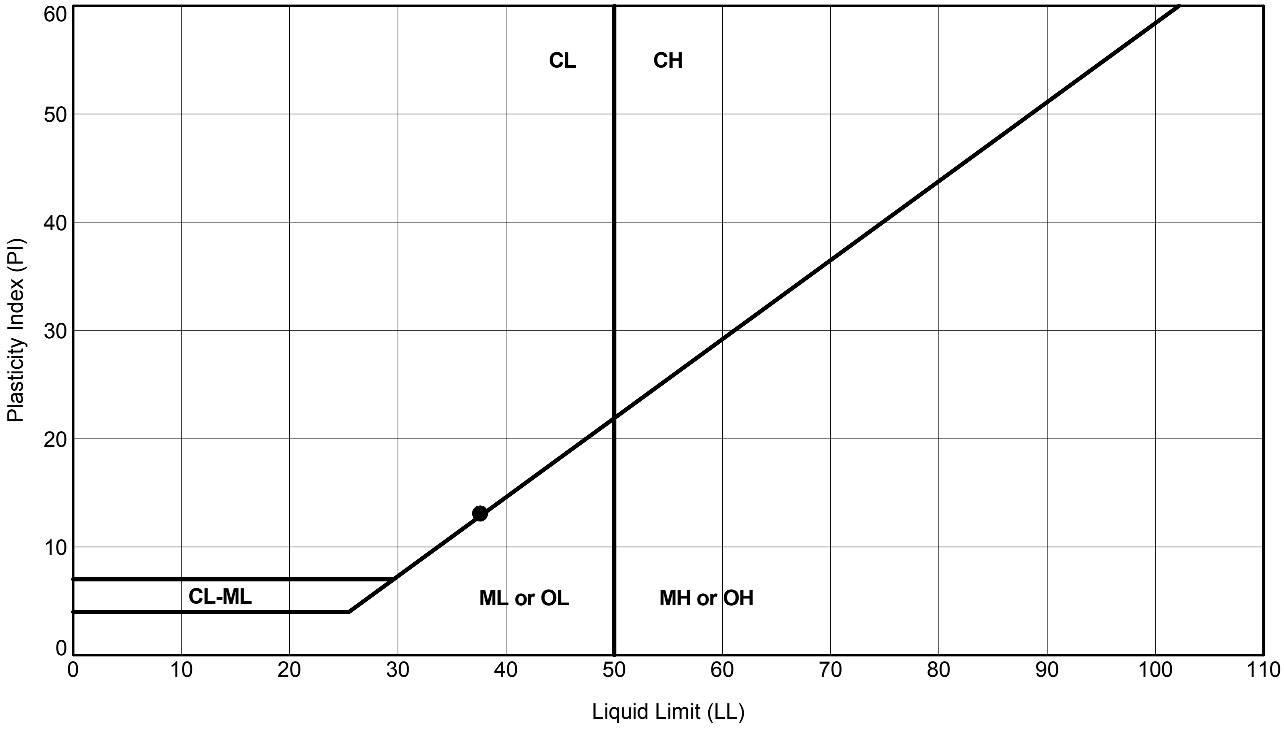
1374013.010.012 4/18/19 O:\1374013.010\T1374013.010.012.GPJ GRAIN SIZE W/STATS

LANDAU ASSOCIATES

NuStar Energy LP Terminal
Improvements
Portland, OR

Grain Size Test Data

Figure
2-1



ATTERBERG LIMIT TEST RESULTS

Symbol	Exploration Number	Sample Number	Depth (ft)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Natural Moisture (%)	Soil Description	Unified Soil Classification
●	TP-1	S-1	2.0	38	25	13	37	CLAY	CL

ASTM D 4318 Test Method

LANDAU ASSOCIATES

NuStar Energy LP Terminal Improvements Portland, OR	Plasticity Chart	Figure 2-2

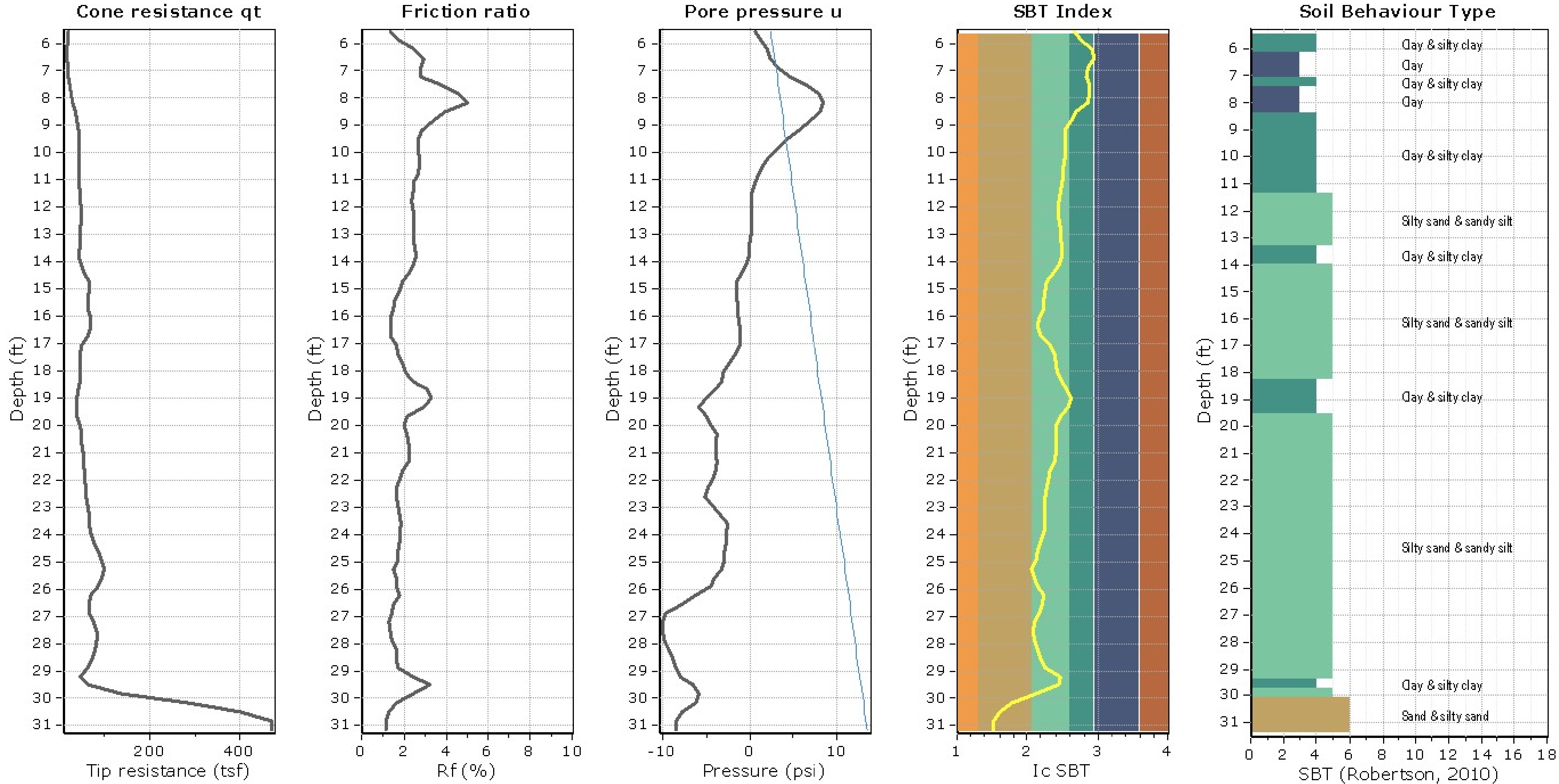
ATTACHMENT 3

Cone Penetration Test Results

Landau Associates
955 Malin Lane SW, Suite B
Tumwater, WA 98501
(360)791-3178

CPT: 19054 CPT-1 Text File
Total depth: 31.17 ft, Date: 3/29/2019
Surface Elevation: 0.00 ft
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator:

Project: **NuStar_CPT-1**
Location: **Portland, OR**

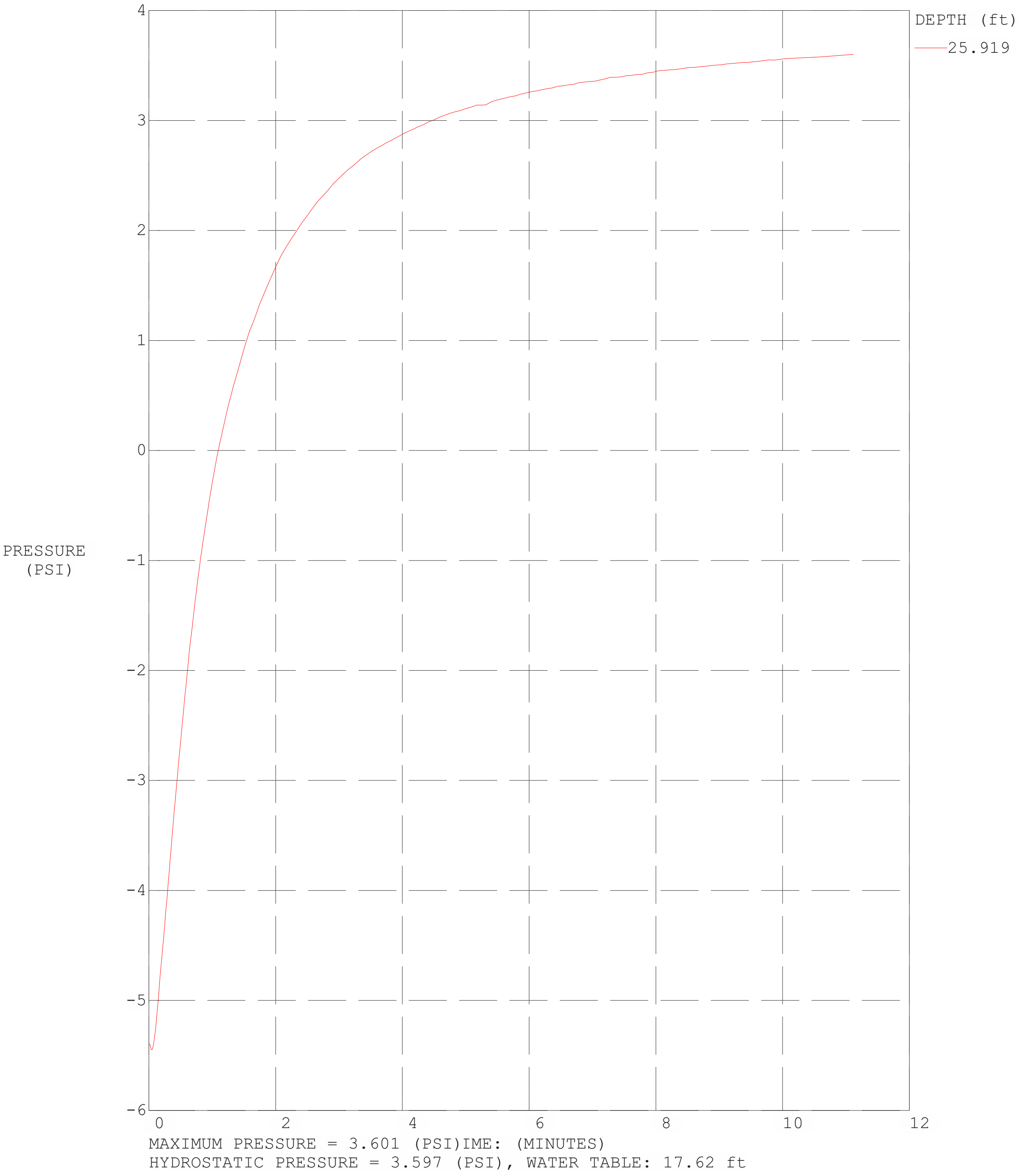


SBT legend

- 🟥 1. Sensitive fine grained
- 🟫 2. Organic material
- 🟦 3. Clay to silty clay
- 🟩 4. Clayey silt to silty clay
- 🟩 5. Silty sand to sandy silt
- 🟫 6. Clean sand to silty sand
- 🟧 7. Gravely sand to sand
- ⬜ 8. Very stiff sand to clayey sand
- ⬜ 9. Very stiff fine grained

COMMENT: Landau / CPT-1 / NuStar Terminal Portland

TEST DATE: 3/21/2019 8:44:16 AM



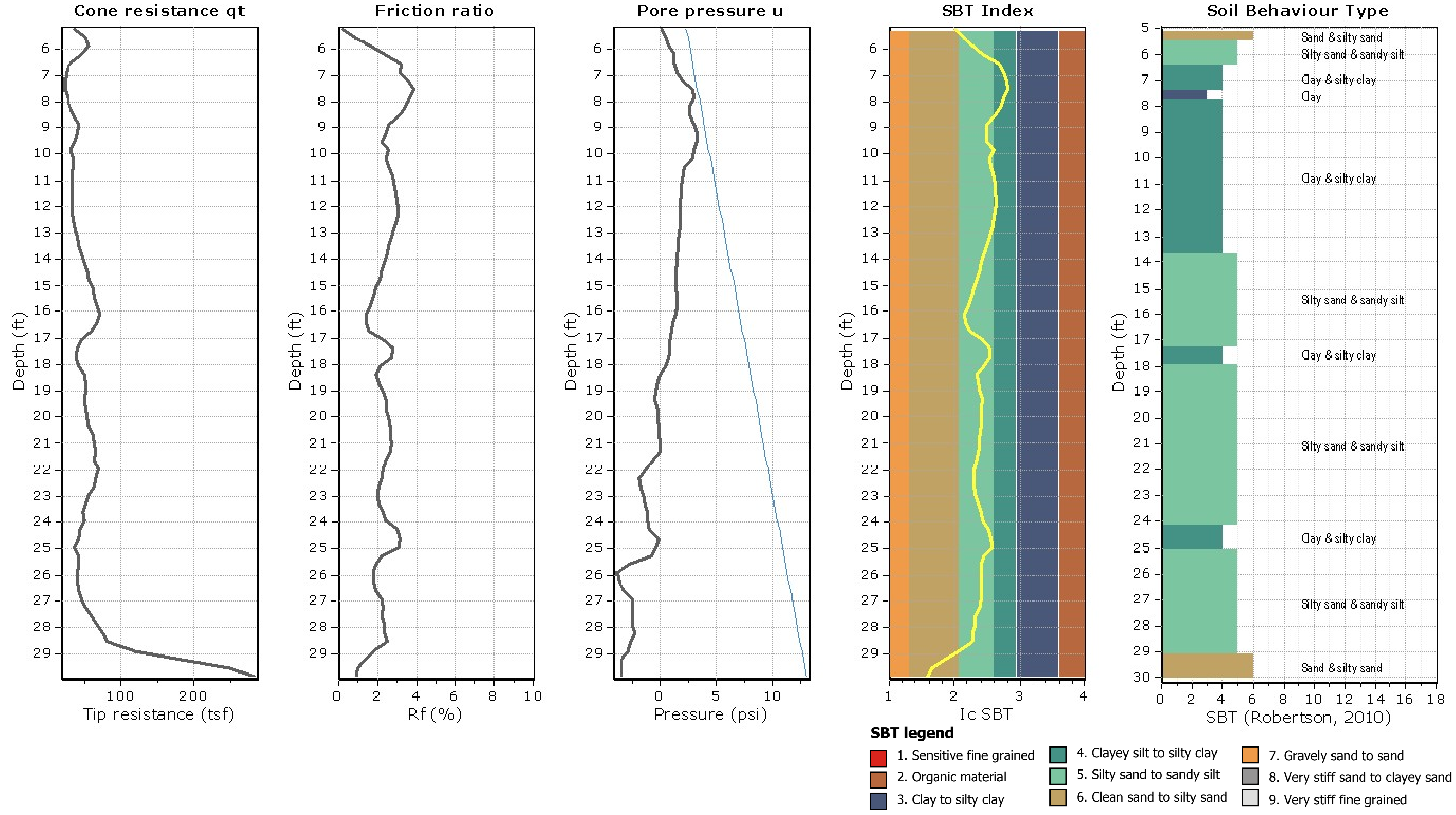
MAXIMUM PRESSURE = 3.601 (PSI)IME: (MINUTES)
HYDROSTATIC PRESSURE = 3.597 (PSI), WATER TABLE: 17.62 ft

Landau Associates
955 Malin Lane SW, Suite B
Tumwater, WA 98501
(360)791-3178

Project: **Nu_Star**
Location: **Portland**

CPT: 19054 CPT-1a Text File

Total depth: 29.86 ft, Date: 4/16/2019
Surface Elevation: 0.00 ft
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator:

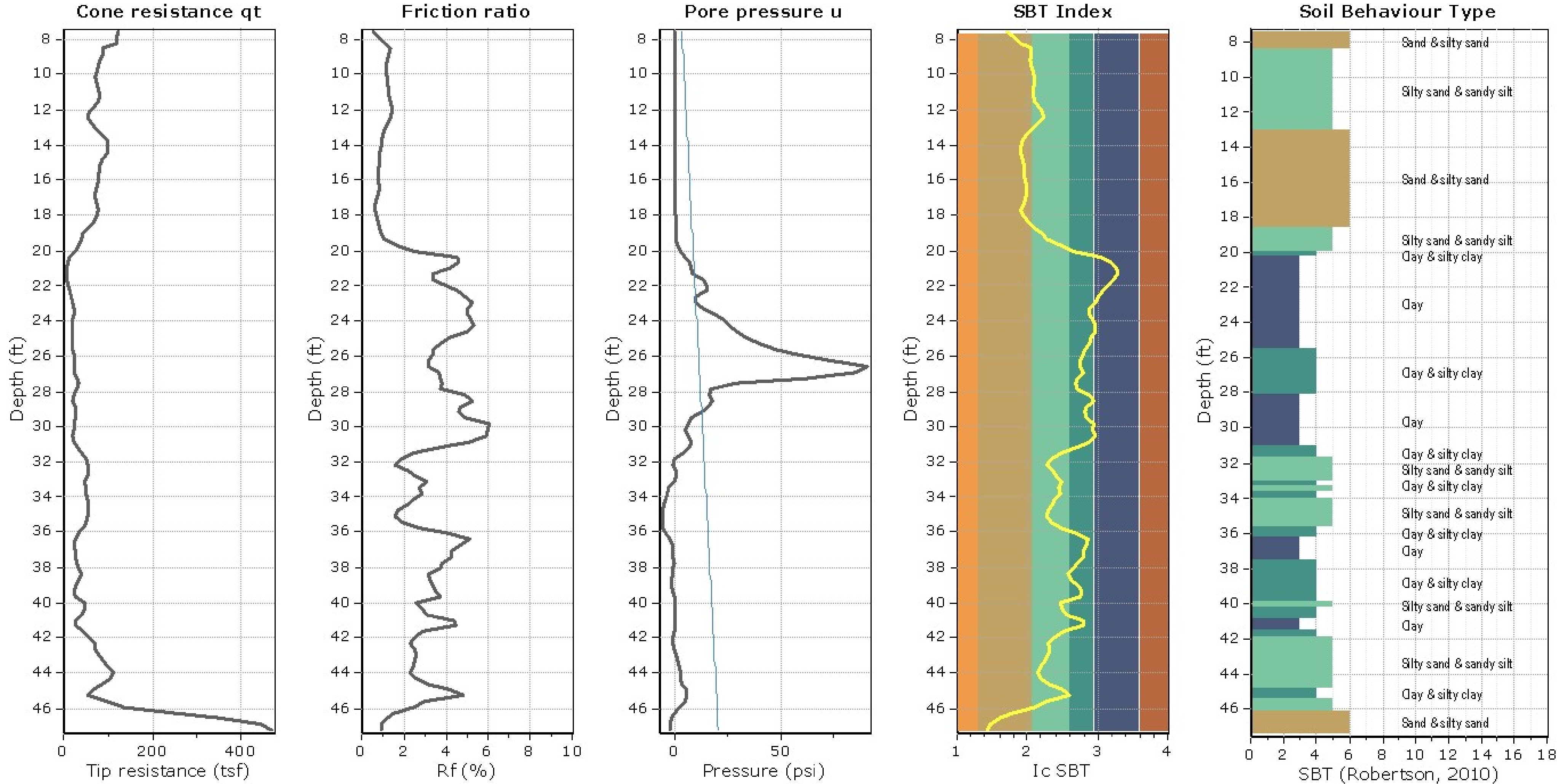


Landau Associates
955 Malin Lane SW, Suite B
Tumwater, WA 98501
(360)791-3178

CPT: 19054 CPT-2 Text File

Total depth: 47.24 ft, Date: 4/2/2019
Surface Elevation: 0.00 ft
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator:

Project:
Location:

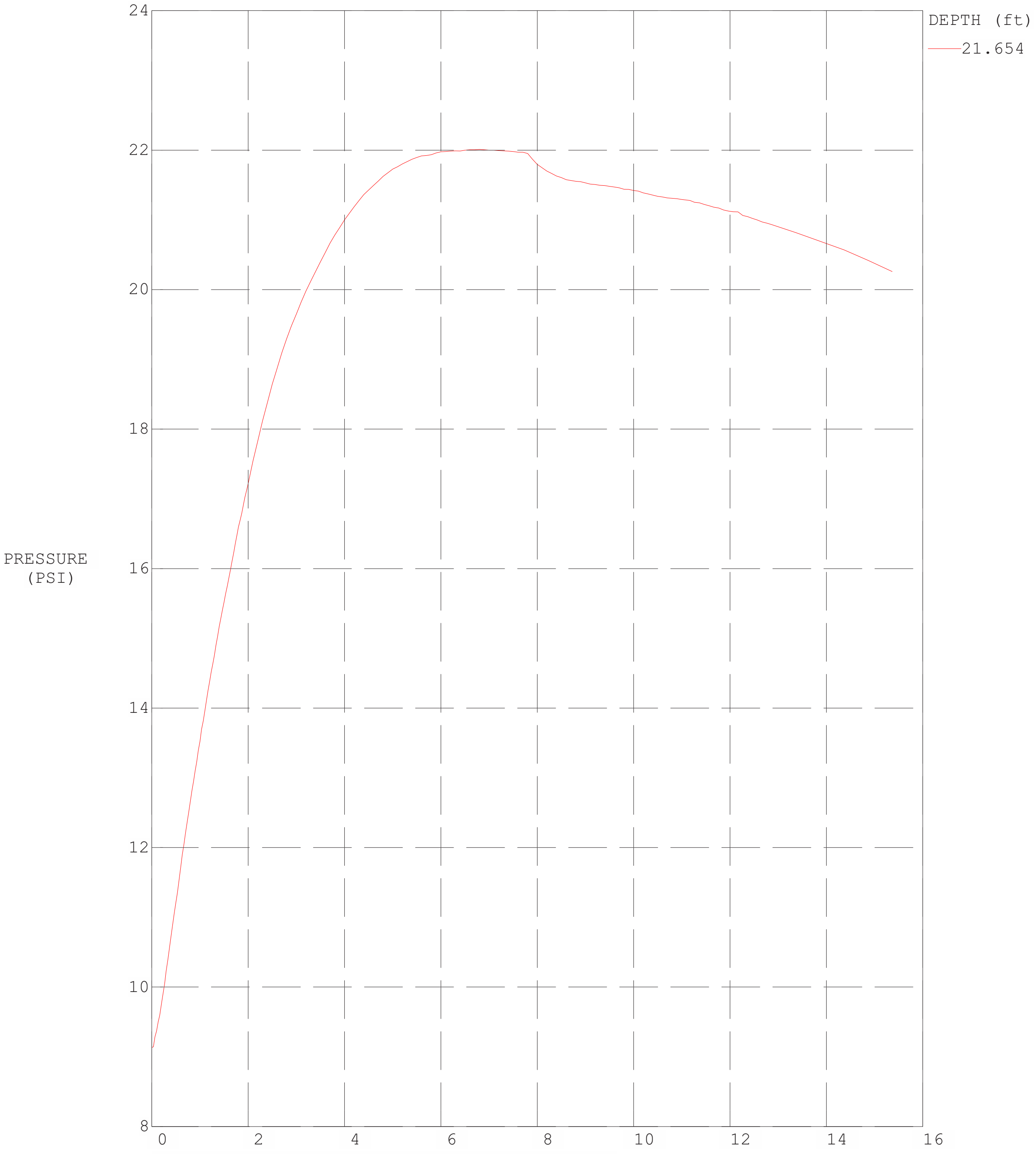


SBT legend

- 🟥 1. Sensitive fine grained
- 🟫 2. Organic material
- 🟦 3. Clay to silty clay
- 🟩 4. Clayey silt to silty clay
- 🟩 5. Silty sand to sandy silt
- 🟨 6. Clean sand to silty sand
- 🟧 7. Gravely sand to sand
- ⬜ 8. Very stiff sand to clayey sand
- ⬜ 9. Very stiff fine grained

COMMENT: Landau / CPT-2 / NuStar Terminal Portland

TEST DATE: 3/21/2019 11:04:11 AM

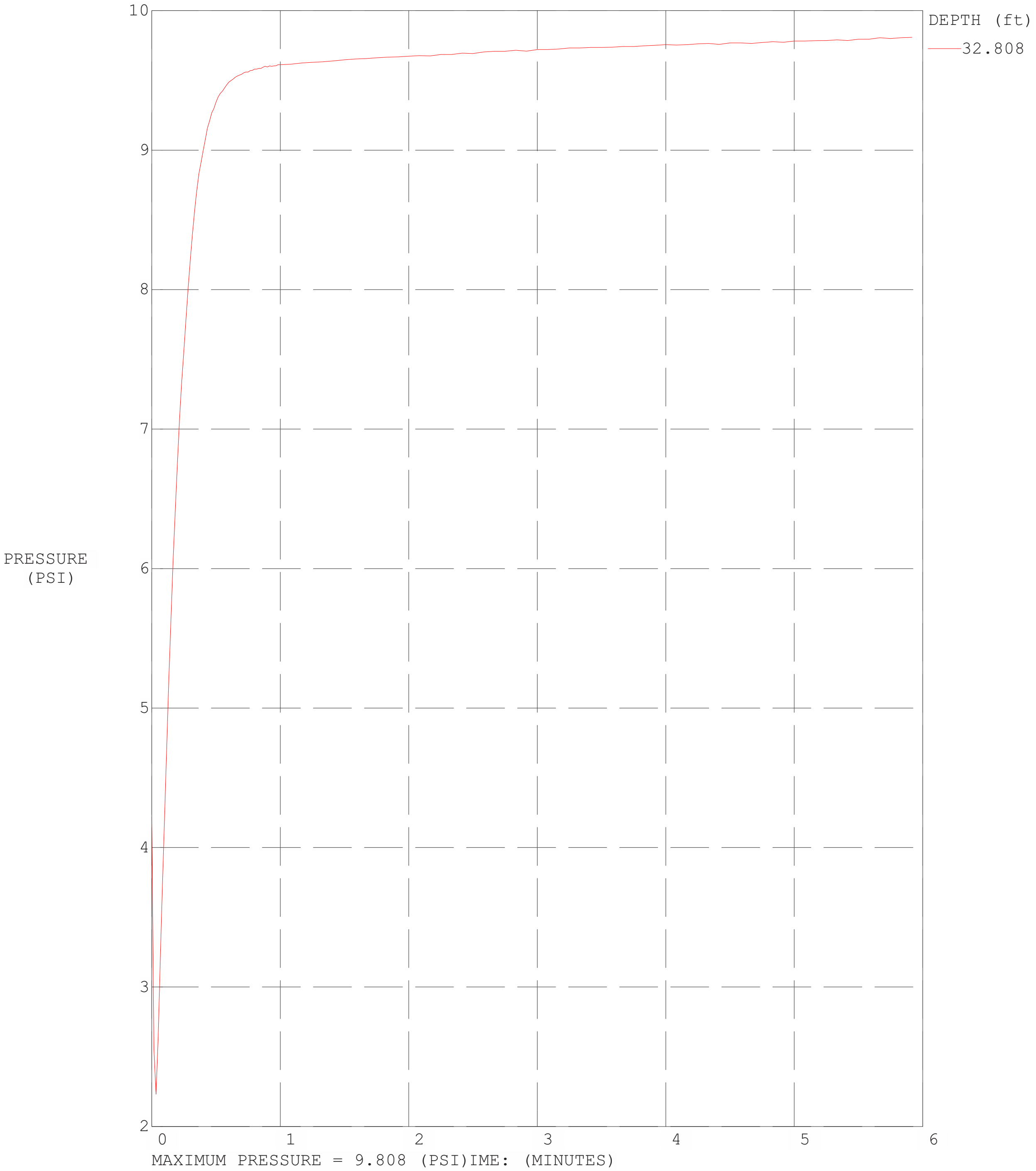


MAXIMUM PRESSURE = 22.011 (PSI)ME: (MINUTES)

HYDROSTATIC PRESSURE = 4.977 (PSI), WATER TABLE: 10.17 ft

COMMENT: Landau / CPT-2 / NuStar Terminal Portland

TEST DATE: 3/21/2019 11:04:11 AM

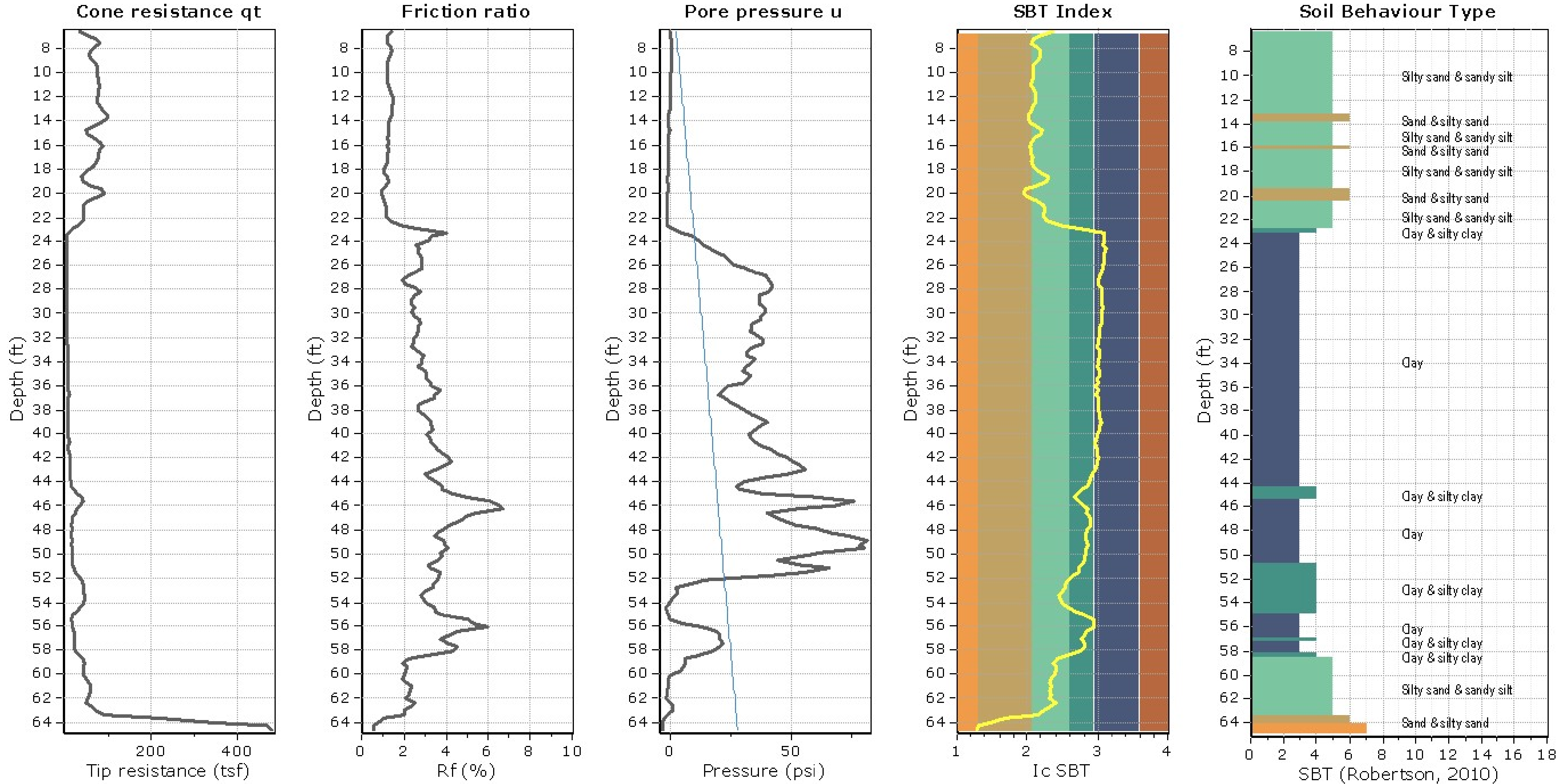


MAXIMUM PRESSURE = 9.808 (PSI)IME: (MINUTES)
HYDROSTATIC PRESSURE = 9.811 (PSI), WATER TABLE: 10.17 ft

Landau Associates
955 Malin Lane SW, Suite B
Tumwater, WA 98501
(360)791-3178

CPT: 19054 CPT-3 Text File
Total depth: 64.63 ft, Date: 4/2/2019
Surface Elevation: 0.00 ft
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator:

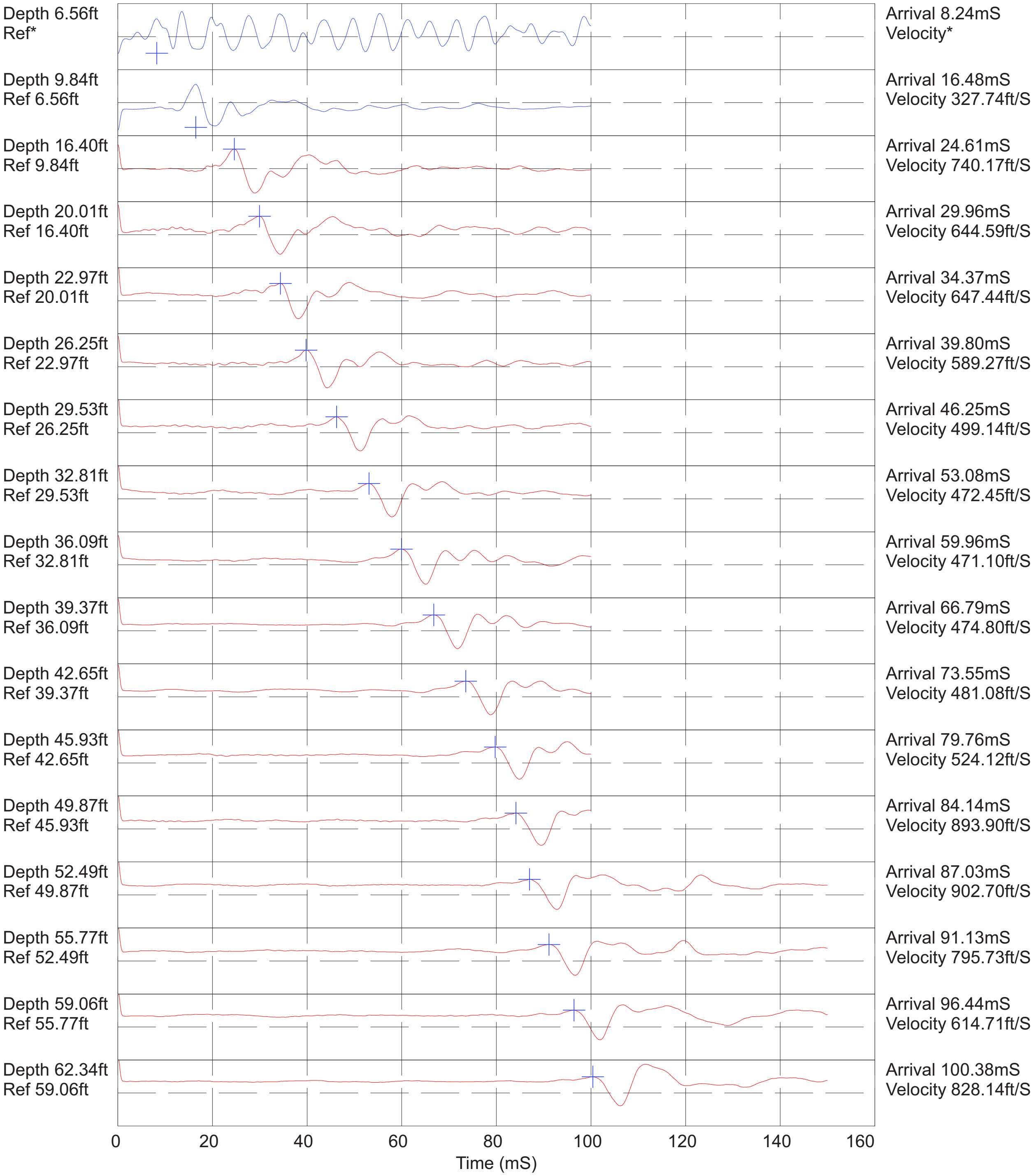
Project:
Location:



SBT legend

🟥 1. Sensitive fine grained
🟫 2. Organic material
🟦 3. Clay to silty clay
🟩 4. Clayey silt to silty clay
🟩 5. Silty sand to sandy silt
🟫 6. Clean sand to silty sand
🟧 7. Gravely sand to sand
⬜ 8. Very stiff sand to clayey sand
⬜ 9. Very stiff fine grained

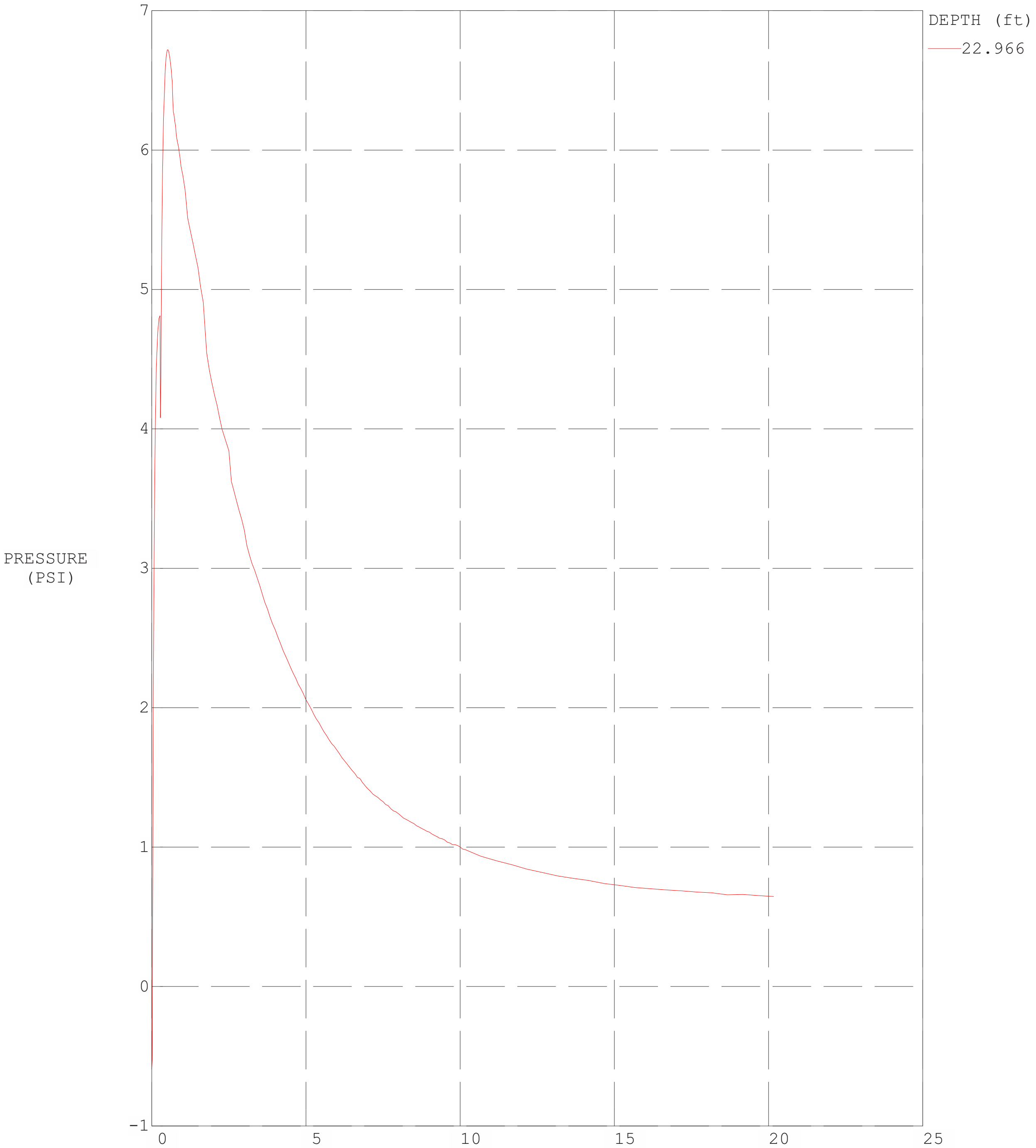
COMMENT: Landau / CPT-3 / NuStar Terminal Portland



Hammer to Rod String Distance (ft): 5.58
* = Not Determined

COMMENT: Landau / CPT-3 / NuStar Terminal Portland

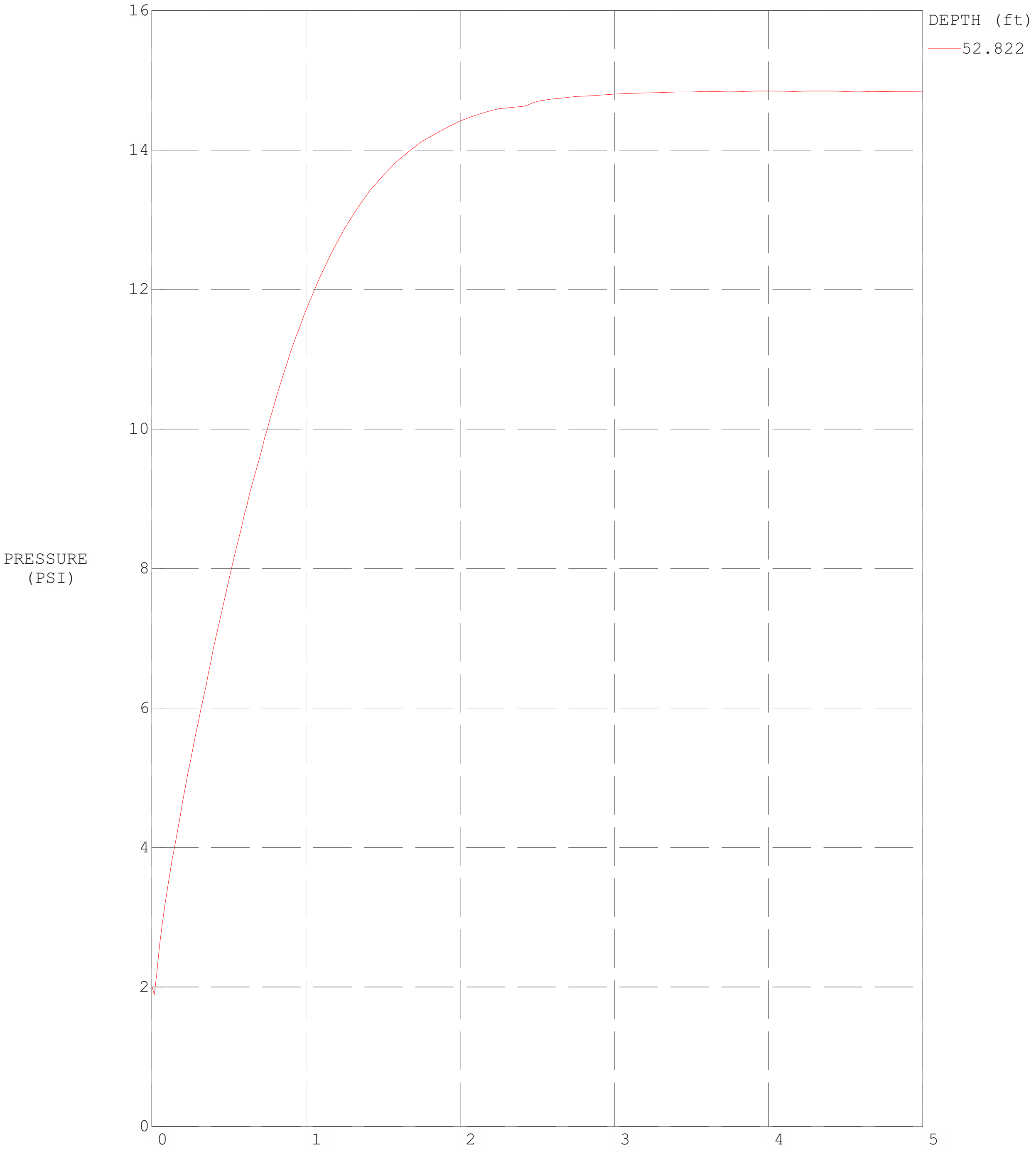
TEST DATE: 3/21/2019 12:37:01 PM



MAXIMUM PRESSURE = 6.72 (PSI)TIME: (MINUTES)
HYDROSTATIC PRESSURE = 1.924 (PSI), WATER TABLE: 18.53 ft

COMMENT: Landau / CPT-3 / NuStar Terminal Portland

TEST DATE: 3/21/2019 12:37:01 PM



MAXIMUM PRESSURE = 14.848 (PSI)ME: (MINUTES)

HYDROSTATIC PRESSURE = 14.863 (PSI), WATER TABLE: 18.53 ft

APPENDIX B – SEISMIC CPT DATA

PRESENTATION OF SITE INVESTIGATION RESULTS

NuStar Portland

Prepared for:

Gannett Fleming

ConeTec Job No: 24-59-27209

Project Start Date: 2024-02-22
Project End Date: 2024-02-22
Release Date: 2024-03-01

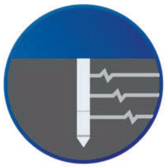
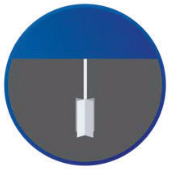
Report Prepared by:

ConeTec, Inc.

3530 NW St Helens Rd, Portland, OR 97210

Tel: (253) 397-4861

ConeTecOR@conetec.com
www.conetec.com
www.conetecdataservices.com

ABOUT THIS REPORT

The enclosed report presents the results of the site investigation program conducted by ConeTec, Inc. for Gannett Fleming.

Please note that this report, which also includes all accompanying data, are subject to the 3rd Party Disclaimer and Client Disclaimer that follow in the 'Limitations' section of this report. Please refer to the list of attached documents following the text of this report. A site map, test summaries, and test plots are all included in the body of this report.

Project	
Client	Gannett Fleming
Project	NuStar Portland
ConeTec Project Number	24-59-27209
Test Types	SCPTu
Additional Comments	None

Contents

The following listed below are included in the body of this report:

SITE MAP

All soundings are approximate unless otherwise stated in the body of the report.

ConeTec Job Number: 24-59-27209

Client: Gannett Fleming

Project: NuStar Portland

Release Date: 2024-03-01

LIMITATIONS

3rd Party Disclaimer

The “Report” refers to this report titled: NuStar Portland

The Report was prepared by ConeTec for: Gannett Fleming

The Report is confidential and may not be distributed to or relied upon by any third parties without the express written consent of ConeTec. Any third parties gaining access to the Report do not acquire any rights as a result of such access. Any use which a third party makes of the Report, or any reliance on or decisions made based on it, are the responsibility of such third parties. ConeTec accepts no responsibility for loss, damage and/or expense, if any, suffered by any third parties as a result of decisions made, or actions taken or not taken, which are in any way based on, or related to, the Report or any portion(s) thereof.

Client Disclaimer

ConeTec was retained by: Gannett Fleming

The “Report” refers to this report titled: NuStar Portland

ConeTec was retained to collect and provide the raw data (“Data”) which is included in the Report.

ConeTec has collected and reported the Data in accordance with current industry standards. No other warranty, express or implied, with respect to the Data is made by ConeTec. In order to properly understand the Data included in the Report, reference must be made to the documents accompanying and other sources referenced in the Report in their entirety. Other than the Data, the contents of the Report (including any Interpretations) should not be relied upon in any fashion without independent verification and ConeTec is in no way responsible for any loss, damage or expense resulting from the use of, and/or reliance on, such material by any party.

Closure

Thank you for the opportunity to work on this project. The equipment used as well the field procedures followed, all complied with current accepted best practice standards.

Report prepared by: Alex Leibold
Report Reviewed by: Jessie Martinez

PROJECT INFORMATION

Rig		
Description	**Deployment System**	**Test Type**
C02-020 CPT Truck Rig	Twin mounted cylinders	SCPTu

Coordinates		
Test Type	**Collection Method**	**EPSG Number**
SCPTu	Consumer Grade GPS	4326 (WGS84 / LatLong)

Piezocones Used for this Project						
Cone Description	**Cone Number**	**Cross Sectional Area** (cm^2)	**Sleeve Area** (cm^2)	**Tip Capacity** (bar)	**Sleeve Capacity** (bar)	**Pore Pressure Capacity** (bar)
EC859:T1500F15U35	859	15	225	1500	15	35

Cone Penetration Test (CPTu)	
Depth reference	Depths are referenced to the existing ground surface at the time of each test.
Tip and sleeve data offset	0.1 Meters. This has been accounted for in the CPT data files.

Calculated Geotechnical Parameters	
Additional information	The Normalized Soil Behaviour Type Chart based on Q_{tn} (SBT Q_{tn}) (Robertson, 2009) was used to classify the soil for this project. A detailed set of calculated CPTu parameters have been generated and are provided in Excel format files in the release folder. The CPTu parameter calculations are based on values of corrected tip resistance (q_t) sleeve friction (f_s) and pore pressure (u_2). Effective stresses are calculated based on unit weights that have been assigned to the individual soil behaviour type zones and the assumed equilibrium pore pressure profile. Soils were classified as either drained or undrained based on the Q_{tn} Normalized Soil Behaviour Type Chart (Robertson, 2009). Calculations for both drained and undrained parameters were included for materials that classified as silt mixtures (zone 4).

Methodology Statements and Data File Formats

METHODOLOGY STATEMENTS

CONE PENETRATION TEST (CPTu) - eSeries

Cone penetration tests (CPTu) are conducted using an integrated electronic piezocone penetrometer and data acquisition system manufactured by Adara Systems Ltd., a subsidiary of ConeTec.

ConeTec's piezocone penetrometers are compression type designs in which the tip and friction sleeve load cells are independent and have separate load capacities. The piezocones use strain gauged load cells for tip and sleeve friction and a strain gauged diaphragm type transducer for recording pore pressure. The piezocones also have a platinum resistive temperature device (RTD) for monitoring the temperature of the sensors, an accelerometer type dual axis inclinometer and two geophone sensors for recording seismic signals. All signals are amplified and measured with minimum sixteen-bit resolution down hole within the cone body, and the signals are sent to the surface using a high bandwidth, error corrected digital interface through a shielded cable.

ConeTec penetrometers are manufactured with various tip, friction and pore pressure capacities in both 10 cm^2 and 15 cm^2 tip base area configurations in order to maximize signal resolution for various soil conditions. The specific piezocone used for each test is described in the CPT summary table. The 15 cm^2 penetrometers do not require friction reducers as they have a diameter larger than the deployment rods. The 10 cm^2 piezocones use a friction reducer consisting of a rod adapter extension behind the main cone body with an enlarged cross sectional area (typically 44 millimeters diameter over a length of 32 millimeters with tapered leading and trailing edges) located at a distance of 585 millimeters above the cone tip.

The penetrometers are designed with equal end area friction sleeves, a net end area ratio of 0.8 and cone tips with a 60 degree apex angle.

All ConeTec piezocones can record pore pressure at various locations. Unless otherwise noted, the pore pressure filter is located directly behind the cone tip in the "u_2" position (ASTM Type 2). The filter is six millimeters thick, made of porous plastic (polyethylene) having an average pore size of 125 microns (90-160 microns). The function of the filter is to allow rapid movements of extremely small volumes of water needed to activate the pressure transducer while preventing soil ingress or blockage.

The piezocone penetrometers are manufactured with dimensions, tolerances and sensor characteristics that are in general accordance with the current ASTM D5778 standard. ConeTec's calibration criteria also meets or exceeds those of the current ASTM D5778 standard. An illustration of the piezocone penetrometer is presented in Figure CPTu.

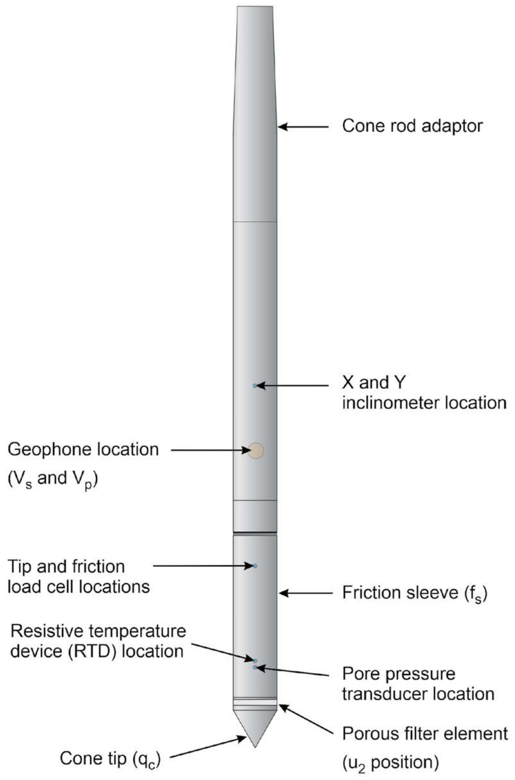


Figure CPTu. Piezocone Penetrometer (15 cm^2)

The ConeTec data acquisition system consists of a Windows based computer, signal interface box, and power supply. The signal interface combines depth increment signals, seismic trigger signals and the downhole digital data. This combined data is then sent to the Windows based computer for collection and presentation. The data is recorded at fixed depth increments using a depth encoder that is either portable or integrated into the rig. The typical recording interval is 2.5 centimeters; custom recording intervals are possible.

The system displays the CPTu data in real time and records the following parameters to a storage media during penetration:

- Depth
- Uncorrected tip resistance (q_c)
- Sleeve friction (f_s)
- Dynamic pore pressure (u)
- Additional sensors such as resistivity, passive gamma, ultra violet induced fluorescence, if applicable

All testing is performed in accordance to ConeTec's CPTu operating procedures which are in general accordance with the current ASTM D5778 standard.

Prior to the start of a CPTu sounding a suitable cone is selected, the cone and data acquisition system are powered on, the pore pressure system is saturated with silicone oil and the baseline readings are recorded with the cone hanging freely in a vertical position.

The CPTu is conducted at a steady rate of two centimeters per second, within acceptable tolerances. Typically one meter length rods with an outer diameter of 1.5 inches are added to advance the cone to the sounding termination depth. After cone retraction final baselines are recorded.

Additional information pertaining to ConeTec's cone penetration testing procedures:

- Each filter is saturated in silicone oil under vacuum pressure prior to use
- Baseline readings are compared to previous readings
- Soundings are terminated at the client's target depth or at a depth where an obstruction is encountered, excessive rod flex occurs, excessive inclination occurs, equipment damage is likely to take place, or a dangerous working environment arises
- Differences between initial and final baselines are calculated to ensure zero load offsets have not occurred and to ensure compliance with ASTM standards

The interpretation of piezocone data for this report is based on the corrected tip resistance (q_t), sleeve friction (f_s) and pore water pressure (u). The interpretation of soil type is based on the correlations developed by Robertson, P.K., 2010. The Soil Behavior Type (SBT) classification chart developed by Robertson, P.K., 2010 is presented in Figure SBT. It should be noted that it is not always possible to accurately identify a soil behavior type based on these parameters. In these situations, experience, judgment and an assessment of other parameters may be used to infer soil behavior type.

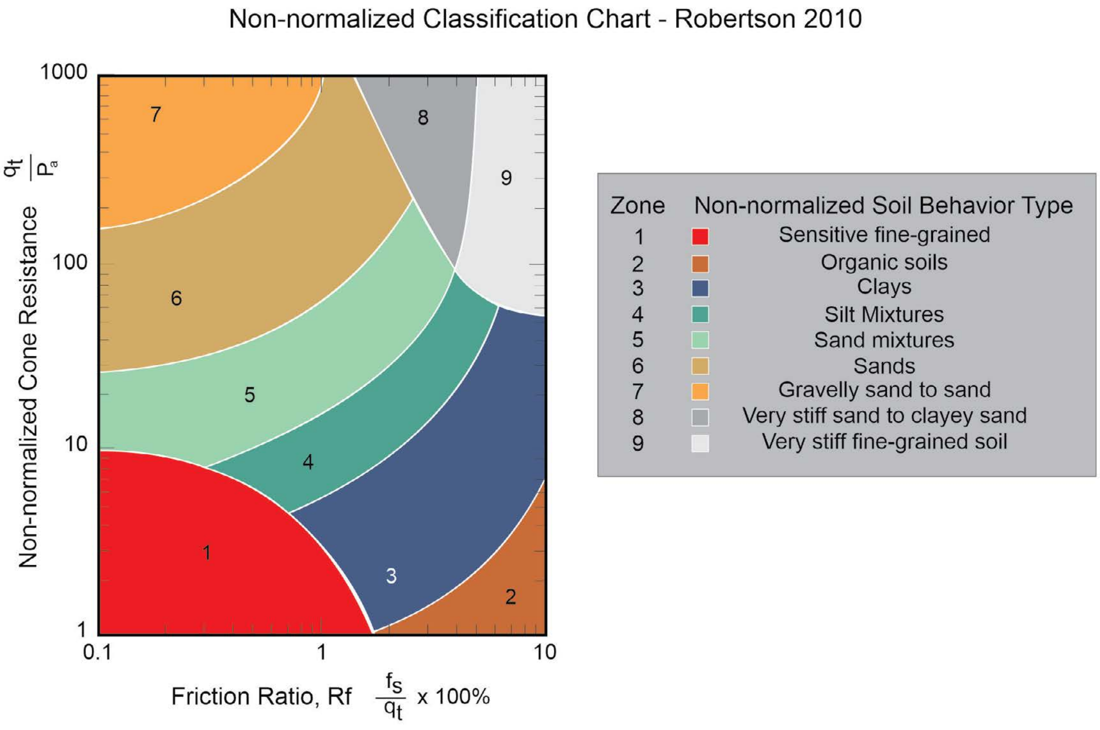


Figure SBT. Non-Normalized Soil Behavior Type Classification Chart (SBT)

The recorded tip resistance (q_c) is the total force acting on the piezocone tip divided by its base area. The tip resistance is corrected for pore pressure effects and termed corrected tip resistance (q_t) according to the following expression presented in Robertson et al. (1986):

$$q_t = q_c + (1\text{-}a) \cdot u_2$$

where: q_t is the corrected tip resistance
q_c is the recorded tip resistance
u_2 is the recorded dynamic pore pressure behind the tip (u_2 position)
a is the Net Area Ratio for the piezocone (0.8 for ConeTec probes)

The sleeve friction (f_s) is the frictional force on the sleeve divided by its surface area. As all ConeTec piezocones have equal end area friction sleeves, pore pressure corrections to the sleeve data are not required.

The dynamic pore pressure (u) is a measure of the pore pressures generated during cone penetration. To record equilibrium pore pressure, the penetration must be stopped to allow the dynamic pore pressures to stabilize. The rate at which this occurs is predominantly a function of the permeability of the soil and the diameter of the cone.

The friction ratio (Rf) is a calculated parameter. It is defined as the ratio of sleeve friction to the tip resistance expressed as a percentage. Generally, saturated cohesive soils have low tip resistance, high friction ratios and generate large excess pore water pressures. Cohesionless soils have higher tip resistances, lower friction ratios and do not generate significant excess pore water pressure.

For additional information on CPTu interpretations and calculated geotechnical parameters, refer to Robertson et al. (1986), Lunne et al. (1997), Robertson (2009), Mayne (2013, 2014) and Mayne and Peuchen (2012).

REFERENCES

ASTM D5778-20, 2020, “Standard Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils”, ASTM International, West Conshohocken, PA. DOI: 10.1520/D5778-20.

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Robertson, P.K., Campanella, R.G., Gillespie, D. and Greig, J., 1986, “Use of Piezometer Cone Data”, Proceedings of InSitu 86, ASCE Specialty Conference, Blacksburg, Virginia.

Robertson, P.K., 2009, “Interpretation of cone penetration tests – a unified approach”, Canadian Geotechnical Journal, Volume 46: 1337-1355. DOI: 10.1139/T09-065.

Robertson, P.K., 2010. Soil behavior type from the CPT: an update. 2nd International Symposium on Cone Penetration Testing, CPT’10, Huntington Beach, CA, USA

PORE PRESSURE DISSIPATION TEST

The cone penetration test is halted at specific depths to carry out pore pressure dissipation (PPD) tests, shown in Figure PPD-1. For each dissipation test the cone and rods are decoupled from the rig and the data acquisition system measures and records the variation of the pore pressure (u) with time (t).

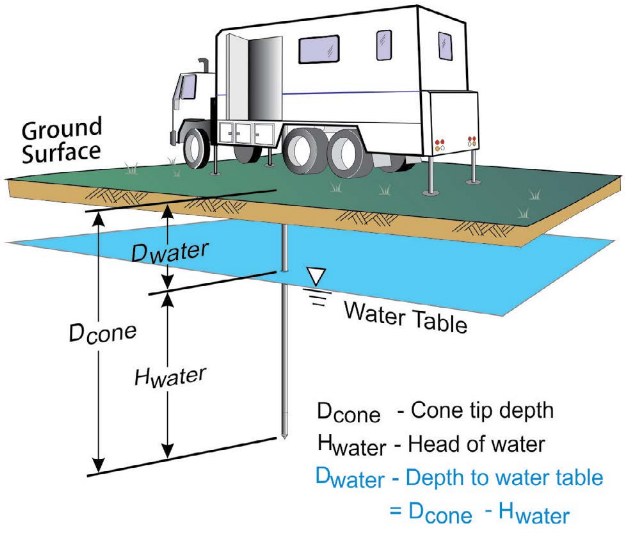

Figure PPD-1. Pore pressure dissipation test setup

Pore pressure dissipation data can be interpreted to provide estimates of ground water conditions, permeability, consolidation characteristics and soil behavior.

The typical shapes of dissipation curves shown in Figure PPD-2 are very useful in assessing soil type, drainage, in situ pore pressure and soil properties. A flat curve that stabilizes quickly is typical of a freely draining sand. Undrained soils such as clays will typically show positive excess pore pressure and have long dissipation times. Dilative soils will often exhibit dynamic pore pressures below equilibrium that then rise over time. Overconsolidated fine-grained soils will often exhibit an initial dilatory response where there is an initial rise in pore pressure before reaching a peak and dissipating.

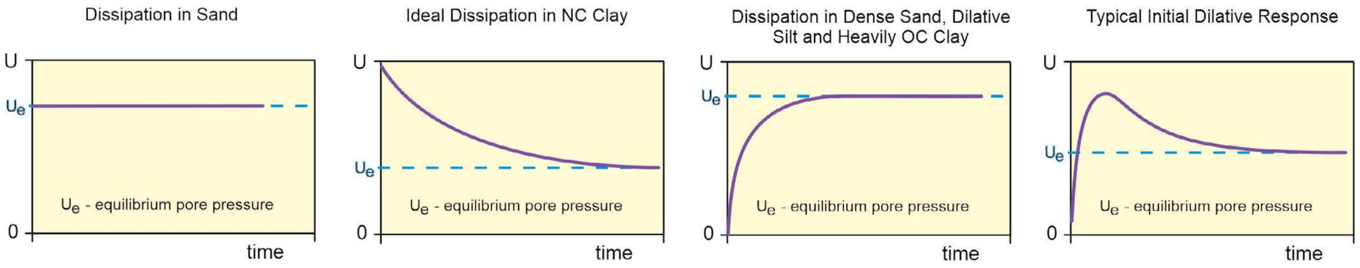

Figure PPD-2. Pore pressure dissipation curve examples

In order to interpret the equilibrium pore pressure (u_{eq}) and the apparent phreatic surface, the pore pressure should be monitored until such time as there is no variation in pore pressure with time as shown for each curve in Figure PPD-2.

SEISMIC CONE PENETRATION TEST (SCPTu) - eSeries

Shear wave velocity (Vs) testing is performed in conjunction with the piezocone penetration test (SCPTu) in order to collect interval velocities. For some projects seismic compression wave velocity (Vp) testing is also performed.

ConeTec's piezocone penetrometers are manufactured with one horizontally active geophone (28 hertz) and one vertically active geophone (28 hertz). Both geophones are rigidly mounted in the body of the cone penetrometer, 0.2 meters behind the cone tip. The vertically mounted geophone is more sensitive to compression waves.

Shear waves are typically generated by using an impact hammer horizontally striking a beam that is held in place by a normal load. In some instances, an auger source or an imbedded impulsive source may be used for both shear waves and compression waves. The hammer and beam act as a contact trigger that initiates the recording of the seismic wave traces. For impulsive devices an accelerometer trigger may be used. The traces are recorded in the memory of the cone using a fast analog to digital converter. The seismic trace is then transmitted digitally uphole to a Windows based computer through a signal interface box for recording and analysis. An illustration of the shear wave testing configuration is presented in Figure SCPTu-1.

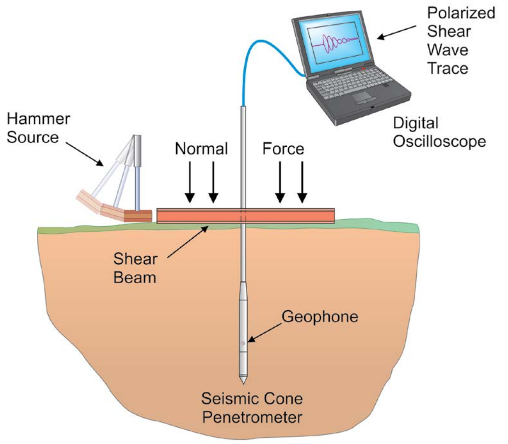

Figure SCPTu-1. Illustration of the SCPTu system

All testing is performed in accordance to ConeTec's SCPTu operating procedures which are in general accordance with the current ASTM D5778 and ASTM D7400 standards.

Prior to the start of a SCPTu sounding, the procedures described in the Cone Penetration Test section are followed. In addition, the active axis of the geophone is aligned parallel to the beam (or source) and the horizontal offset between the cone and the source is measured and recorded.

Prior to recording seismic waves at each test depth, cone penetration is stopped and the rods are decoupled from the rig to avoid transmission of rig energy down the rods. Typically, five wave traces for each orientation are recorded for quality control and uncertainty analysis purposes. After reviewing wave traces for consistency the cone is pushed to the next test depth (typically one meter intervals or as requested by the client). Figure SCPTu-2 presents an illustration of a SCPTu test.

For additional information on seismic cone penetration testing refer to Robertson et al. (1986).

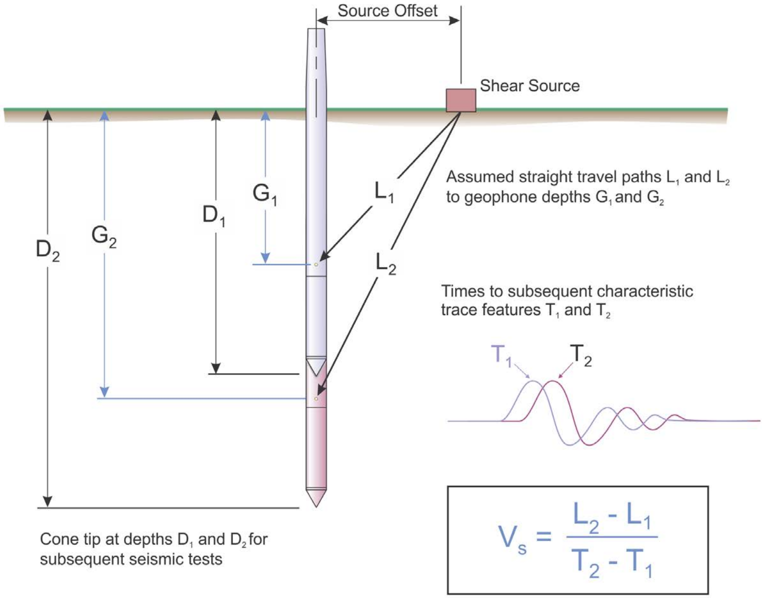

Figure SCPTu-2. Illustration of a seismic cone penetration test

For the determination of interval travel times the wave traces from all depths are displayed in analysis software. The results of the interval picks are supplied in the relevant appendix of this report. Standard practice for ConeTec is to record five wave traces for each source direction at each test depth. Outlier impacts are identified in the field and the impacts are repeated. For the final wave trace profile, the traces are stacked in the time domain to display a single average trace.

Calculation of the interval velocities are performed by visually picking a common feature (e.g. the first characteristic peak, trough, or crossover) on all of the recorded wave sets and taking the difference in ray path divided by the time difference between subsequent features. Ray path is defined as the straight line distance from the seismic source to the geophone, accounting for beam offset, source depth and geophone offset from the cone tip.

In some cases, usually for shear wave velocity testing, more than one characteristic marker may be used. If there is an overlap between different sets of characteristic markers, then the average time value for those sets of interval times is applied to the determination of velocity.

Ideally, all depths are used for the determination of the velocity profile. However, an interval may be skipped if there is some ambiguity or quality concern with a particular depth, resulting in a larger interval.

Tabular velocity results and SCPTu plots are presented in the relevant appendix.

For all SCPTu soundings that have achieved a depth of at least 100 feet (30 meters), the average shear wave velocity to a depth of 100 feet $(\overline{v}_S)$ has been calculated and provided for all applicable soundings using the following equation presented in ASCE (2010).

$$\bar{v}_s = \frac{\sum_{i=1} d_i}{\sum_{i=1}^{n} \frac{d_i}{v_{si}}}$$

where: $\bar{v}_S$ = average shear wave velocity ft/s (m/s)
d_i = the thickness of any layer between 0 and 100 ft (30 m)
v_{si} = the shear wave velocity in ft/s (m/s)
$\sum_{i=1}^{n} d_i$ = the total thickness of all layers between 0 and 100 ft (30 m)

Average shear wave velocity, $\bar{v}_S$ is also referenced to V_S100 or V_S30.

The layer travel times refers to the travel times propagating in the vertical direction, not the measured travel times from an offset source.

REFERENCES

American Society of Civil Engineers (ASCE), 2010, “Minimum Design Loads for Buildings and Other Structures”, Standard ASCE/SEI 7-10, American Society of Civil Engineers, ISBN 978-0-7844-1085-1, Reston, Virginia. DOI: 10.1061/9780784412916.

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ASTM D7400/D7400M-19, 2019, “Standard Test Methods for Downhole Seismic Testing”, ASTM International, West Conshohocken, PA. DOI: 10.1520/D7400_D7400M-19.

Robertson, P.K., Campanella, R.G., Gillespie D and Rice, A., 1986, “Seismic CPT to Measure In-Situ Shear Wave Velocity”, Journal of Geotechnical Engineering ASCE, Vol. 112, No. 8: 791-803. DOI: 10.1061/(ASCE)0733-9410(1986)112:8(791).

CONE PENETRATION DIGITAL FILE FORMATS - eSeries

CPT Data Files (COR Extension)

ConeTec CPT data files are stored in ASCII text files that are readable by almost any text editor. ConeTec file names start with the job number (which includes the two digit year number) an underscore as a separating character, followed by two letters based on the type of test and the sounding ID. The last character position is reserved for an identifier letter (such as b, c, d etc) used to uniquely distinguish multiple soundings at the same location. The CPT sounding file has the extension COR. As an example, for job number 21-02-00001 the first CPT sounding will have file name 21-02-00001_CP01.COR

The sounding (COR) file consists of the following components:

1. Two lines of header information
2. Data records
3. End of data marker
4. Units information

Header Lines

Line 1: Columns 1-6 may be blank or may indicate the version number of the recording software
Columns 7-21 contain the sounding Date and Time (Date is MM:DD:YY)
Columns 23-38 contain the sounding Operator
Columns 51-100 contain extended Job Location information

Line 2: Columns 1-16 contain the Job Location
Columns 17-32 contain the Cone ID
Columns 33-47 contain the sounding number
Columns 51-100 may contain extended sounding ID information

Data Records

The data records contain 4 or more columns of data in floating point format. A comma and spaces separate each data item:

Column 1: Sounding Depth (meters)
Column 2: Tip (q_c), recorded in units selected by the operator
Column 3: Sleeve (f_s), recorded in units selected by the operator
Column 4: Dynamic pore pressure (u), recorded in units selected by the operator
Column 5: Empty or may contain other requested data such as Gamma, Resistivity or UVIF data

End of Data Marker

After the last line of data there is a line containing an ASCII 26 (CTL-Z) character (small rectangular shaped character) followed by a newline (carriage return / line feed). This is used to mark the end of data.

Units Information

The last section of the file contains information about the units that were selected for the sounding. A separator bar makes up the first line. The second line contains the type of units used for depth, q_c, f_s and u. The third line contains the conversion values required for ConeTec's software to convert the recorded data to an internal set of base units (bar for q_c, bar for f_s and meters for u). Additional lines intended for internal ConeTec use may appear following the conversion values.

CPT Data Files (XLS Extension)

Excel format files of ConeTec CPT data are also generated from corresponding COR files. The XLS files have the same base file name as the COR file with a -BSC suffix. The information in the file is presented in table format and contains additional information about the sounding such as coordinate information, and tip net area ratio.

The BSCI suffix is given to XLS files which are enhanced versions of the BSC files and include the same data records in addition to inclination data collected for each sounding.

CPT Dissipation Files (XLS Extension)

Pore pressure dissipation files are provided in Excel format and contain each dissipation trace that exceeds a minimum duration (selected during post-processing) formatted column wise within the spreadsheet. The first column (Column A) contains the time in seconds and the second column (Column B) contains the time in minutes. Subsequent columns contain the dissipation trace data. The columns extend to the longest trace of the data set.

Detailed header information is provided at the top of the worksheet. The test depth in meters and feet, the number of points in the trace and the particular units are all presented at the top of each trace column.

CPT Dissipation files have the same naming convention as the CPT sounding files with a "–PPD" suffix.

Data Records

Each file will contain dissipation traces that exceed a minimum duration (selected during post-processing) in a particular column. The dissipation pore pressure values are typically recorded at varying time intervals throughout the trace; rapidly to start and increasing as the duration of the test lengthens. The test depth in meters and feet, the number of points in the trace and the trace number are identified at the top of each trace column.

Cone Type Designations

Cone ID	Cone Description	Tip Cross Sect. Area (cm^2)	Tip Capacity (bar)	Sleeve Area (cm^2)**	Sleeve Capacity (bar)	Pore Pressure Capacity (bar)
EC###	A15T1500F15U35	15	1500	225	15	35
EC###	A15T375F10U35	15	375	225	10	35
EC###	A10T1000F10U35	10	1000	150	10	35

refers to the Cone ID number
**Outer Cylindrical Area

REPORT APPENDICES

The appendices listed below are included in the report:

- **Cone Penetration Test (CPTu) Summary and Standard CPTu Plots**
- **Advanced Cone Penetration Test Plots**
- **Soil Behavior Type (SBT) Scatter Plots**
- **Pore Pressure Dissipation Test (PPDT) Summary and PPDT Plots**
- **Seismic Cone Penetration Test (SCPTu) Tabular Results**
- **SCPTu Plots**
- **SCPTu Velocity Wave Traces**
- **Piezocone Baseline Summary and Calibration Sheets**
- **Description of Methods for Calculated CPTu Geotechnical Parameters**
- **Piezocone Calibration Sheets**

Cone Penetration Test (CPTu) Summary and Standard CPTu Plots

Job No: 24-59-27209
Client: Gannett Fleming
Project: NuStar Portland
Start Date: 2024-02-22
End Date: 2024-02-22

CONE PENETRATION TEST SUMMARY

Sounding ID	File Name	Date	Cone	Cone Area (cm^2)	Assumed Phreatic Surface[1] (ft)	Final Depth (ft)	Seismic Intervals	Northing[2]	Easting[2]	Refer to Notation Number
SCPT-1	24-59-27209_SP01	2024-02-22	859:T1500F15U35	15	13.5	50.69	16	45.59080	-122.77537	
SCPT-2	24-59-27209_SP02	2024-02-22	859:T1500F15U35	15	12.6	41.67	13	45.59087	-122.77652	
SCPT-3	24-59-27209_SP03	2024-02-22	859:T1500F15U35	15	4.6	20.67	7	45.59067	-122.77744	
Totals	3 Soundings					113.03 ft	36			

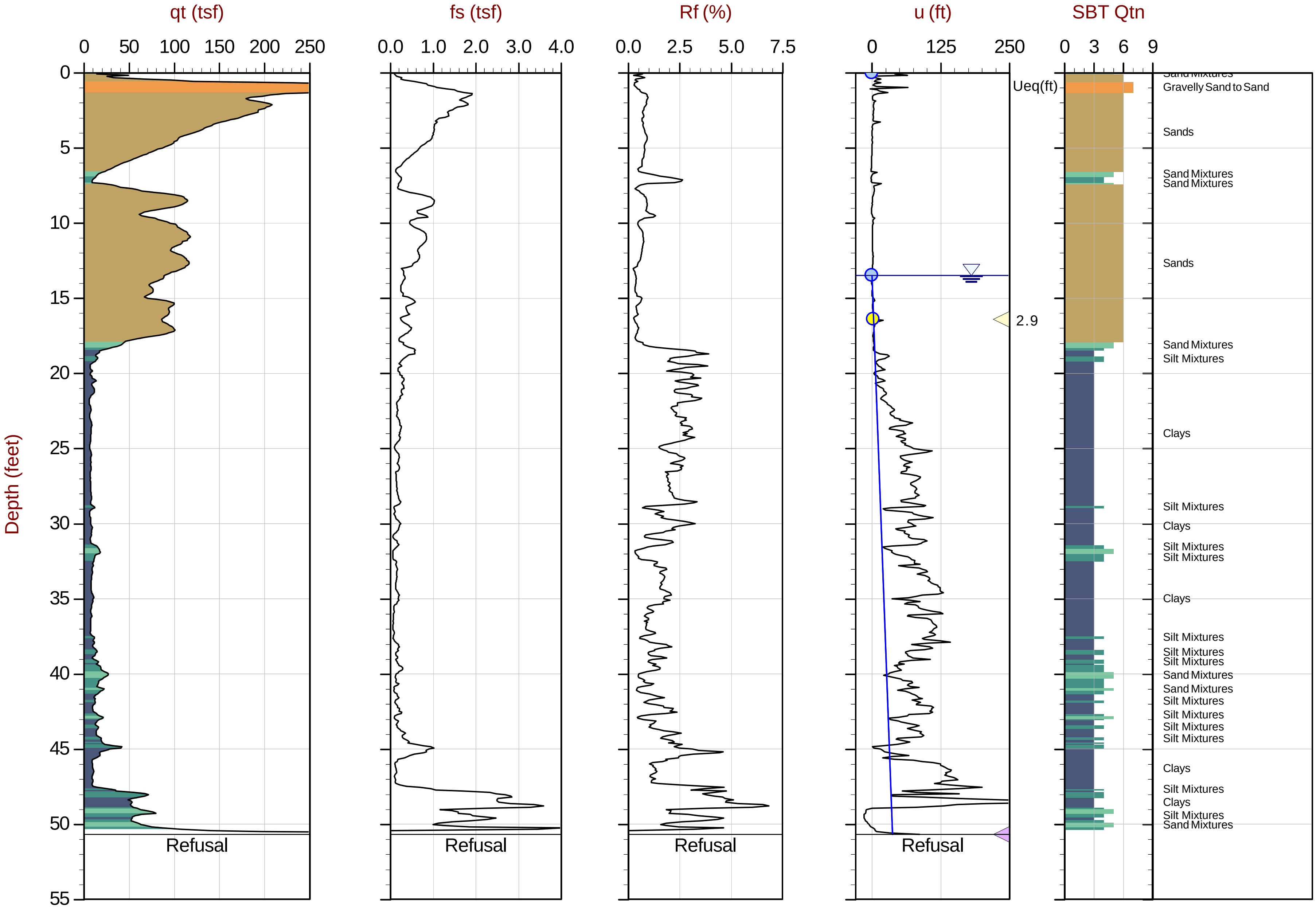
1. The assumed phreatic surface was based off the shallowest pore pressure dissipation tests performed within or nearest the sounding. Hydrostatic conditions were assumed for the calculated parameters.
2. The coordinates were collected using consumer grade GPS. EPSG number: 4326 (WGS84 / LatLong).

CONETEC

Gannett Fleming

Job No: 24-59-27209
Date: 2024-02-22 09:30
Site: NuStar Portland

Sounding: SCPT-1
Cone: 859:T1500F15U35



Max Depth: 15.450 m / 50.69 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 24-59-27209_SP01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: Lat: 45.59080 Long: -122.77537

● Equilibrium Pore Pressure (Ueq) ○ Assumed Ueq ◁ Dissipation, Ueq achieved ◁ Dissipation, Ueq not achieved — Hydrostatic Line

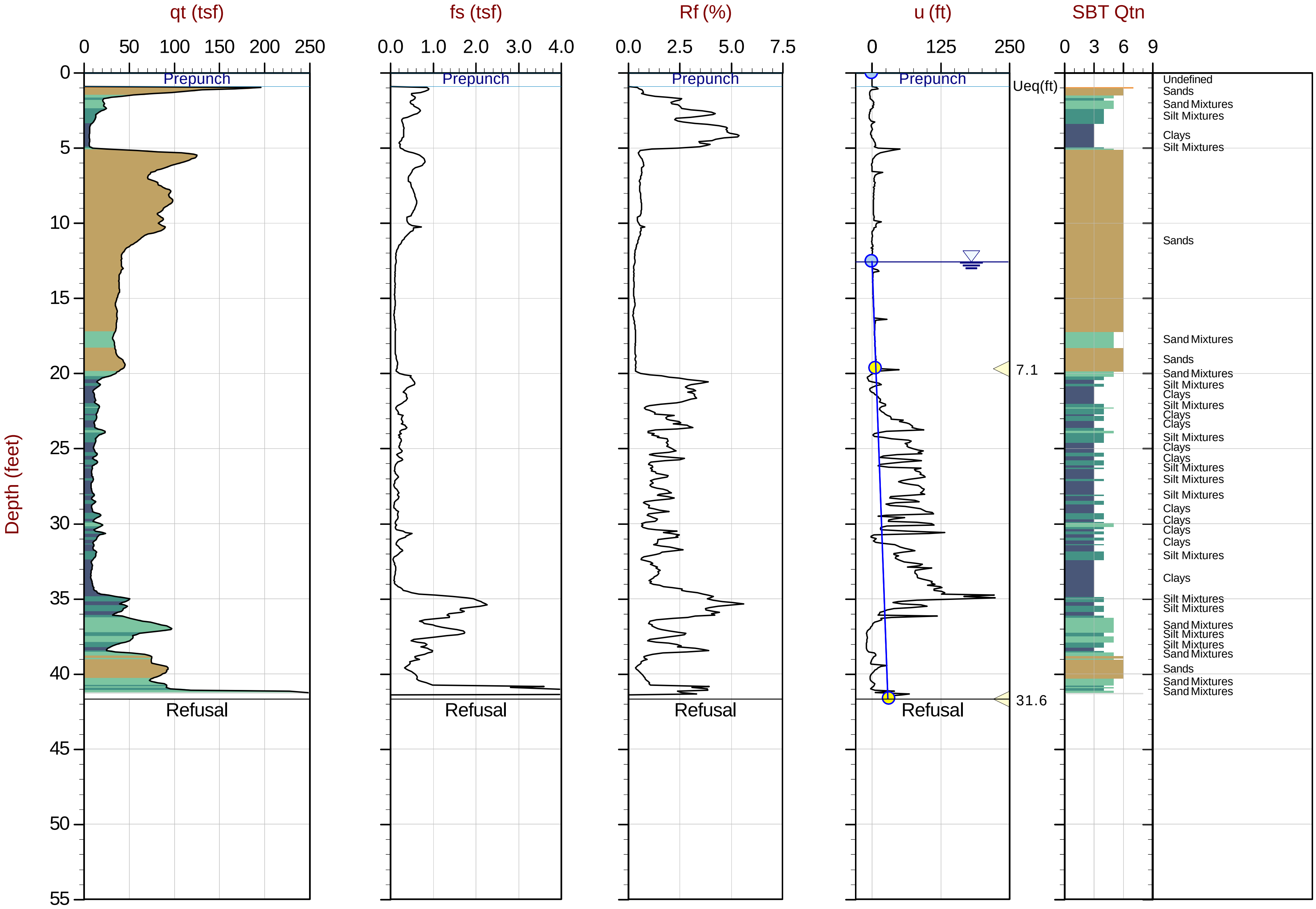
The reported coordinates were acquired from consumer-grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Gannett Fleming

Job No: 24-59-27209
Date: 2024-02-22 11:13
Site: NuStar Portland

Sounding: SCPT-2
Cone: 859:T1500F15U35



Max Depth: 12.700 m / 41.67 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 24-59-27209_SP02.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: Lat: 45.59087 Long: -122.77652

● Equilibrium Pore Pressure (Ueq) ● Assumed Ueq ◁ Dissipation, Ueq achieved ◁ Dissipation, Ueq not achieved — Hydrostatic Line

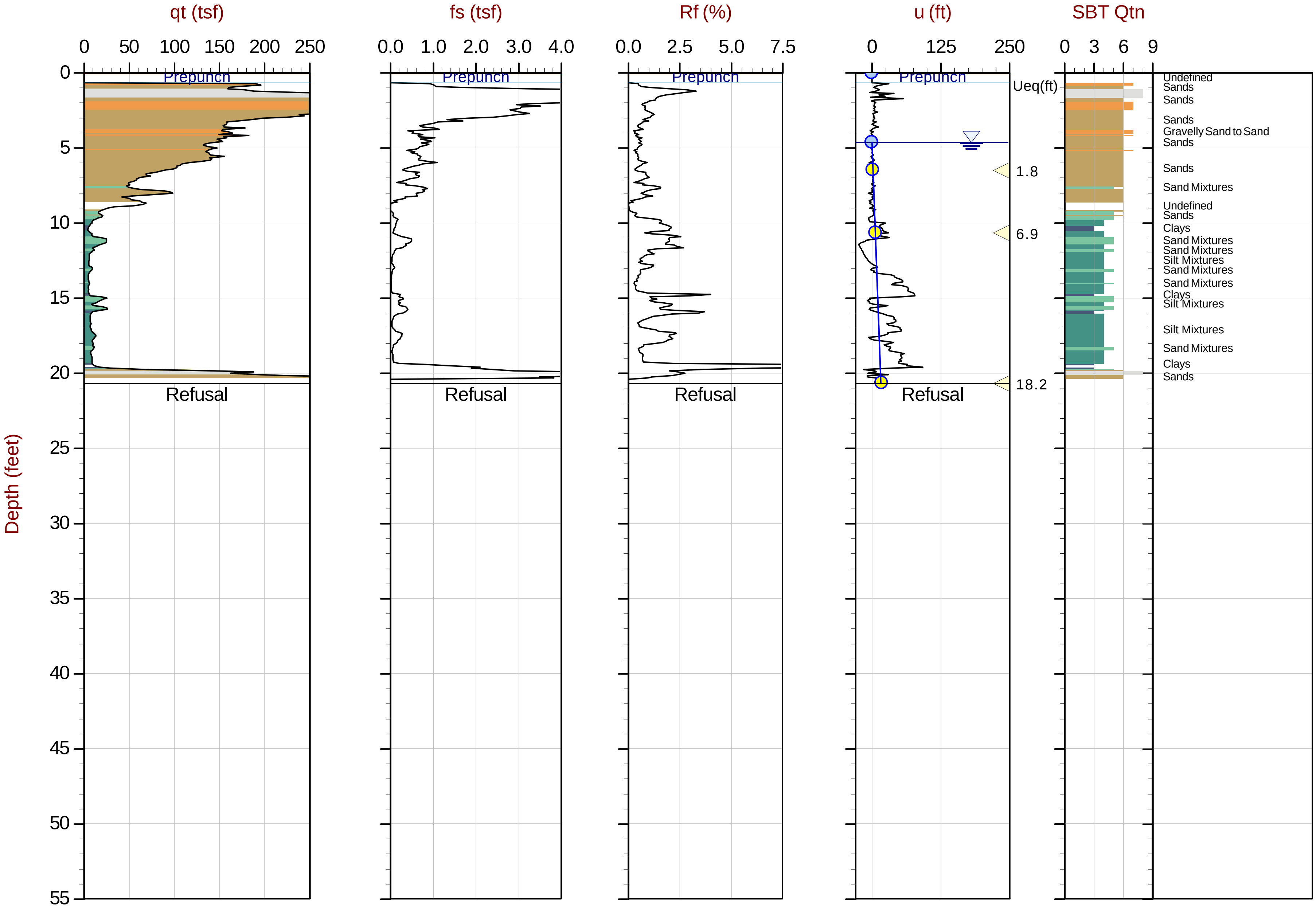
The reported coordinates were acquired from consumer-grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Gannett Fleming

Job No: 24-59-27209
Date: 2024-02-22 12:58
Site: NuStar Portland

Sounding: SCPT-3
Cone: 859:T1500F15U35



Max Depth: 6.300 m / 20.67 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 24-59-27209_SP03.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: Lat: 45.59067 Long: -122.77744

● Equilibrium Pore Pressure (Ueq) ● Assumed Ueq ◁ Dissipation, Ueq achieved ◁ Dissipation, Ueq not achieved — Hydrostatic Line

The reported coordinates were acquired from consumer-grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

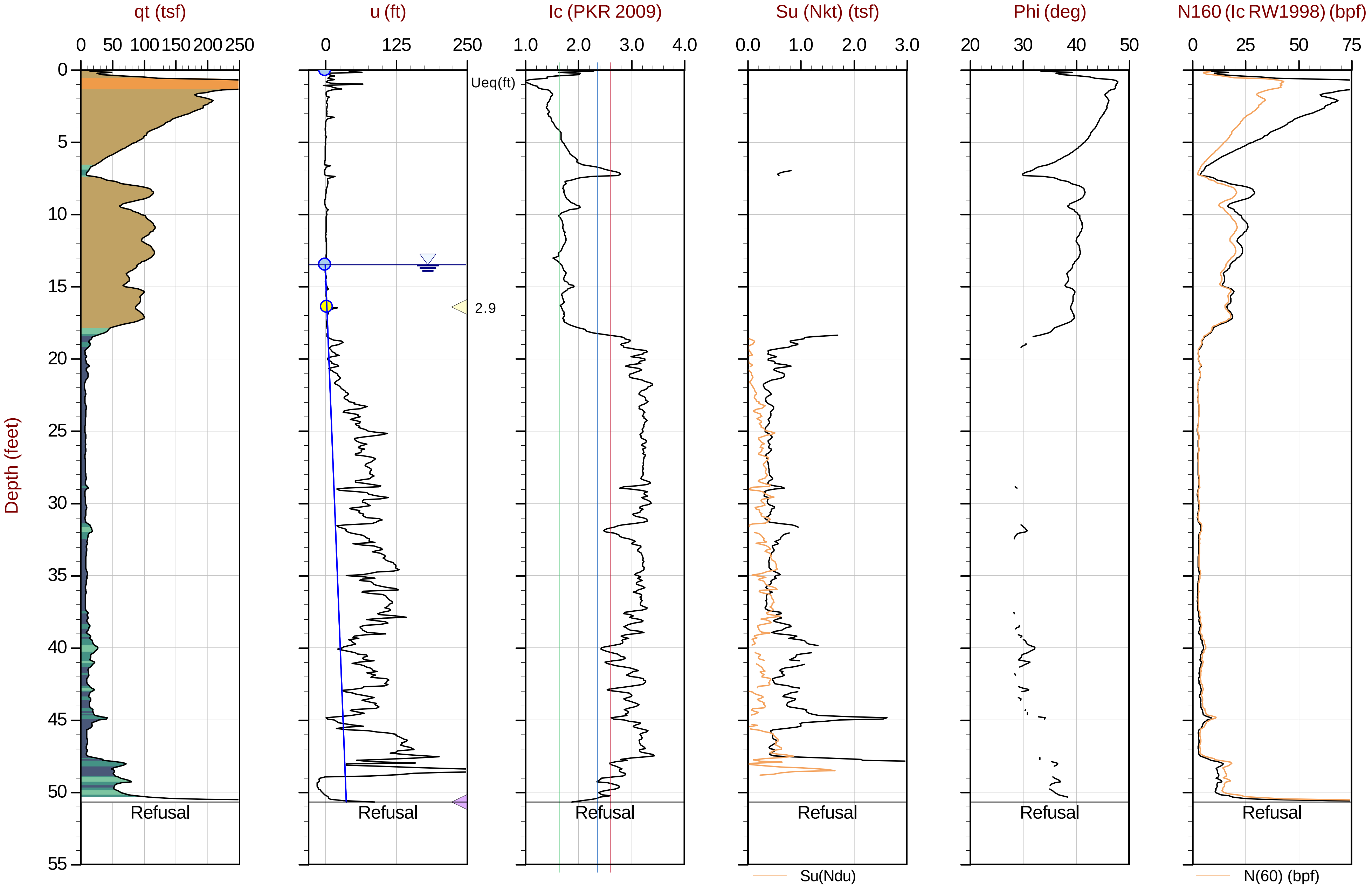
Advanced Cone Penetration Test Plots with Ic, Su(Nkt), Phi, and N1(60)Ic

CONETEC

Gannett Fleming

Job No: 24-59-27209
Date: 2024-02-22 09:30
Site: NuStar Portland

Sounding: SCPT-1
Cone: 859:T1500F15U35



Max Depth: 15.450 m / 50.69 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 24-59-27209_SP01.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt/Ndu: 15.0 / 6.0

SBT: Robertson, 2009 and 2010
Coords: Lat: 45.59080 Long: -122.77537

● Equilibrium Pore Pressure (Ueq) ● Assumed Ueq ◁ Dissipation, Ueq achieved ◁ Dissipation, Ueq not achieved — Hydrostatic Line

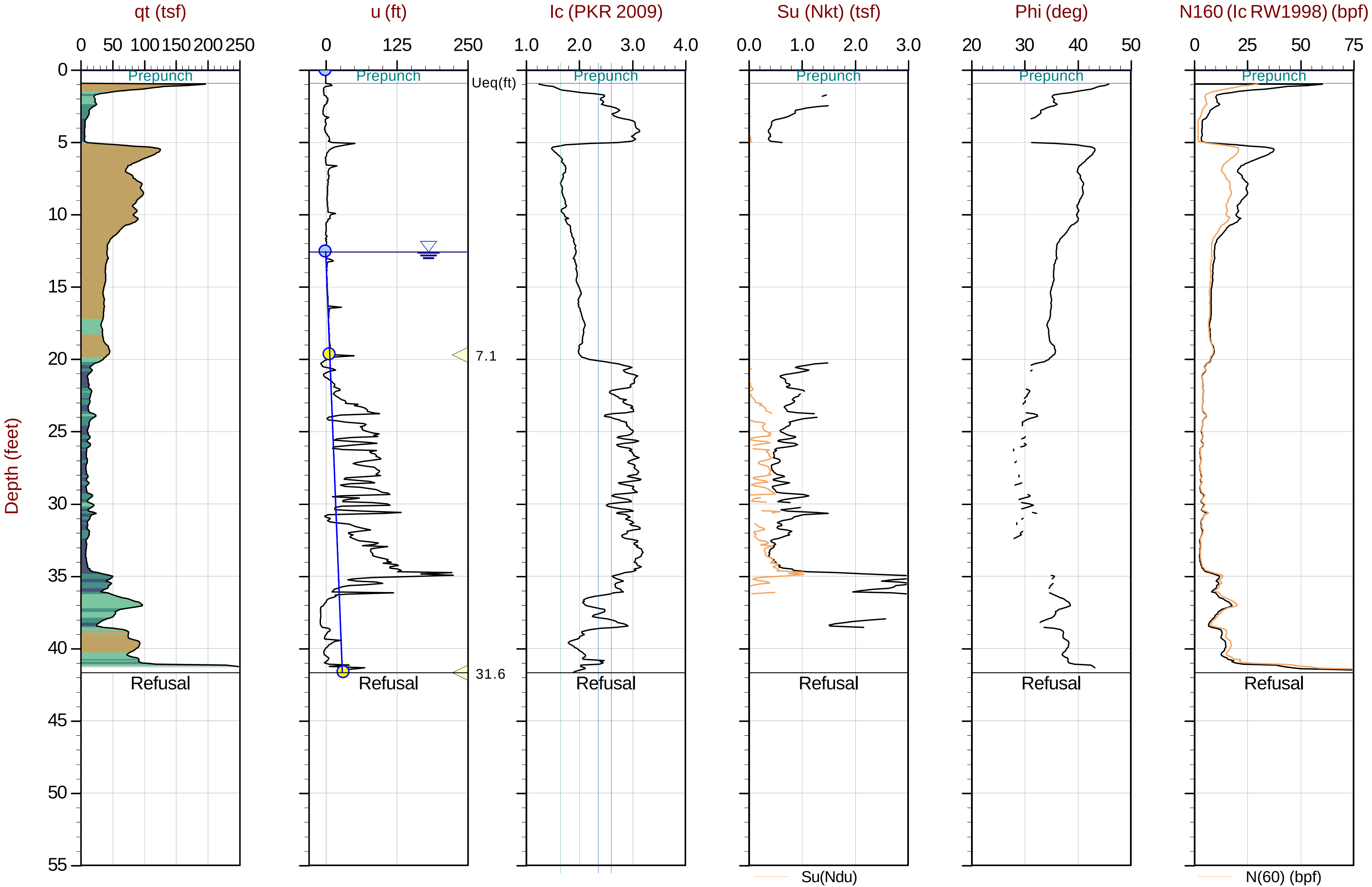
The reported coordinates were acquired from consumer-grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

CONETEC

Gannett Fleming

Job No: 24-59-27209
Date: 2024-02-22 11:13
Site: NuStar Portland

Sounding: SCPT-2
Cone: 859:T1500F15U35



Max Depth: 12.700 m / 41.67 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 24-59-27209_SP02.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt/Ndu: 15.0 / 6.0

SBT: Robertson, 2009 and 2010
Coords: Lat: 45.59087 Long: -122.77652

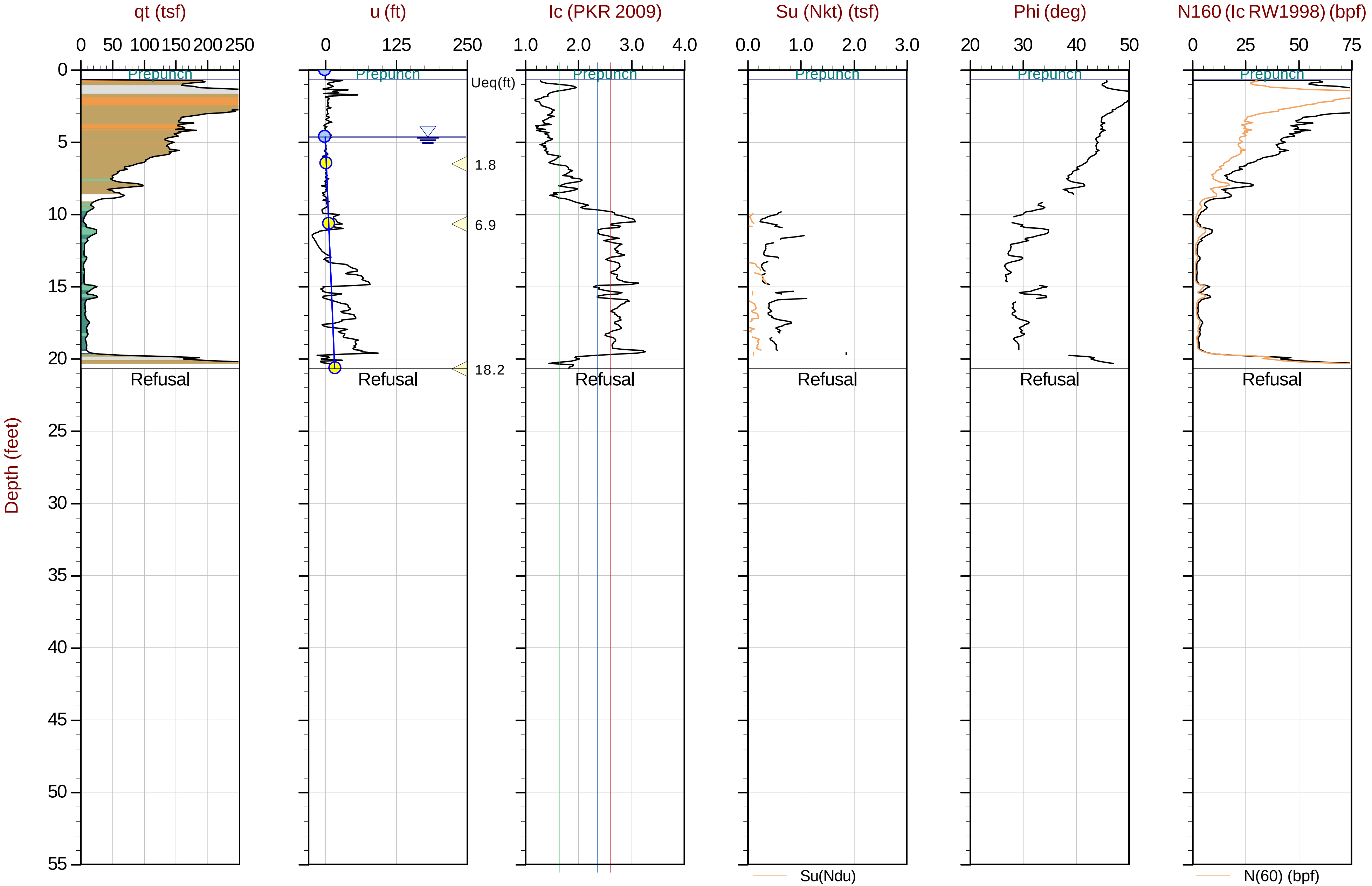
● Equilibrium Pore Pressure (Ueq) ● Assumed Ueq ◁ Dissipation, Ueq achieved ◁ Dissipation, Ueq not achieved — Hydrostatic Line

The reported coordinates were acquired from consumer-grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

Gannett Fleming

Job No: 24-59-27209
Date: 2024-02-22 12:58
Site: NuStar Portland

Sounding: SCPT-3
Cone: 859:T1500F15U35



Max Depth: 6.300 m / 20.67 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 24-59-27209_SP03.COR
Unit Wt: SBTQtn (PKR2009)
Su Nkt/Ndu: 15.0 / 6.0

SBT: Robertson, 2009 and 2010
Coords: Lat: 45.59067 Long: -122.77744

● Equilibrium Pore Pressure (Ueq) ● Assumed Ueq ◁ Dissipation, Ueq achieved ◁ Dissipation, Ueq not achieved — Hydrostatic Line

The reported coordinates were acquired from consumer-grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

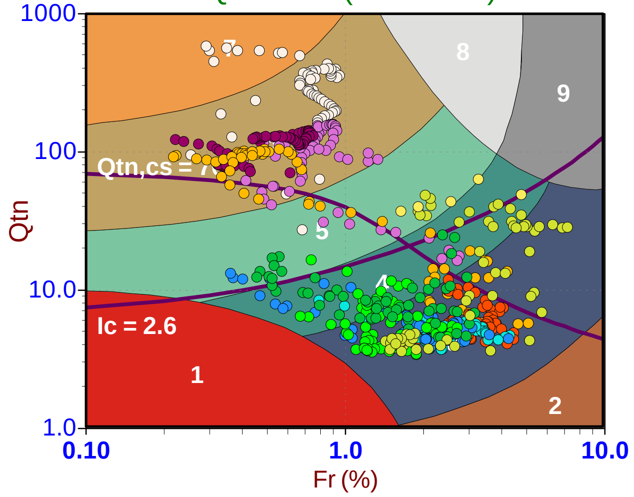
Soil Behavior Type (SBT) Scatter Plots

CONETEC

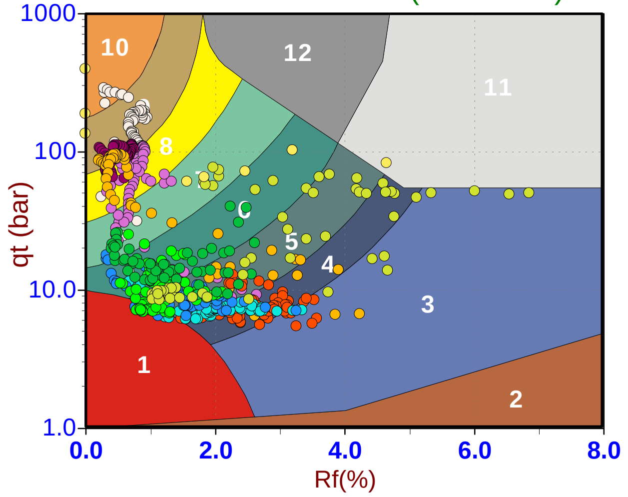
Gannett Fleming

Job No: 24-59-27209
Date: 2024-02-22 09:30
Site: NuStar Portland

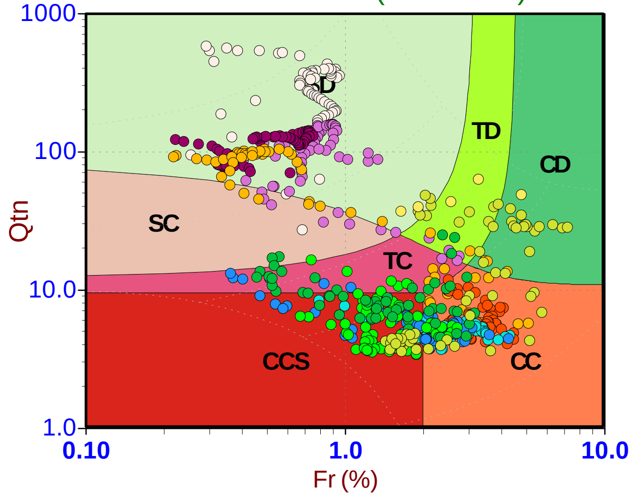
Sounding: SCPT-1
Cone: 859:T1500F15U35



Qtn Chart (PKR 2009)



Standard SBT Chart (UBC 1986)



Modified SBTn (PKR 2016)

Depth Ranges

- ● >0.0 to 5.0 ft
- ● >5.0 to 10.0 ft
- ● >10.0 to 15.0 ft
- ● >15.0 to 20.0 ft
- ● >20.0 to 25.0 ft
- ● >25.0 to 30.0 ft
- ● >30.0 to 35.0 ft
- ● >35.0 to 40.0 ft
- ● >40.0 to 45.0 ft
- ● >45.0 to 50.0 ft
- ● >50.0 ft

Legend

- ■ Sensitive, Fine Grained
- ■ Organic Soils
- ■ Clays
- ■ Silt Mixtures
- ■ Sand Mixtures
- ■ Sands
- ■ Gravelly Sand to Sand
- ■ Stiff Sand to Clayey Sand
- ■ Very Stiff Fine Grained

Legend

- ■ Sensitive Fines
- ■ Organic Soil
- ■ Clay
- ■ Silty Clay
- ■ Clayey Silt
- ■ Silt
- ■ Sandy Silt
- ■ Silty Sand/Sand
- ■ Sand
- ■ Gravelly Sand
- ■ Stiff Fine Grained
- ■ Cemented Sand

Legend

- ■ CCS (Cont. sensitive clay like)
- ■ CC (Cont. clay like)
- ■ TC (Cont. transitional)
- ■ SC (Cont. sand like)
- ■ CD (Dil. clay like)
- ■ TD (Dil. transitional)
- ■ SD (Dil. sand like)

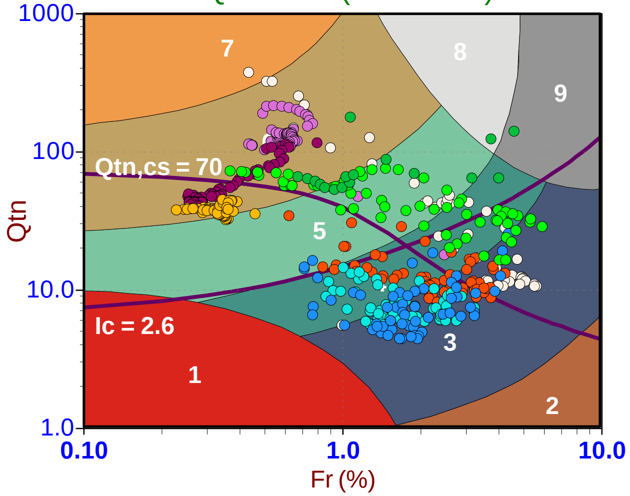
CONETEC

Gannett Fleming

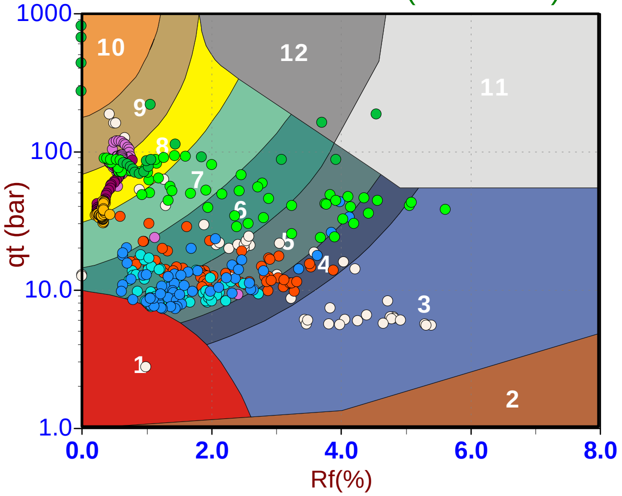
Job No: 24-59-27209
Date: 2024-02-22 11:13
Site: NuStar Portland

Sounding: SCPT-2
Cone: 859:T1500F15U35

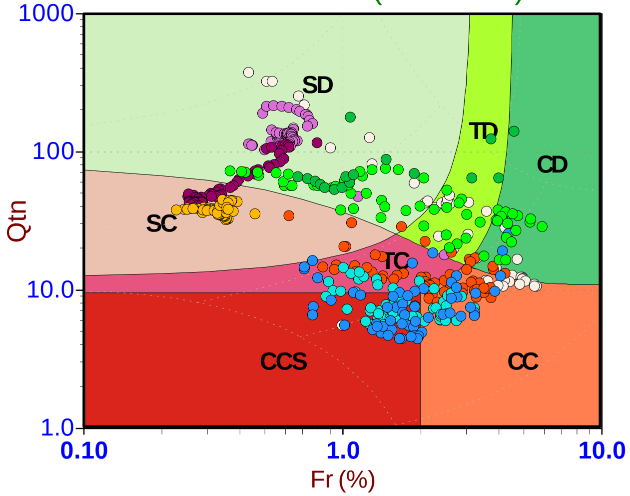
Qtn Chart (PKR 2009)



Standard SBT Chart (UBC 1986)



Modified SBTn (PKR 2016)



Depth Ranges
- ○ >0.0 to 5.0 ft
- ● >5.0 to 10.0 ft
- ● >10.0 to 15.0 ft
- ● >15.0 to 20.0 ft
- ● >20.0 to 25.0 ft
- ● >25.0 to 30.0 ft
- ● >30.0 to 35.0 ft
- ● >35.0 to 40.0 ft
- ● >40.0 to 45.0 ft
- ● >45.0 to 50.0 ft
- ● >50.0 ft

Legend
- 🟥 Sensitive, Fine Grained
- 🟫 Organic Soils
- 🟦 Clays
- 🟩 Silt Mixtures
- 🟩 Sand Mixtures
- 🟫 Sands
- 🟧 Gravelly Sand to Sand
- ⬜ Stiff Sand to Clayey Sand
- ⬛ Very Stiff Fine Grained

Legend
- 🟥 Sensitive Fines
- 🟫 Organic Soil
- 🟦 Clay
- 🟦 Silty Clay
- ⬛ Clayey Silt
- 🟩 Silt
- 🟩 Sandy Silt
- 🟨 Silty Sand/Sand
- 🟫 Sand
- 🟧 Gravelly Sand
- ⬜ Stiff Fine Grained
- ⬛ Cemented Sand

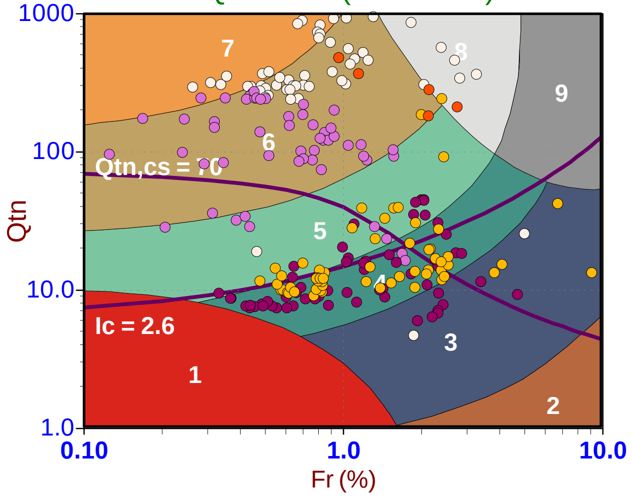
Legend
- 🟥 CCS (Cont. sensitive clay like)
- 🟧 CC (Cont. clay like)
- 🟥 TC (Cont. transitional)
- 🟫 SC (Cont. sand like)
- 🟩 CD (Dil. clay like)
- 🟩 TD (Dil. transitional)
- 🟩 SD (Dil. sand like)

Gannett Fleming

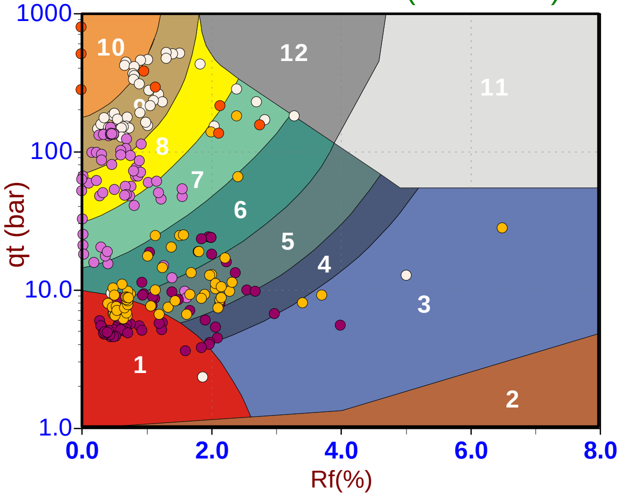
Job No: 24-59-27209
Date: 2024-02-22 12:58
Site: NuStar Portland

Sounding: SCPT-3
Cone: 859:T1500F15U35

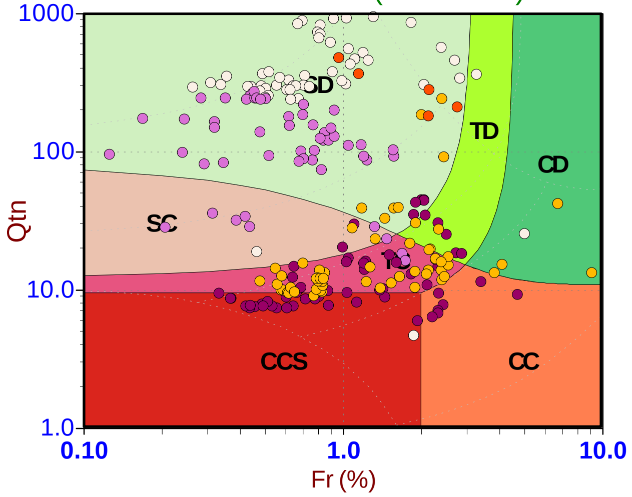
Qtn Chart (PKR 2009)



Standard SBT Chart (UBC 1986)



Modified SBTn (PKR 2016)



Depth Ranges

- ○ >0.0 to 5.0 ft
- ● >5.0 to 10.0 ft
- ● >10.0 to 15.0 ft
- ● >15.0 to 20.0 ft
- ● >20.0 to 25.0 ft
- ● >25.0 to 30.0 ft
- ● >30.0 to 35.0 ft
- ● >35.0 to 40.0 ft
- ● >40.0 to 45.0 ft
- ● >45.0 to 50.0 ft
- ● >50.0 ft

Legend

- 🟥 Sensitive, Fine Grained
- 🟫 Organic Soils
- 🟦 Clays
- 🟩 Silt Mixtures
- 🟩 Sand Mixtures
- 🟨 Sands
- 🟧 Gravelly Sand to Sand
- ⬜ Stiff Sand to Clayey Sand
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Legend

- 🟥 Sensitive Fines
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- 🟩 Clayey Silt
- 🟩 Silt
- 🟩 Sandy Silt
- 🟨 Silty Sand/Sand
- 🟨 Sand
- 🟧 Gravelly Sand
- ⬜ Stiff Fine Grained
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Legend

- 🟥 CCS (Cont. sensitive clay like)
- 🟧 CC (Cont. clay like)
- 🟥 TC (Cont. transitional)
- 🟫 SC (Cont. sand like)
- 🟩 CD (Dil. clay like)
- 🟩 TD (Dil. transitional)
- 🟩 SD (Dil. sand like)

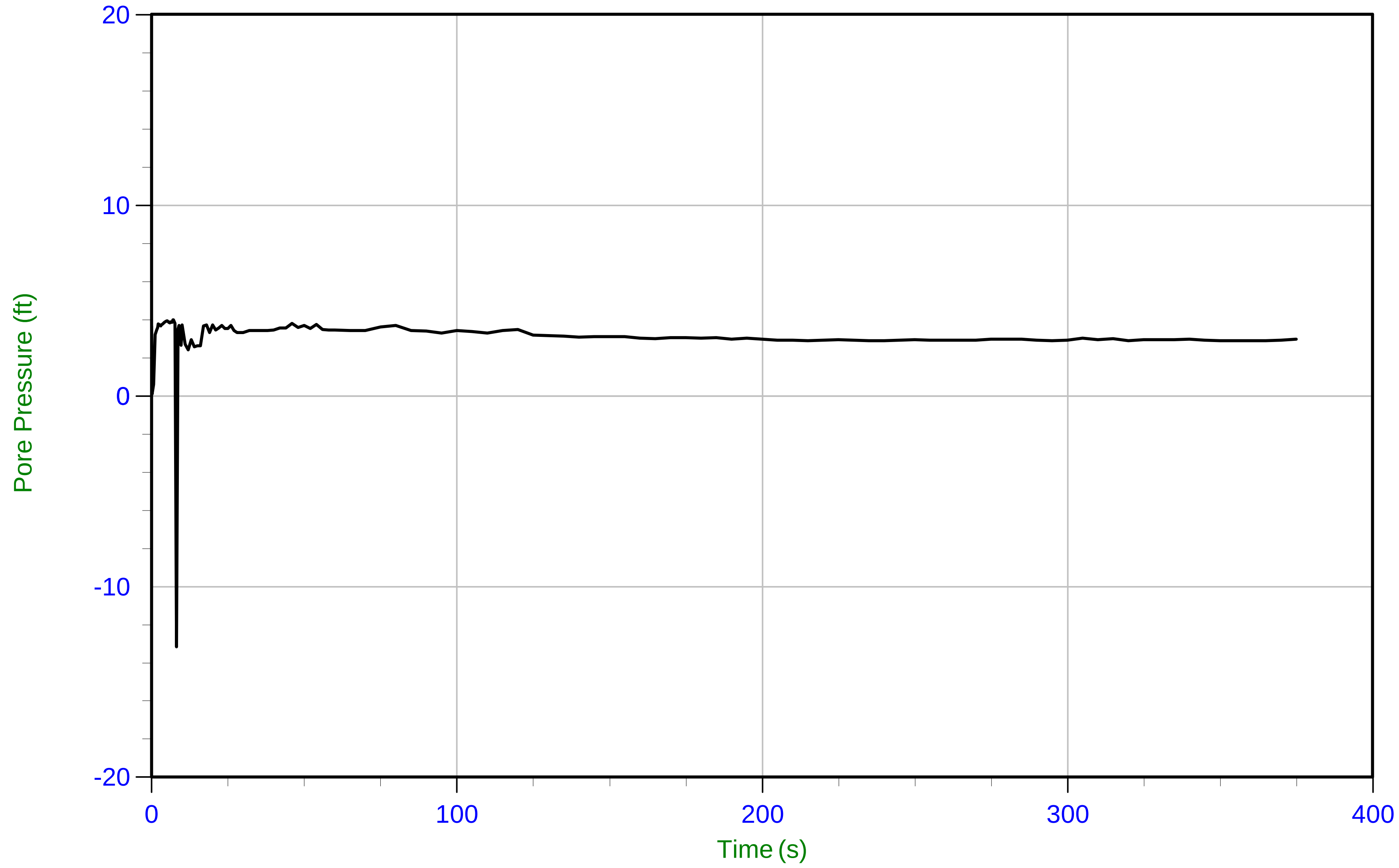
Pore Pressure Dissipation Test (PPDT) Summary and PPDT Plots

Job No: 24-59-27209
Client: Gannett Fleming
Project: NuStar Portland
Start Date: 2024-02-22
End Date: 2024-02-22

CPTu PORE PRESSURE DISSIPATION SUMMARY

Sounding ID	File Name	Cone Area (cm2)	Duration (s)	Test Depth (ft)	Estimated Equilibrium Pore Pressure Ueq (ft)	Calculated Phreatic Surface (ft)	Refer to Notation Number
SCPT-1	24-59-27209_SP01	15	375	16.40	2.9	13.5	
SCPT-1	24-59-27209_SP01	15	280	50.69			1
SCPT-2	24-59-27209_SP02	15	555	19.68	7.1	12.6	
SCPT-2	24-59-27209_SP02	15	180	41.67	31.6	10.1	
SCPT-3	24-59-27209_SP03	15	75	6.48	1.8	4.6	
SCPT-3	24-59-27209_SP03	15	345	10.66	6.9	3.8	
SCPT-3	24-59-27209_SP03	15	495	20.67	18.2	2.4	
Totals			38 min				

1. Equilibrium pore pressure was not achieved.



Trace Summary:

Filename: 24-59-27209_SP01.ppd2
Depth: 5.000 m / 16.404 ft
Duration: 375.0 s

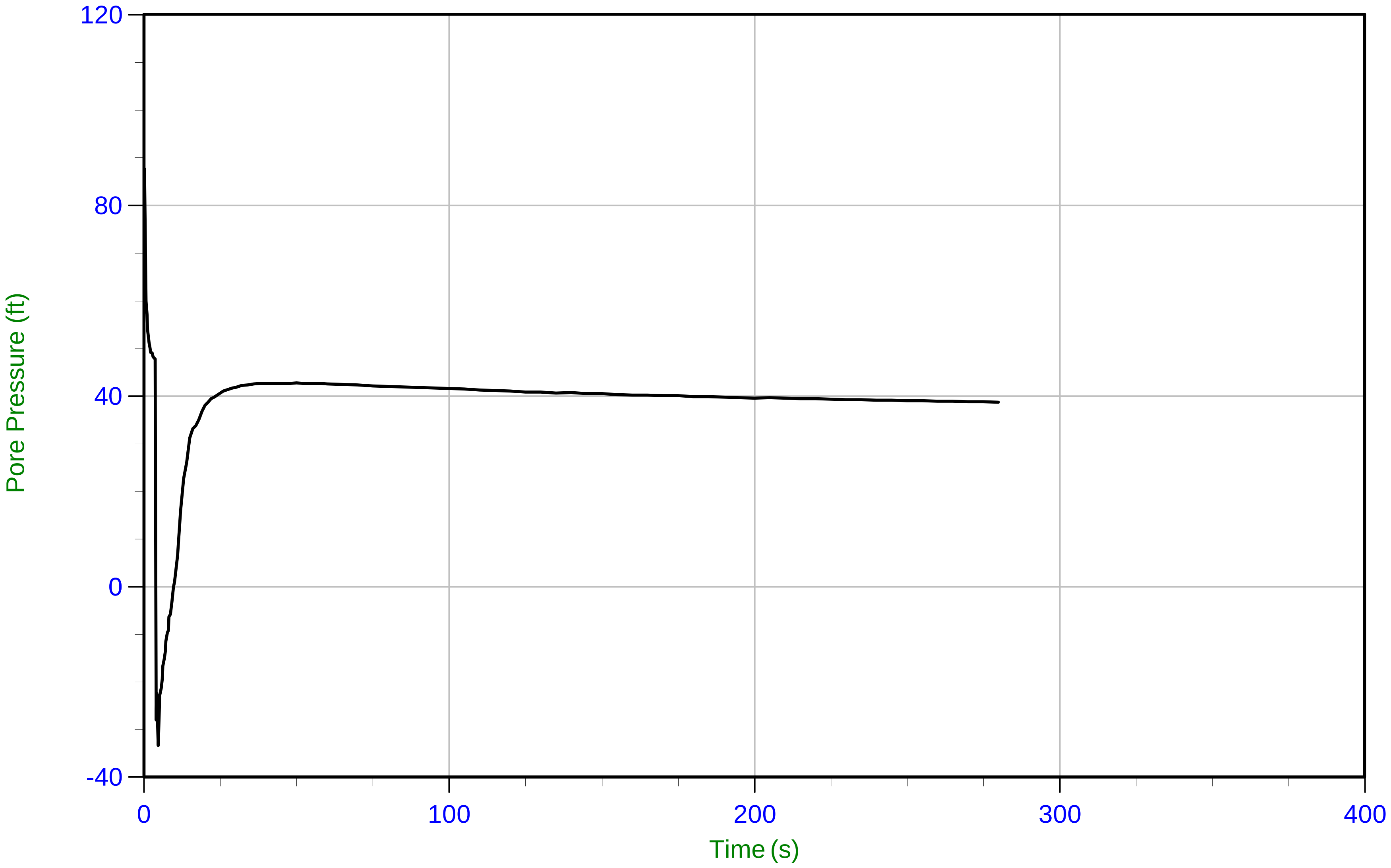
u Min: -13.2 ft
u Max: 4.0 ft
u Final: 3.0 ft

WT: 4.110 m / 13.484 ft
Ueq: 2.9 ft

Gannett Fleming

Job No: 24-59-27209
Date: 2024-02-22 09:30
Site: NuStar Portland

Sounding: SCPT-1
Cone: 859:T1500F15U35 Area=15 cm²



Trace Summary:

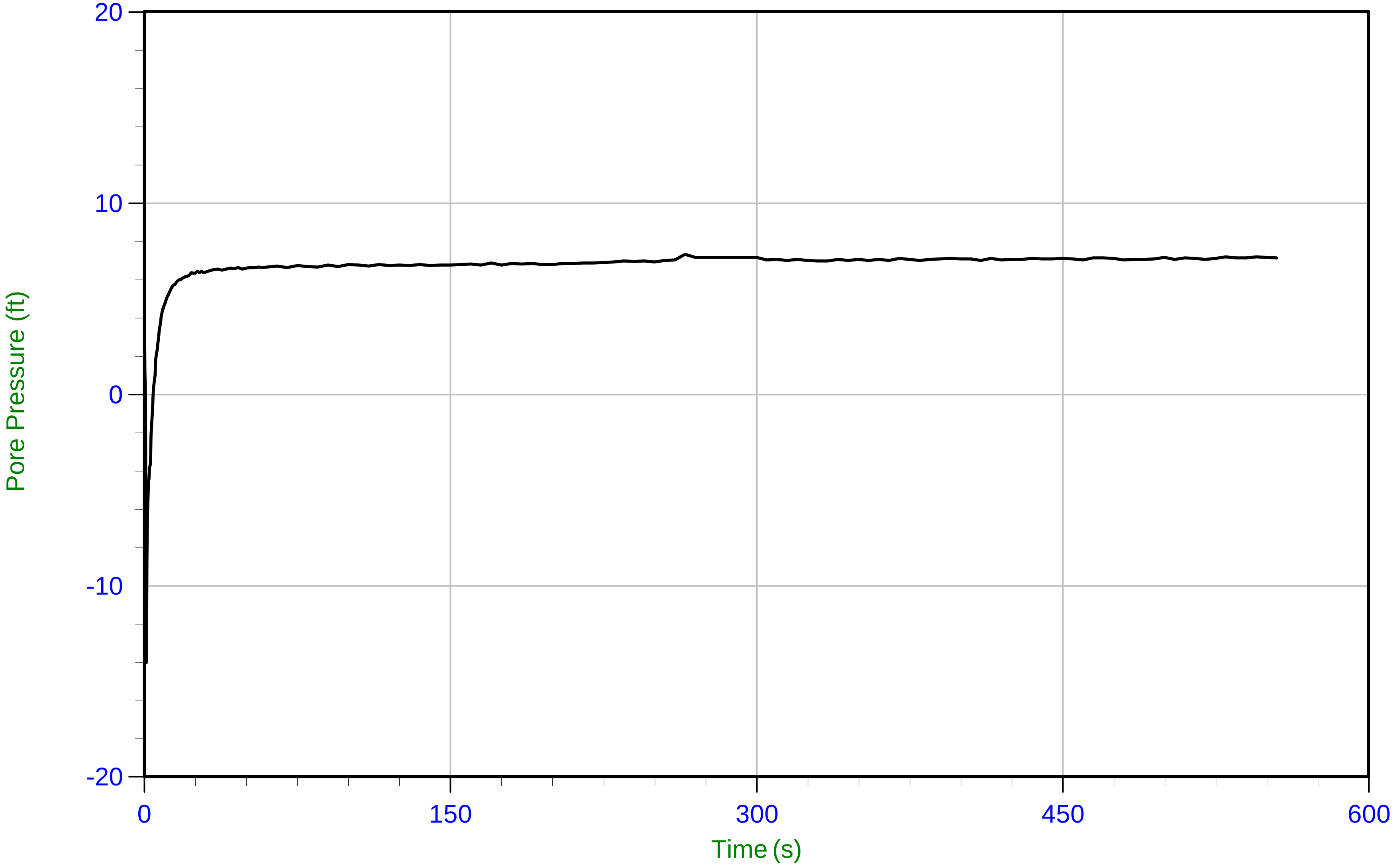
Filename: 24-59-27209_SP01.ppd2
Depth: 15.450 m / 50.688 ft
Duration: 280.0 s

u Min: -33.3 ft
u Max: 87.6 ft
u Final: 38.7 ft

Gannett Fleming

Job No: 24-59-27209
Date: 2024-02-22 11:13
Site: NuStar Portland

Sounding: SCPT-2
Cone: 859:T1500F15U35 Area=15 cm²

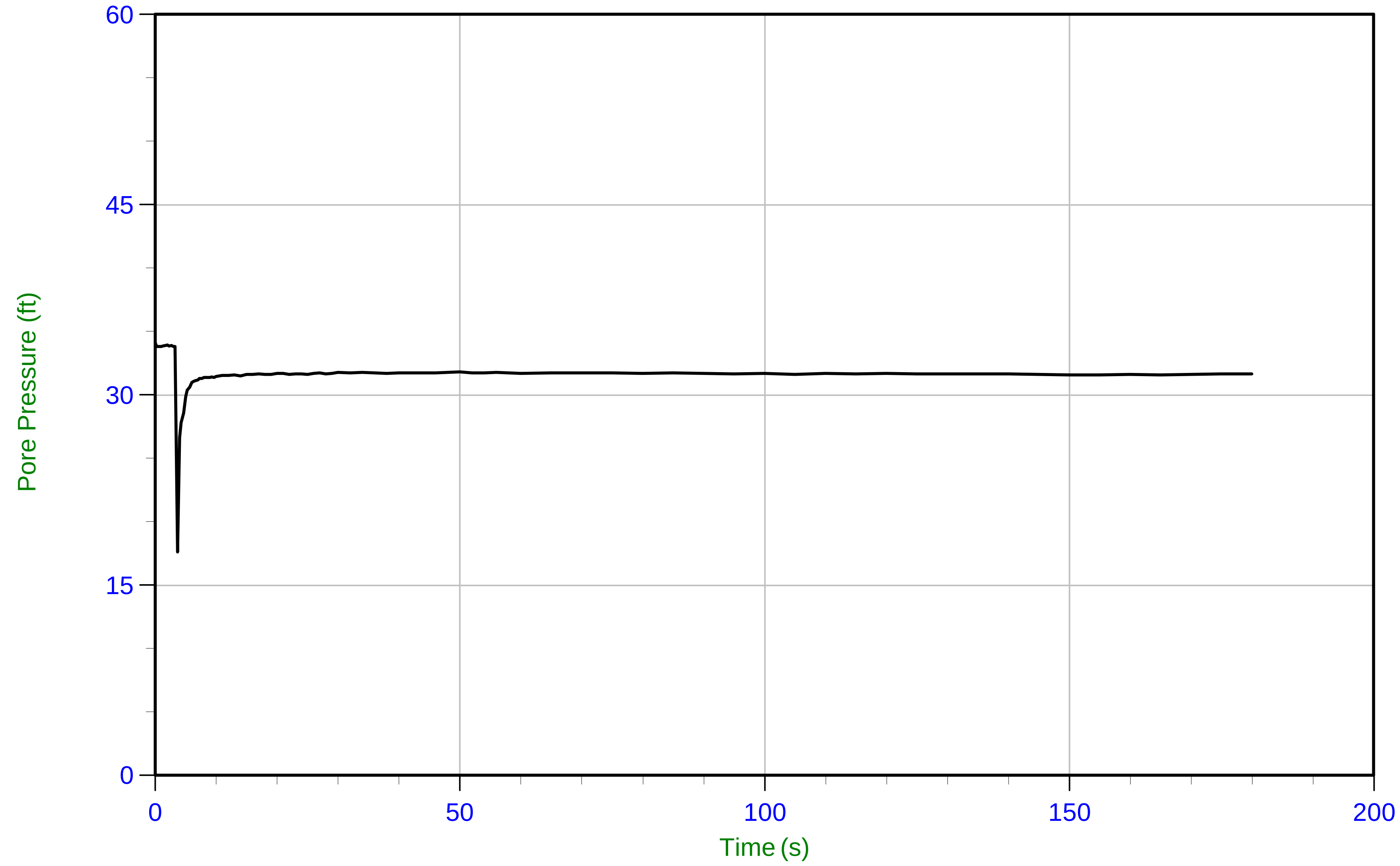


Trace Summary:

Filename: 24-59-27209_SP02.ppd2
Depth: 6.000 m / 19.685 ft
Duration: 555.0 s

u Min: -14.0 ft
u Max: 7.3 ft
u Final: 7.1 ft

WT: 3.832 m / 12.572 ft
Ueq: 7.1 ft



Trace Summary:

Filename: 24-59-27209_SP02.ppd2
Depth: 12.700 m / 41.666 ft
Duration: 180.0 s

u Min: 17.6 ft
u Max: 34.1 ft
u Final: 31.7 ft

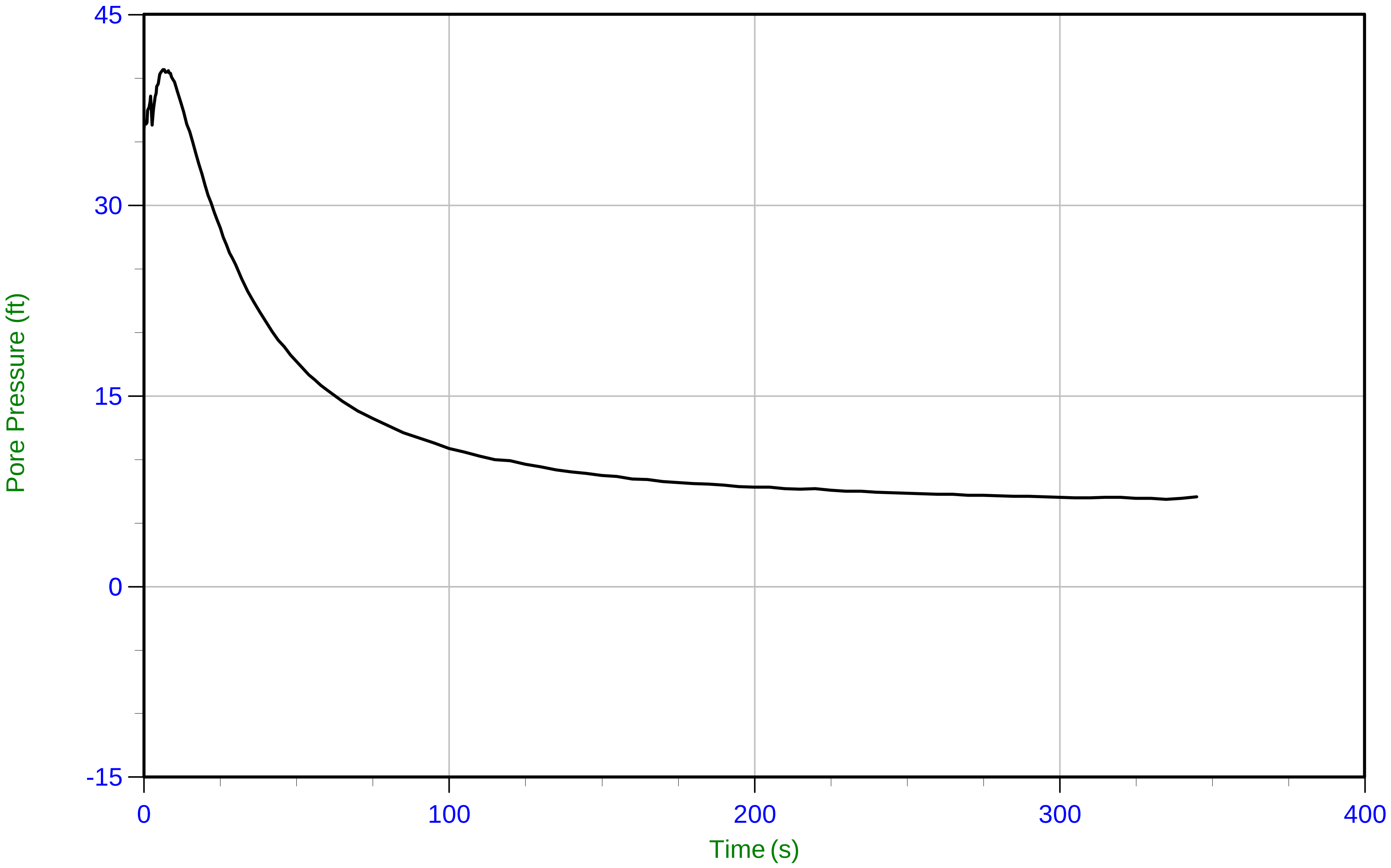
WT: 3.071 m / 10.075 ft
Ueq: 31.6 ft

CONETEC

Gannett Fleming

Job No: 24-59-27209
Date: 2024-02-22 12:58
Site: NuStar Portland

Sounding: SCPT-3
Cone: 859:T1500F15U35 Area=15 cm²

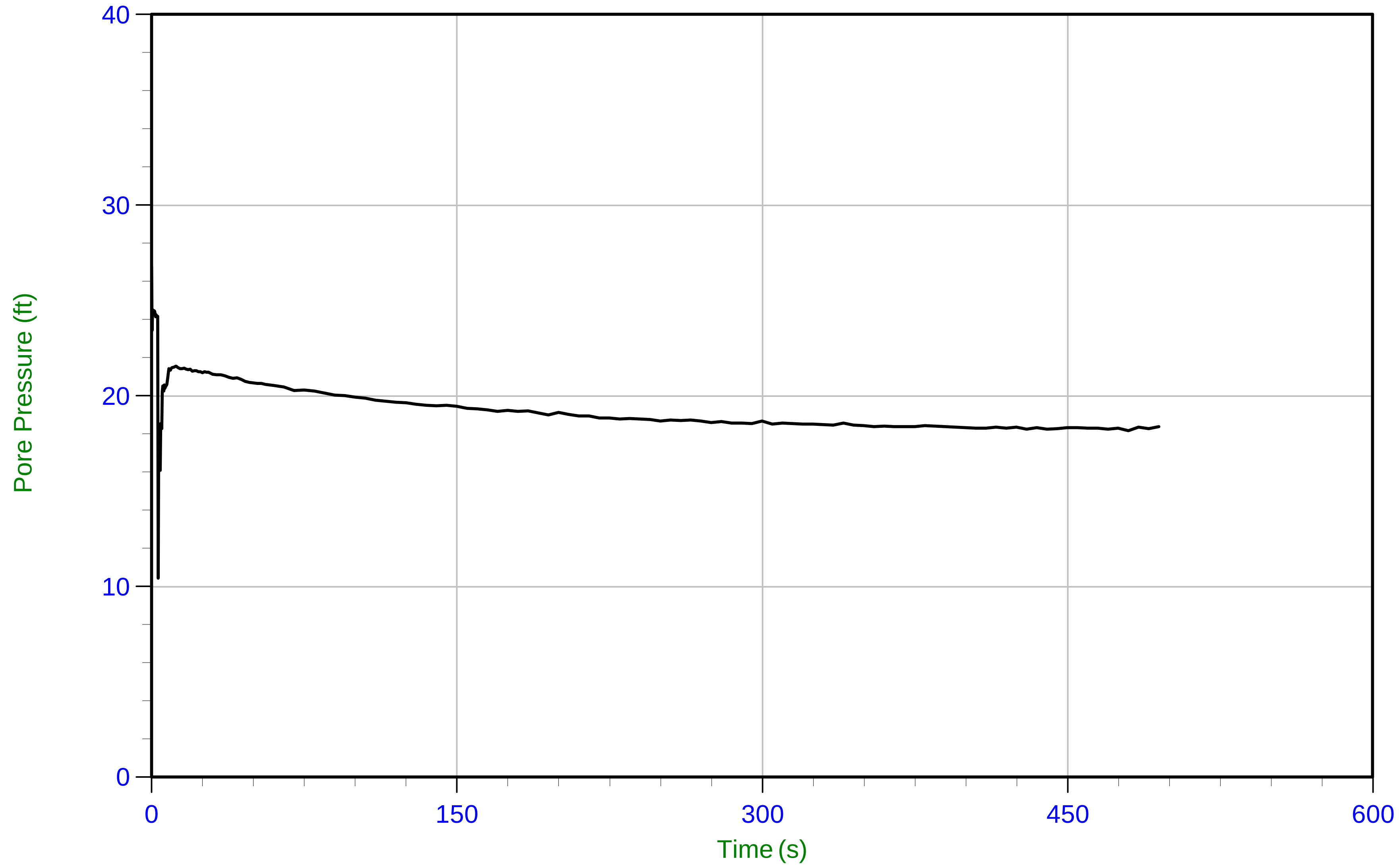


Trace Summary:

Filename: 24-59-27209_SP03.ppd2
Depth: 3.250 m / 10.663 ft
Duration: 345.0 s

u Min: 6.9 ft
u Max: 40.7 ft
u Final: 7.0 ft

WT: 1.160 m / 3.806 ft
Ueq: 6.9 ft



Trace Summary:

Filename: 24-59-27209_SP03.ppd2
Depth: 6.300 m / 20.669 ft
Duration: 495.0 s

u Min: 10.4 ft
u Max: 26.8 ft
u Final: 18.4 ft

WT: 0.738 m / 2.421 ft
Ueq: 18.2 ft

Seismic Cone Penetration Test (SCPTu) Tabular Results

Job No: 24-59-27209
Client: Gannett Fleming
Project: NuStar Portland
Sounding ID: SCPT-1
Date: 2024-02-22

Seismic Source: Beam
Seismic Offset (ft): 8.86
Source Depth (ft): 0.00
Geophone Offset (ft): 0.66

SCPTu SHEAR WAVE VELOCITY TEST RESULTS - Vs

Tip Depth (ft)	Geophone Depth (ft)	Ray Path (ft)	Ray Path Difference (ft)	Travel Time Interval (ms)	Interval Velocity (ft/s)
3.22	2.56	9.22			
6.56	5.91	10.65	1.43	6.41	222
9.94	9.28	12.83	2.19	4.55	480
13.22	12.57	15.38	2.54	3.34	761
16.40	15.75	18.07	2.69	5.28	510
19.69	19.03	20.99	2.92	5.32	549
26.18	25.53	27.02	6.03	11.73	514
29.53	28.87	30.20	3.18	7.60	418
32.74	32.09	33.29	3.09	7.01	440
36.02	35.37	36.46	3.17	6.36	499
39.37	38.71	39.72	3.26	5.58	584
42.65	42.00	42.92	3.20	5.53	579
45.93	45.28	46.14	3.22	5.44	591
49.21	48.56	49.36	3.22	2.62	1231

Job No: 24-59-27209
Client: Gannett Fleming
Project: NuStar Portland
Sounding ID: SCPT-2
Date: 2024-02-22

Seismic Source: Beam
Seismic Offset (ft): 8.86
Source Depth (ft): 0.00
Geophone Offset (ft): 0.66

SCPTu SHEAR WAVE VELOCITY TEST RESULTS - Vs

Tip Depth (ft)	Geophone Depth (ft)	Ray Path (ft)	Ray Path Difference (ft)	Travel Time Interval (ms)	Interval Velocity (ft/s)
3.22	2.56	9.22			
6.56	5.91	10.65	1.43	6.81	209
9.84	9.19	12.76	2.12	5.14	412
13.06	12.40	15.24	2.48	6.18	401
16.34	15.68	18.01	2.77	4.72	587
19.69	19.03	20.99	2.98	6.02	495
23.03	22.38	24.07	3.07	6.57	468
26.25	25.59	27.08	3.02	6.34	476
29.53	28.87	30.20	3.12	6.95	449
32.87	32.22	33.41	3.21	7.95	404
36.09	35.43	36.52	3.11	4.26	731
39.37	38.71	39.72	3.19	5.52	579

CONETEC

Job No: 24-59-27209
Client: Gannett Fleming
Project: NuStar Portland
Sounding ID: SCPT-3
Date: 2024-02-22

Seismic Source: Beam
Seismic Offset (ft): 8.86
Source Depth (ft): 0.00
Geophone Offset (ft): 0.66

SCPTu SHEAR WAVE VELOCITY TEST RESULTS - Vs

Tip Depth (ft)	Geophone Depth (ft)	Ray Path (ft)	Ray Path Difference (ft)	Travel Time Interval (ms)	Interval Velocity (ft/s)
3.28	2.63	9.24			
6.50	5.84	10.61	1.37	1.63	843
9.94	9.28	12.83	2.22	3.56	623
13.12	12.47	15.30	2.46	6.49	379
16.57	15.91	18.21	2.92	7.13	409
19.75	19.09	21.05	2.84	6.88	412
20.67	20.01	21.89	0.84	0.96	875

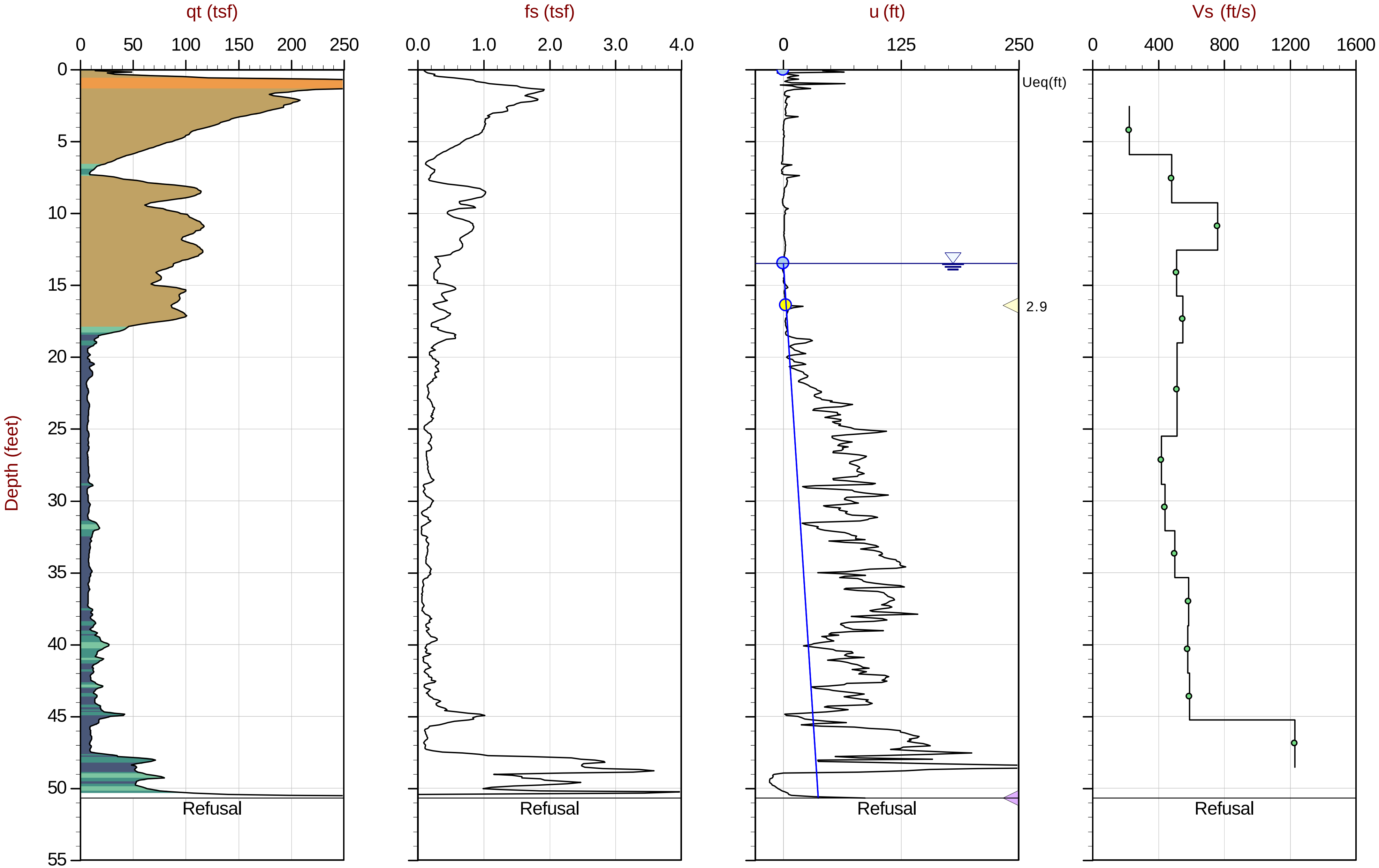
SCPTu Test Plots

CONETEC

Gannett Fleming

Job No: 24-59-27209
Date: 2024-02-22 09:30
Site: NuStar Portland

Sounding: SCPT-1
Cone: 859:T1500F15U35



Max Depth: 15.450 m / 50.69 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 24-59-27209_SP01.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: Lat: 45.59080 Long: -122.77537

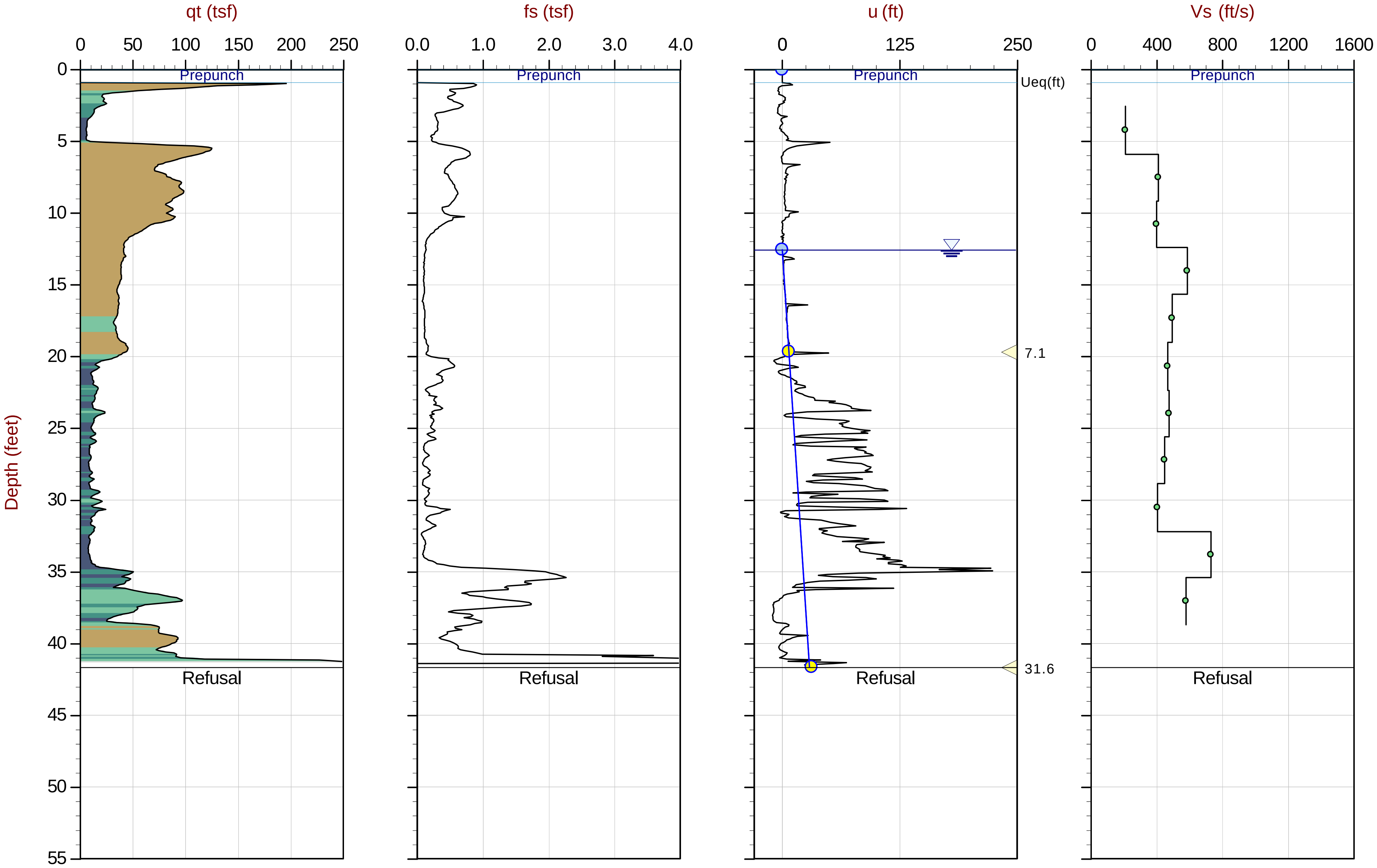
● Equilibrium Pore Pressure (Ueq) ● Assumed Ueq ◁ Dissipation, Ueq achieved ◁ Dissipation, Ueq not achieved — Hydrostatic Line

The reported coordinates were acquired from consumer-grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

Gannett Fleming

Job No: 24-59-27209
Date: 2024-02-22 11:13
Site: NuStar Portland

Sounding: SCPT-2
Cone: 859:T1500F15U35



Max Depth: 12.700 m / 41.67 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 24-59-27209_SP02.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: Lat: 45.59087 Long: -122.77652

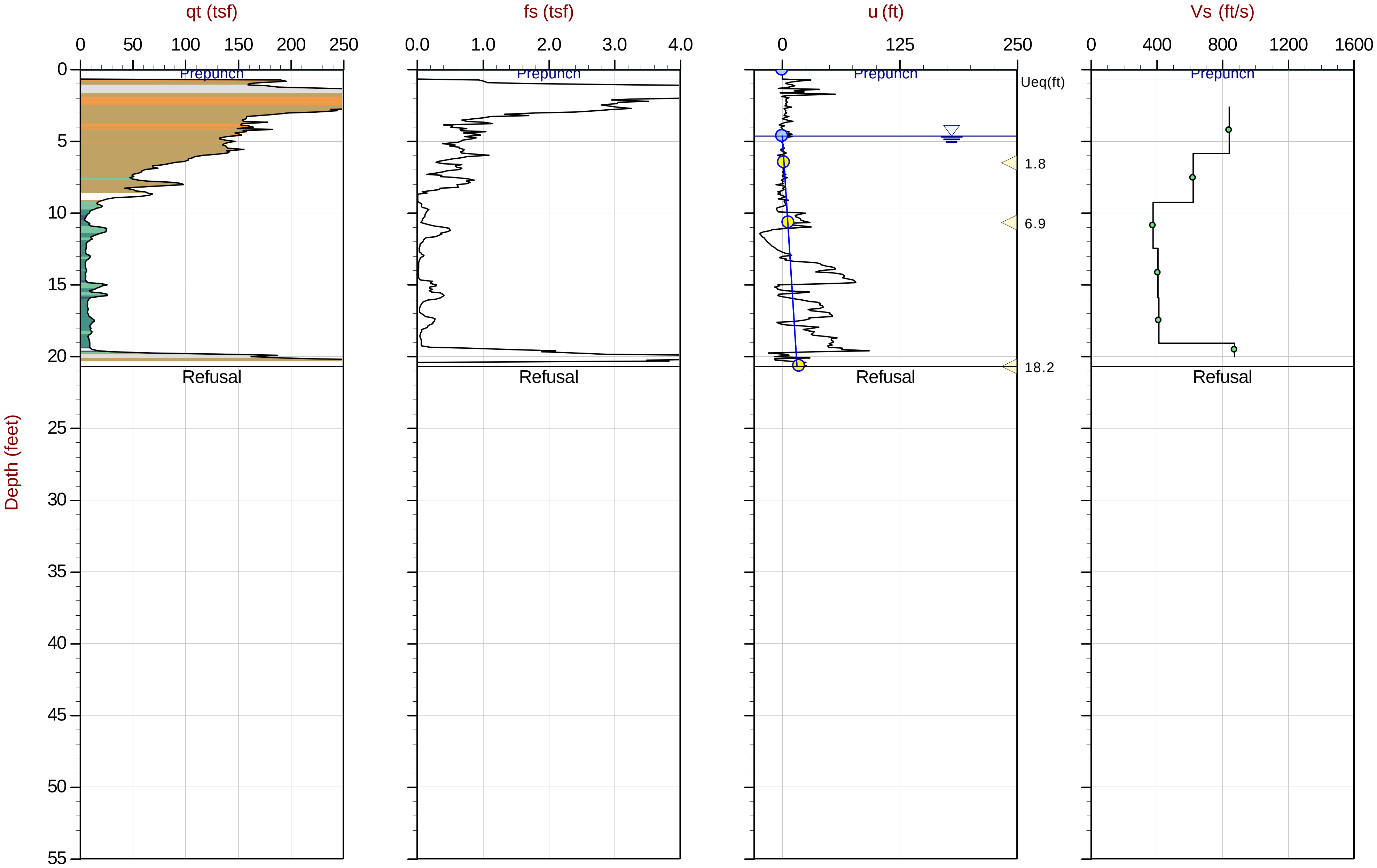
● Equilibrium Pore Pressure (Ueq) ● Assumed Ueq ◁ Dissipation, Ueq achieved ◁ Dissipation, Ueq not achieved — Hydrostatic Line

The reported coordinates were acquired from consumer-grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

Gannett Fleming

Job No: 24-59-27209
Date: 2024-02-22 12:58
Site: NuStar Portland

Sounding: SCPT-3
Cone: 859:T1500F15U35



Max Depth: 6.300 m / 20.67 ft
Depth Inc: 0.025 m / 0.082 ft
Avg Int: Every Point

File: 24-59-27209_SP03.COR
Unit Wt: SBTQtn (PKR2009)

SBT: Robertson, 2009 and 2010
Coords: Lat: 45.59067 Long: -122.77744

● Equilibrium Pore Pressure (Ueq) ● Assumed Ueq ◁ Dissipation, Ueq achieved ◁ Dissipation, Ueq not achieved — Hydrostatic Line

The reported coordinates were acquired from consumer-grade GPS equipment and are only approximate locations. The coordinates should not be used for design purposes.

SCPTu Velocity Wave Traces

Job No: 24-59-27209
Date: February 22, 2024

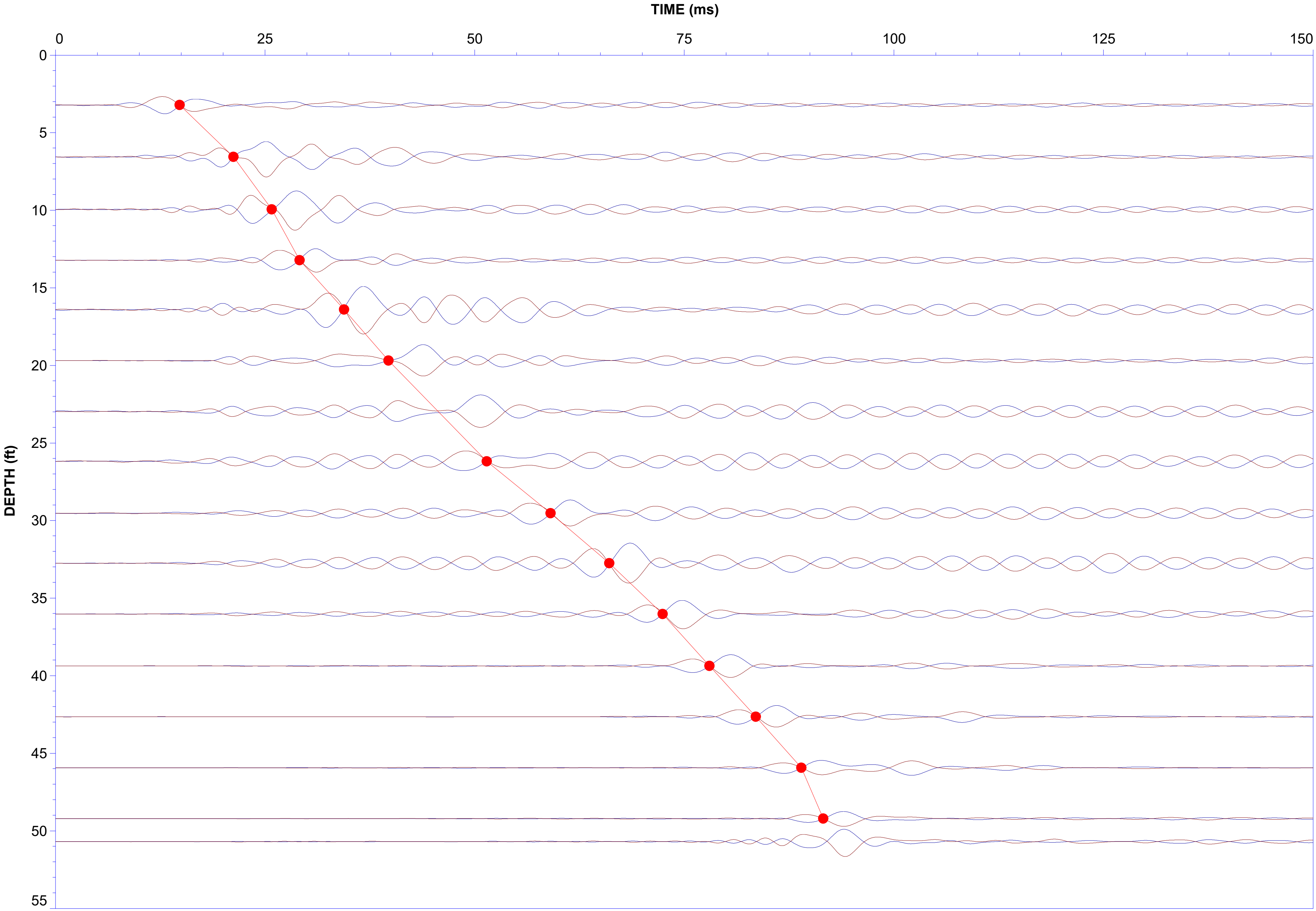
Client: Gannett Fleming
Cone: 859:T1500F15U35

Project: NuStar Portland

Analysis: Shear Wave

Sounding: SCPT-1

Filter: BP 10 - 300 Hz



Job No: 24-59-27209
Date: February 22, 2024

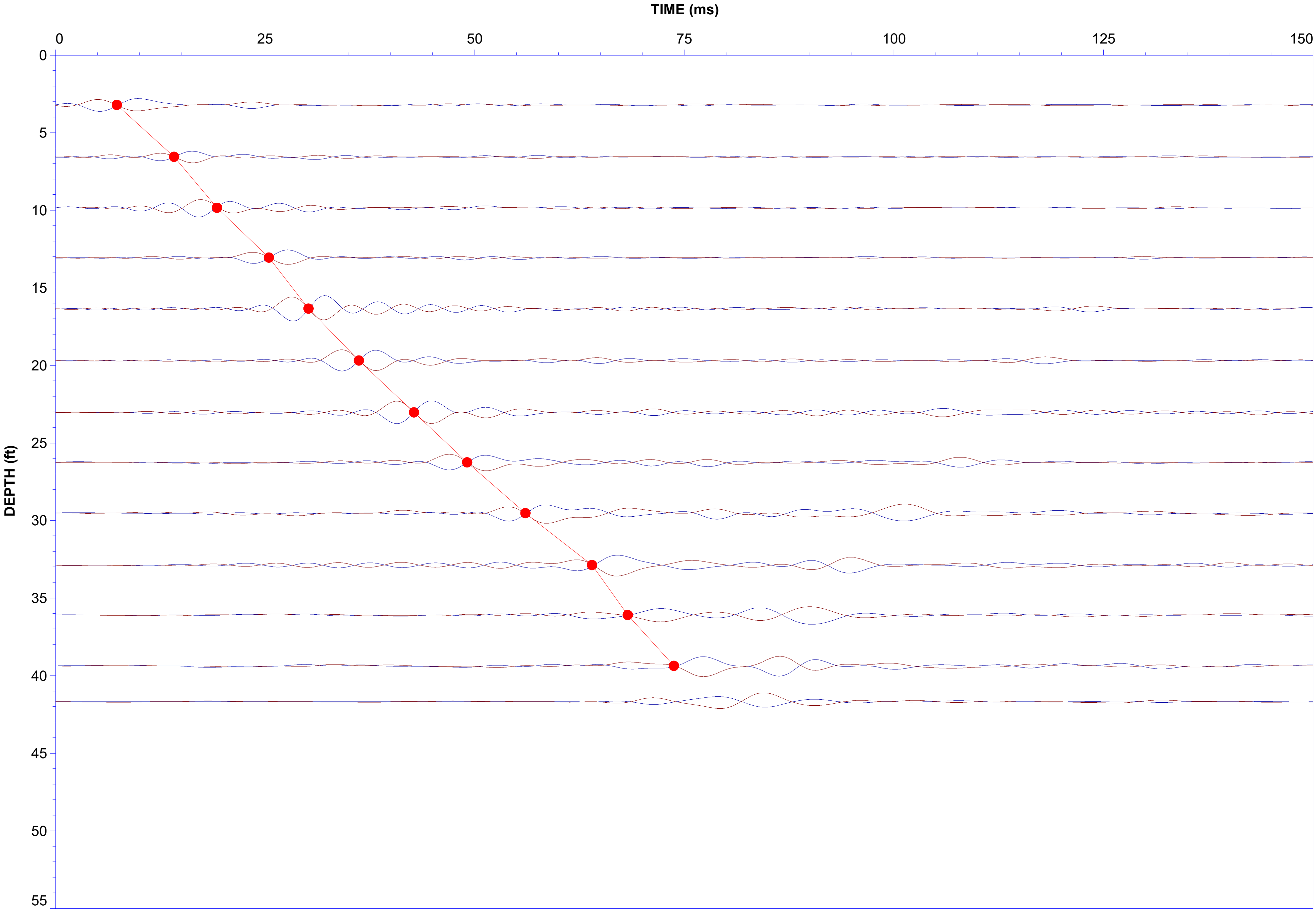
Client: Gannett Fleming
Cone: 859:T1500F15U35

Project: NuStar Portland

Analysis: Shear Wave

Sounding: SCPT-2

Filter: BP 10 - 175 Hz



Job No: 24-59-27209
Date: February 22, 2024

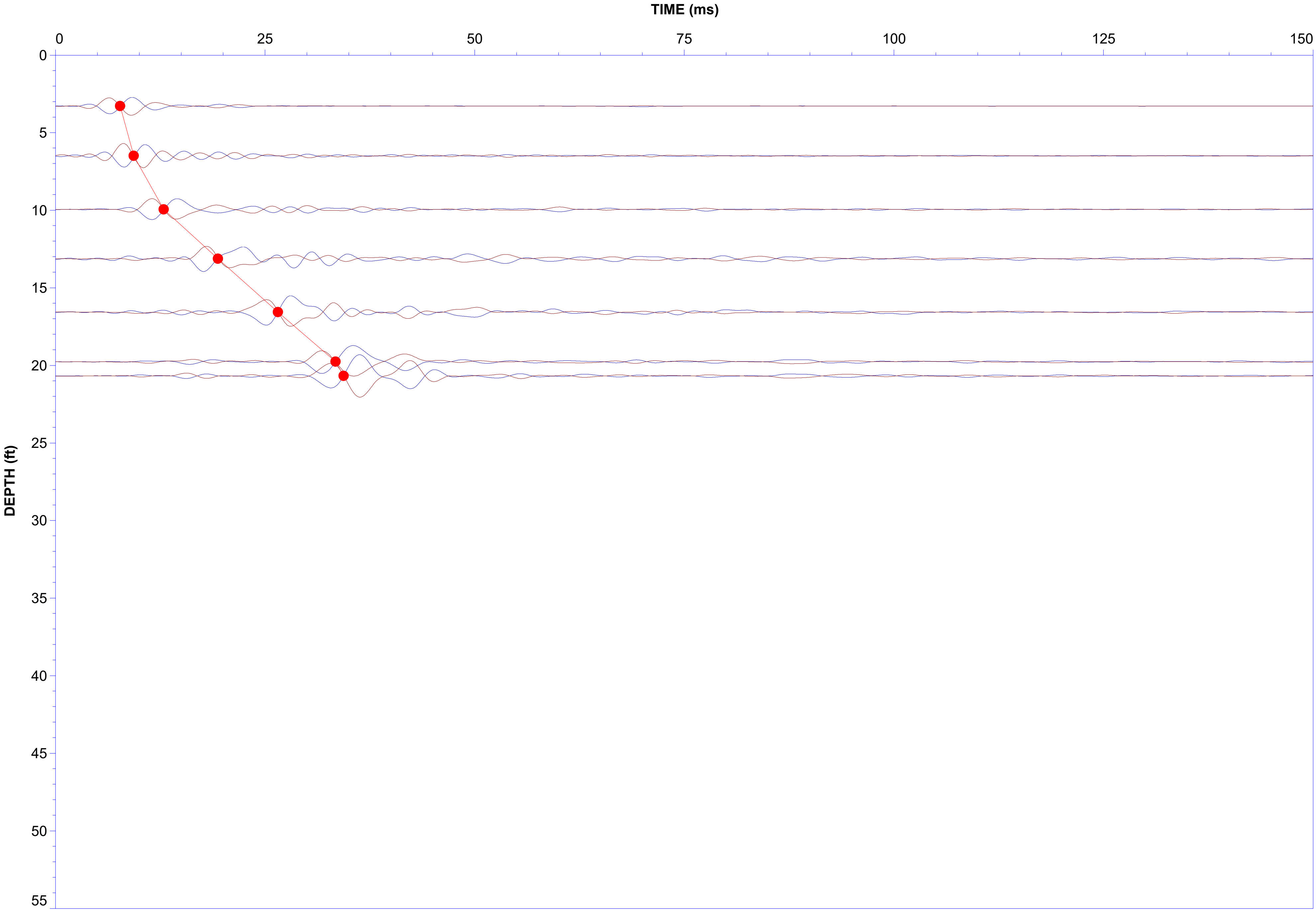
Client: Gannett Fleming
Cone: 859:T1500F15U35

Project: NuStar Portland

Analysis: Shear Wave

Sounding: SCPT-3

Filter: BP 10 - 300 Hz



Description of Methods for Calculated CPT Geotechnical Parameters

CALCULATED CPT GEOTECHNICAL PARAMETERS

A Detailed Description of the Methods Used in ConeTec's CPT Geotechnical Parameter Calculation and Plotting Software

Revision SZW-Rev 18

Revised February 10, 2023
Prepared by Jim Greig, M.A.Sc, P.Eng (BC, AB, ON)

Limitations

The geotechnical parameter output was prepared specifically for the site and project named in the accompanying report subject to objectives, site conditions and criteria provided to ConeTec by the client. The output may not be relied upon by any other party or for any other site without the express written permission of ConeTec Group (ConeTec) or any of its affiliates. For this project, ConeTec has provided site investigation services, prepared factual data reporting and produced geotechnical parameter calculations consistent with current best practices. No other warranty, expressed or implied, is made.

To understand the calculations that have been performed and to be able to reproduce the calculated parameters the user is directed to the basic descriptions for the methods in this document and the detailed descriptions and their associated limitations and appropriateness in the technical references cited for each parameter.

ConeTec's Calculated CPT Geotechnical Parameters as of February 10, 2023.

ConeTec's CPT parameter calculation and plotting routine provides a tabular output of geotechnical parameters based on current published CPT correlations and is subject to change to reflect the current state of practice. Due to drainage conditions and the basic assumptions and limitations of the correlations, not all geotechnical parameters provided are considered applicable for all soil types. The results are presented only as a guide for geotechnical use and should be carefully examined for consideration in any geotechnical design. Reference to current literature is strongly recommended. ConeTec does not warranty the correctness or the applicability of any of the geotechnical parameters calculated by the program and does not assume liability for any use of the results in any design or review. For verification purposes we recommend that representative hand calculations be done for any parameter that is critical for design purposes. The end user of the parameter output should also be fully aware of the techniques and the limitations of any method used by the program. The purpose of this document is to inform the user as to which methods were used and to direct the end user to the appropriate technical papers and/or publications for further reference.

The geotechnical parameter output was prepared specifically for the site and project named in the accompanying report subject to objectives, site conditions and criteria provided to ConeTec by the client. The output may not be relied upon by any other party or for any other site without the express written permission of ConeTec Group (ConeTec) or any of its affiliates.

The CPT calculations are based on values of tip resistance, sleeve friction and pore pressures considered at each data point or averaged over a user specified layer thickness (e.g., 0.20 m). Note that q_t is the tip resistance corrected for pore pressure effects and q_c is the recorded tip resistance. The corrected tip resistance (corrected using u_2 pore pressure values) is used for all calculations. Since all ConeTec cones have equal end area friction sleeves pore pressure corrections to sleeve friction, f_s, are not performed.

Corrected tip resistance: $q_t = q_c + (1-a) \cdot u_2$ (consistent units are required)

where: q_t is the corrected tip resistance
q_c is the recorded tip resistance
u_2 is the recorded dynamic pore pressure from behind the tip (u_2 position)
a is the Net Area Ratio for the cone (typically 0.80 for ConeTec cones)

The total stress calculations are based on soil unit weight values that have been assigned to the Soil Behavior Type (SBT) zones, from a user defined unit weight profile, by using a single uniform value throughout the profile, through unit weight estimation techniques described in various technical papers or from a combination of these methods. The parameter output files indicate the method(s) used.

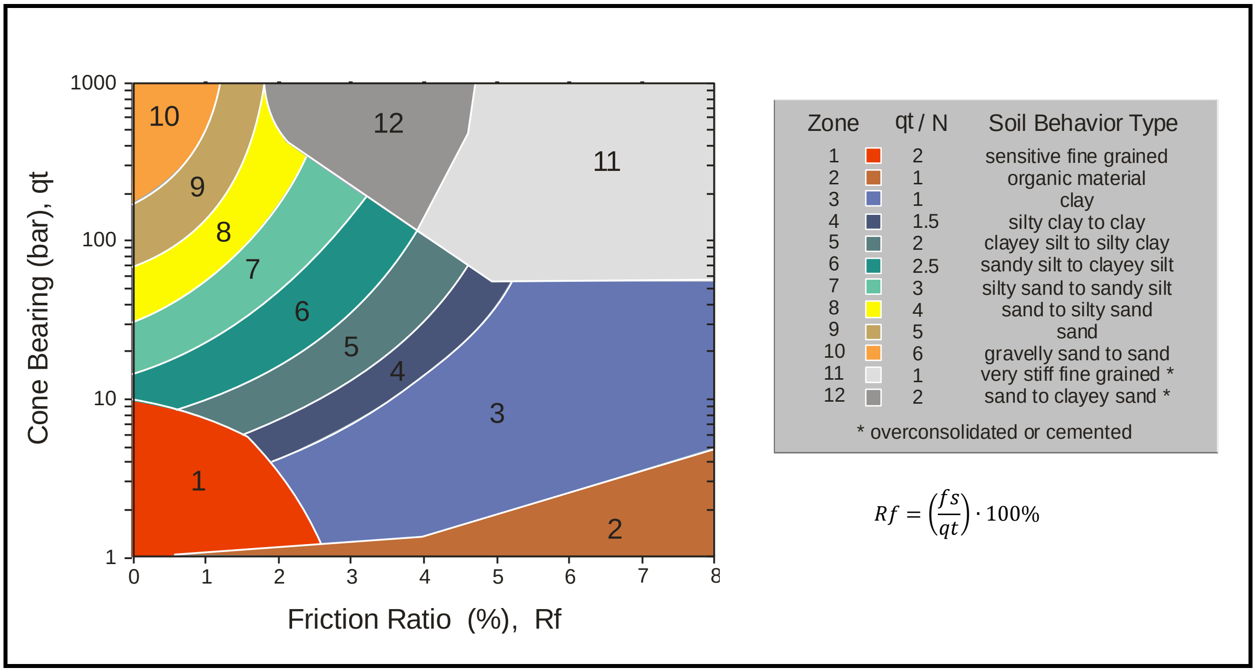
Effective vertical overburden stresses are calculated using the total stress and equilibrium pore pressure (u_{eq} or u_o) values derived from an assumed hydrostatic distribution of pore pressures below the water table or from a user defined equilibrium pore pressure profile (typically obtained from CPT dissipation tests) or a combination of the two. For over water projects the stress effects of the column of water above the mudline are taken into account as is the appropriate unit weight of water. How this is done depends on where the instruments are zeroed (i.e. on deck or at the mudline). The parameter output files indicate the method(s) used.

A majority of parameter calculations are derived from or driven by results based on material types as determined by the various soil behavior type charts depicted in Figures 1 through 6. The parameter output files indicate the method(s) used.

The Soil Behavior Type classification chart shown in Figure 1 is the classic non-normalized SBT Chart developed at the University of British Columbia and reported in Robertson, Campanella, Gillespie and Greig (1986). Figure 2 shows the original normalized (linear method) SBTn chart developed by Robertson (1990). The Bq classification charts shown in Figures 3a and 3b incorporate pore pressures into the SBT classification and are based on the methods described in Robertson (1990). Many of these charts have been summarized in Lunne, Robertson and Powell (1997). The Jefferies and Davies SBT chart shown in Figure 3c is based on the techniques discussed in Jefferies and Davies (1993) which introduced the concept of the Soil Behavior Type Index parameter, I_c. Take note that the I_c parameter developed by Robertson and Fear (1995) and Robertson and Wride (1998) is similar in concept but uses a slightly different calculation method than that defined by Jefferies and Davies (1993) as the latter incorporates pore pressure in their technique through the use of the Bq parameter. The normalized Qtn SBT chart shown in Figure 4 is based on the work by Robertson (2009) utilizing a variable stress ratio exponent, n, for normalization based on a slightly modified redefinition and iterative approach for I_c. The boundary curves drawn on the chart are based on the work described in Robertson (2010).

Figure 5 shows a revised 1986 SBT Chart presented to CPT'10 by Robertson (2010b). It is known as the Updated non-normalized Soil Behavior Chart (also referred to as the Rev SBT Chart (PKR2010) in our output files). This chart was produced to be more in line with all post-1986 Robertson charts having the same 9 soil type zones, a $\log_{10}$ axis for friction ratio, R_f in this case, and a unitless tip resistance axis.

Figure 6 shows a revised behavior based chart by Robertson (2016) depicting contractive-dilative zones. As the zones represent material behavior rather than soil gradation ConeTec has chosen a set of zone colors that are less likely to be confused with material type colors from previous SBT charts. These colors differ from those used by Dr. Robertson. A green palette was selected for the dilative (desirable) side of the chart and a red palette for the contractive side of the chart.



$$Rf = \left(\frac{fs}{qt}\right) \cdot 100\%$$

Figure 1. Non-normalized Soil Behavior Type Classification Chart (SBT)

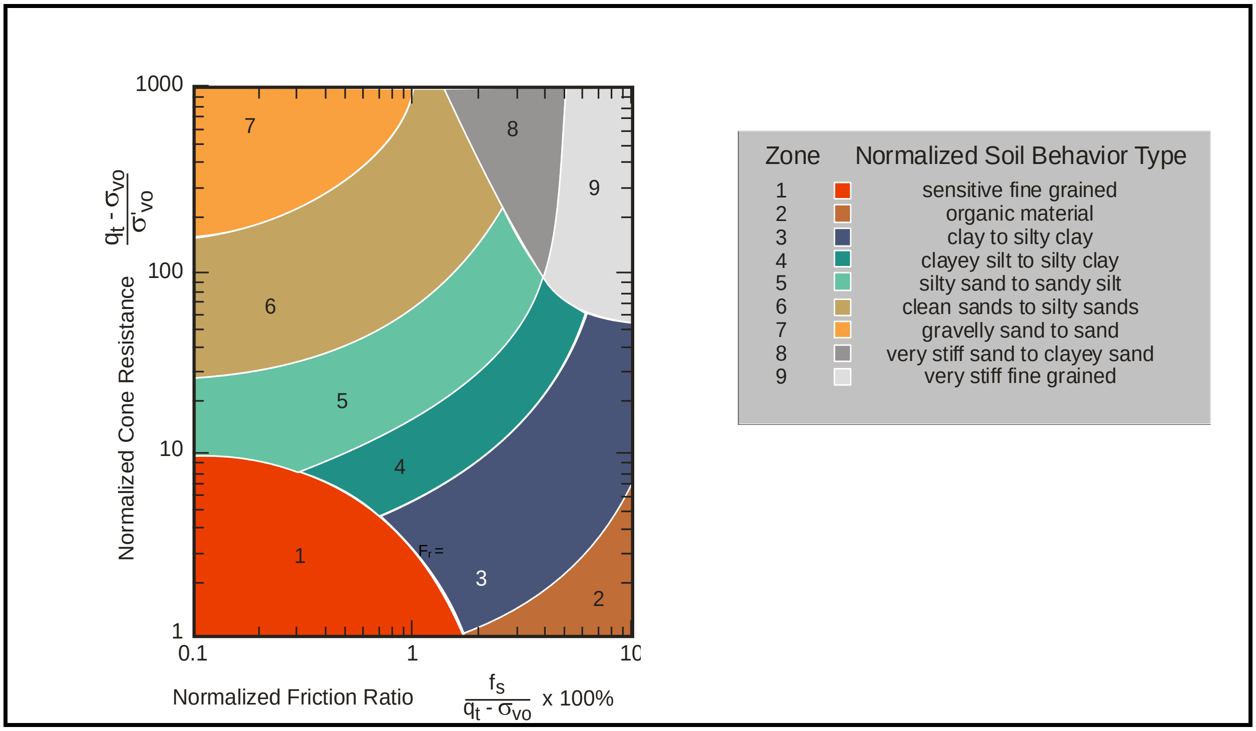


Figure 2. Normalized Soil Behavior Type Classification Chart (SBTn)

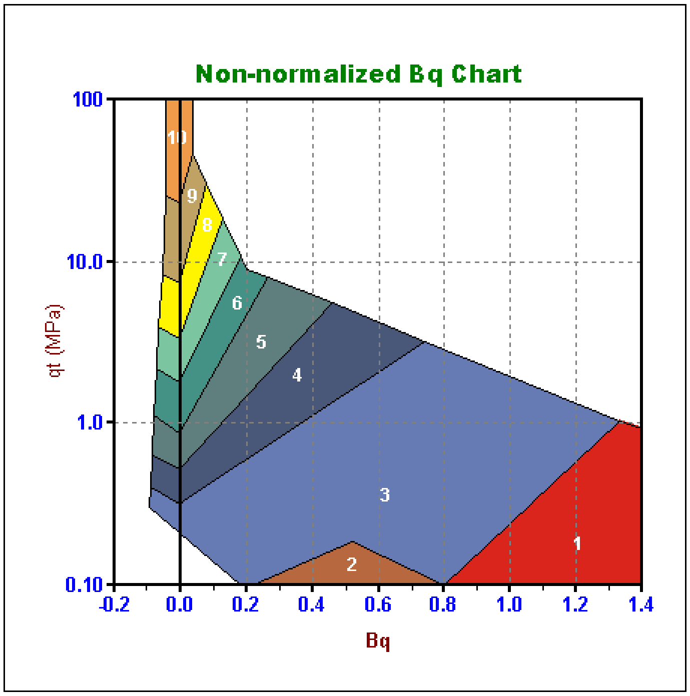


Figure 3a. Alternate Soil Behavior Type Chart (SBT Bq): q_t - B_q

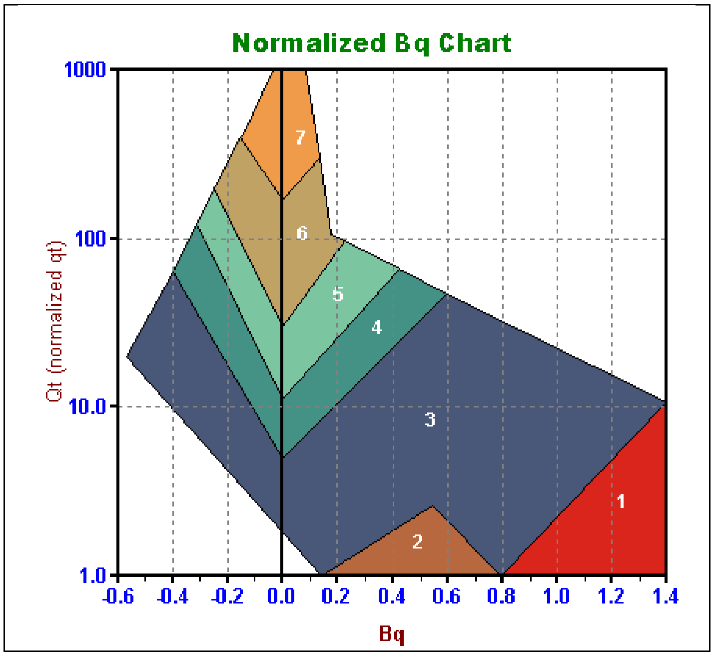


Figure 3b. Alternate Soil Behavior Type Charts (SBT Bqn): Q_t-B_q

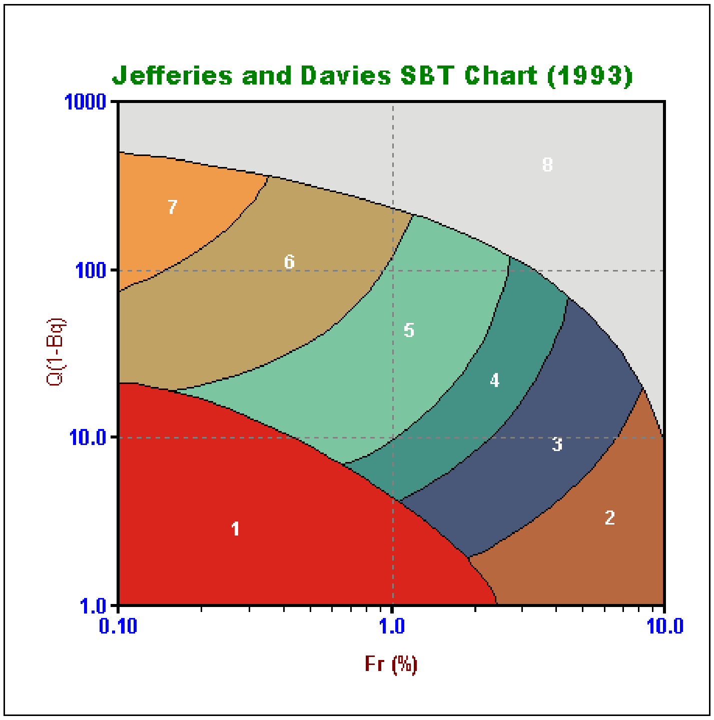


Figure 3c. Alternate Soil Behavior Type Charts: $Q(1\text{-}B_q)$ - F_r

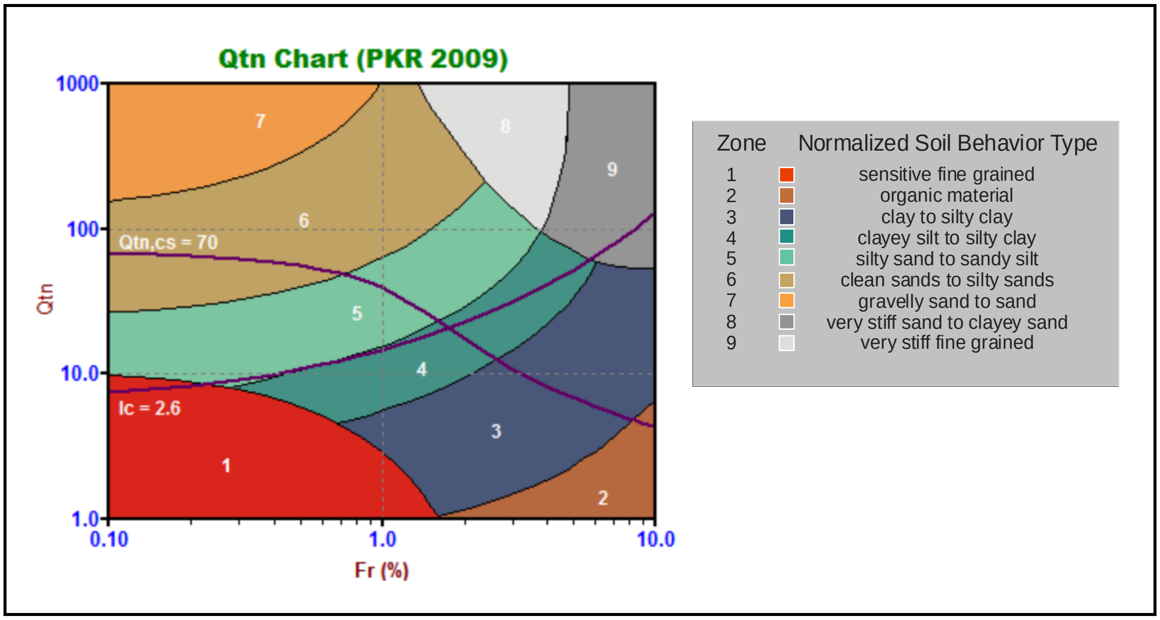


Figure 4. Normalized Soil Behavior Type Chart using Q_{tn} (SBT Qtn)

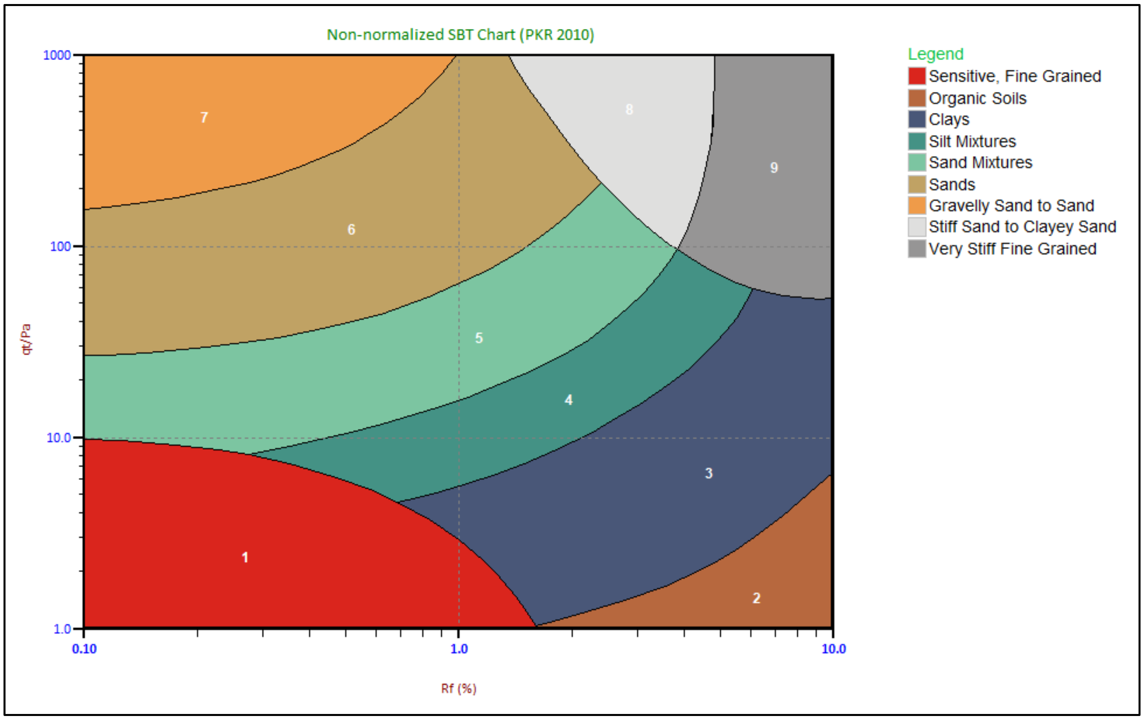


Figure 5. Non-normalized Soil Behavior Type Chart (2010)

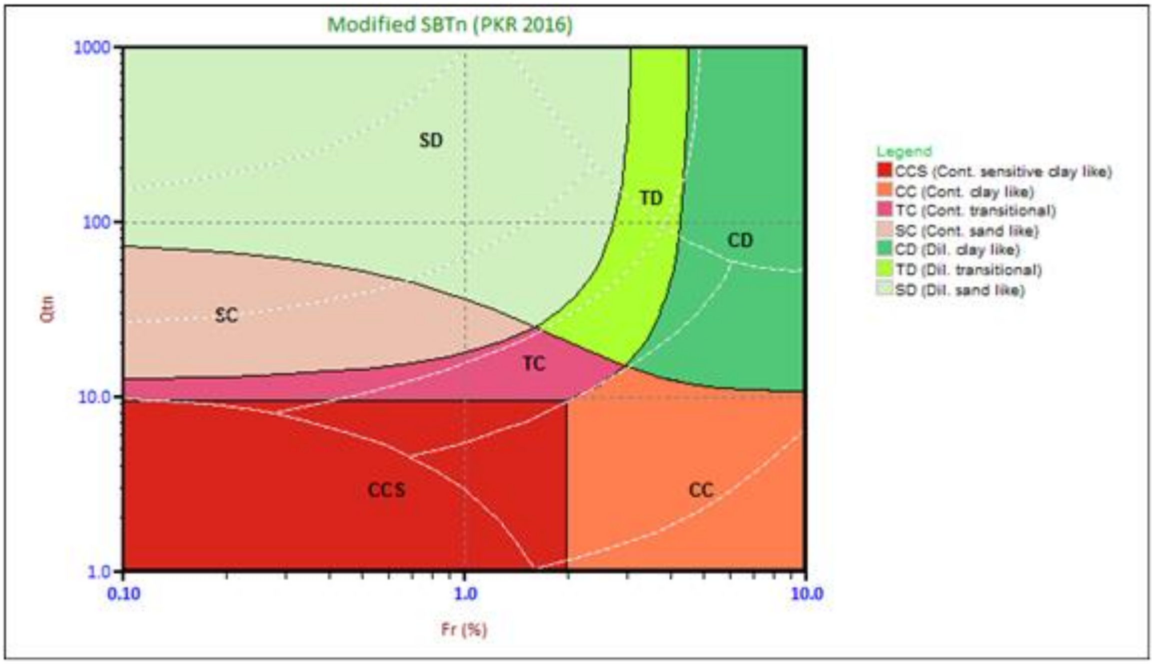


Figure 6. Modified SBTn Behavior Based Chart

Details regarding the geotechnical parameter calculations are provided in Tables 1a and 1b. The appropriate references cited are listed in Table 2. Non-liquefaction specific parameters are detailed in Table 1a and liquefaction specific parameters are detailed in Table 1b.

Where methods are based on charts or techniques that are too complex to describe in this summary, we recommend that the user refer to the cited material. Specific limitations for each method are described in the cited material.

Where the results of a calculation/correlation are deemed *'invalid'* the value will be represented by the text strings "-9999", "-9999.0", the value 0.0 (Zero) or an empty cell. Invalid results will occur because of (and not limited to) one or a combination of:

1. Invalid or undefined CPT data (e.g., drilled out section or data gap).

2. Where the calculation method is inappropriate, for example, drained parameters in a material behaving in an undrained manner (and vice versa).

3. Where input values are beyond the range of the referenced charts or specified limitations of the correlation method.

4. Where pre-requisite or intermediate parameter calculations are invalid.

The parameters selected for output from the program are often specific to a particular project. As such, not all of the calculated parameters listed in Tables 1and 1a may be included in the output files delivered with this report.

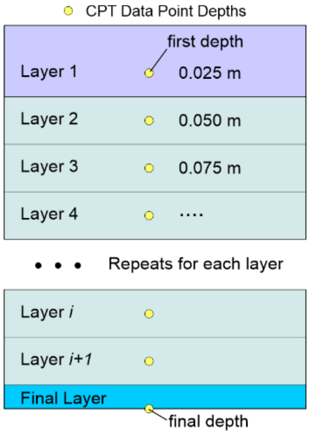
The output files are typically provided in Microsoft Excel XLS, XLSX or CSV format. The ConeTec software has several options for output depending on the number or types of calculated parameters desired or those specifically contracted for by the client. Each output file is named using the original file base name (from the .COR file) followed by a three or four character indicator of the output set selected (e.g. BSC, TBL, NLI, NL2, IFI, IFI2, IFI3) and possibly followed by an operator selected suffix identifying the characteristics of the particular calculation run.

Table 1a. CPT Parameter Calculation Methods – Non liquefaction Parameters

Reference Notes: CK* - Common Knowledge, U* - Unpublished

Calculated Parameter	Description	Equation	Ref
Depth	Mid Layer Depth *(where calculations are done at each point then Mid Layer Depth = Recorded Depth)*	*[Depth (Layer Top) + Depth (Layer Bottom)]/ 2.0*	CK*
Elevation	Elevation of Mid Layer is based on the sounding collar elevation supplied by the client or through a site survey In Sweden a variation of elevation is used where the elevation increases with depth. We refer to this as inverse elevation.	Elevation = Collar Elevation – Depth InverseElevation = Collar Elevation + Depth	CK* N/A
Avg qc	Averaged recorded tip value (q_c)	$Avgqc = \frac{1}{n}\sum_{i=1}^{n} q_c$ *n=1 when calculations are done at each point*	CK*
Avg qt	Averaged corrected tip (q_t) where: $q_t = q_c + (1 - a) \cdot u_2$ Averaged q_t is not calculated using the average q_c and averaged u values. Averaged q_t is based on the average of the q_t values calculated at each data point.	$Avgqt = \frac{1}{n}\sum_{i=1}^{n} q_t$ *n=1 when calculations are done at each point*	1
Avg fs	Averaged sleeve friction (f_s) No pore pressure corrections are applied to f_s.	$Avgfs = \frac{1}{n}\sum_{i=1}^{n} fs$ *n=1 when calculations are done at each point*	CK*
Avg Rf	Averaged friction ratio (R_f) where friction ratio is defined as: $R_f = 100\% \cdot \frac{fs}{q_t}$	$AvgRf = 100\% \cdot \frac{Avgfs}{Avgqt}$ *not an average of individual R_f values*	CK*
Avg u	Averaged dynamic pore pressure *(u)*	$Avgu = \frac{1}{n}\sum_{i=1}^{n} u_i$ *n=1 when calculations are done at each point*	CK*
Avg Res	Averaged Resistivity (this data is not always available since it is a specialized test requiring an additional module)	$AvgRes = \frac{1}{n}\sum_{i=1}^{n} Resistivity_i$ *n=1 when calculations are done at each point*	CK*
Avg UVIF	Averaged UVIF ultra-violet induced fluorescence (this data is not always available since it is a specialized test requiring an additional module)	$AvgUVIF = \frac{1}{n}\sum_{i=1}^{n} UVIF_i$ *n=1 when calculations are done at each point*	CK*
Avg Temp	Averaged Temperature (this data is not always available)	$AvgTemp = \frac{1}{n}\sum_{i=1}^{n} Temperature_i$ *n=1 when calculations are done at each point*	CK*
Avg Gamma	Averaged Gamma Counts (this data is not always available since it is a specialized test requiring an additional module)	$AvgGamma = \frac{1}{n}\sum_{i=1}^{n} Gamma_i$ *n=1 when calculations are done at each point*	CK*
SBT	Soil Behavior Type as defined by Robertson et al 1986 (often referred to as Robertson and Campanella, 1986)	See Figure 1	1, 5
SBTn	Normalized Soil Behavior Type as defined by Robertson 1990 (linear normalization using Q_t, now referred to as Q_{t1})	See Figure 2	2, 5

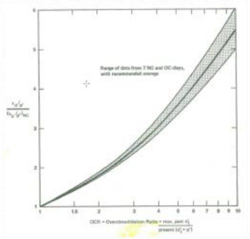
Calculated Parameter	Description	Equation	Ref
SBT-Bq	Non-normalized Soil Behavior type based on non-normalized tip resistance and the B_q parameter	See Figure 3a	1, 2, 5
SBT-Bqn	Normalized Soil Behavior type based on normalized tip resistance (Q_t, now called Q_{t1}) and the B_q parameter	See Figure 3b	2, 5
SBT-JandD	Soil Behavior Type as defined by Jeffries and Davies	See Figure 3c	7
SBT Qtn	Soil Behavior Type as defined by Robertson (2009) using a variable stress ratio exponent for normalization based on $I_{c\ (PKR\ 2009)}$	See Figure 4	15
Modified Non-normalized SBT Chart SBT (PKR2010)	This is a revised version of the simple 1986 non-normalized SBT chart (presented at CPT ’10). The revised version has been reduced from 12 zones to 9 zones to be similar to the normalized Robertson charts. Other updates include a dimensionless tip resistance normalized to atmospheric pressure, q_t/P_a, on the vertical axis and a log scale for non-normalized friction ratio, R_f, along the horizontal axis.	See Figure 5	33
Modified SBTn (contractive /dilative)	Modified SBTn chart as defined by Robertson (2016) indicating zones of contractive/dilative behavior. Note that ConeTec displays the chart with colors different from Robertson. ConeTec’s colors were chosen to avoid confusion with soil type descriptions.	See Figure 6	30
Unit Wt.	Unit Weight of soil determined from one of the following user selectable options: 1) uniform value 2) value assigned to each SBT zone 3) value assigned to each SBTn zone 4) value assigned to SBTn zone as determined from Robertson and Wride (1998) based on q_{c1n} 5) values assigned to SBT Qtn zones 6) values based on Robertson updated non-normalized Soil Behavior Type Chart (2010b) 6) Mayne f_s (sleeve friction) method 7) Robertson and Cabal 2010 method 8) user supplied unit weight profile The last option may co-exist with any of the other options.	See references	3, 5, 15, 21, 24, 29, 33

Calculated Parameter	Description	Equation	Ref
TStress σ_v	Total vertical overburden stress at Mid Layer Depth *A layer is defined as the averaging interval specified by the user where depths are reported at their respective mid-layer depth.* For data calculated at each point layers are defined using the recorded depth as the mid-point of the layer. Thus, a layer starts half-way between the previous depth and the current depth unless this is the first point in which case the layer start is at zero depth. The layer bottom is half-way from the current depth to the next depth unless it is the last data point. Defining layers affects how stresses are calculated since the unit weight attributed to a data point is used throughout the entire layer. This means that to calculate the stresses the total stress at the top and bottom of a layer are required. The stress at mid layer is determined by adding the incremental stress from the layer top to the mid-layer depth. The stress at the layer bottom becomes the stress at the top of the subsequent layer. Stresses are NOT calculated from mid-point to mid-point. For over-water work the total stress due to the column of water above the mud line is taken into account where appropriate.	$TStress = \sum_{i=1}^{n} \gamma_i h_i$ where γ_i is layer unit weight h_i is layer thickness 	CK*
EStress σ_v'	Effective vertical overburden stress at mid-layer depth.	$\sigma_v' = \sigma_v - u_{eq}$	CK*
Equil u u_{eq} or u_0	Equilibrium pore pressures are determined from one of the following user selectable options: 1) hydrostatic below the water table 2) user supplied profile 3) combination of those above When a user supplied profile is used/provided a linear interpolation is performed between equilibrium pore pressures defined at specific depths. If the profile values start below the water table then a linear transition from zero pressure at the water table to the first defined pointed is used. Equilibrium pore pressures may come from dissipation tests, adjacent piezometers or other sources. Occasionally, an extra equilibrium point ("assumed value") will be provided in the profile that does not come from a recorded value to smooth out any abrupt changes or to deal with material interfaces. These "assumed" values will be indicated on our plots and in tabular summaries.	For the hydrostatic option: $u_{eq} = \gamma_w \cdot (D - D_{wt})$ where u_{eq} is equilibrium pore pressure γ_w is the unit weight of water D is the current depth D_{wt} is the depth to the water table	CK*
K_0	Coefficient of earth pressure at rest, K_0.	$K_0 = (1 - \sin\Phi')\ OCR^{\sin\Phi'}$	17
C_n	Overburden stress correction factor used for $(N_1)_{60}$ and older CPT parameters.	$C_n = (P_a/\sigma_v')^{0.5}$ where $0.0 < C_n < 2.0$ *(user adjustable, typically ranging from 1.7 to 2.0)* P_a *is atmospheric pressure (100 kPa)*	4, 12

Calculated Parameter	Description	Equation	Ref
C_q	Overburden stress normalizing factor.	$C_q = 1.8 / [0.8 + (\sigma_v'/P_a)]$ where $0.0 < C_q < 2.0$ *(user adjustable)* P_a *is atmospheric pressure (100 kPa)* *Robertson and Wride define* C_q *to be the same as* C_n*. The Olson definition above is used in the program.*	3, 12
N_{60}	SPT N value at 60% energy calculated from q_t/N ratios assigned to each SBT zone. This method has abrupt N value changes at zone boundaries.	*See Figure 1*	5
$(N_1)_{60}$	SPT N_{60} value corrected for overburden pressure.	$(N_1)_{60} = C_n \bullet N_{60}$	4
$N_{60}I_c$	SPT N_{60} values based on the I_c parameter, as defined by Robertson and Wride 1998 (3), or by Robertson 2009 (15).	$(q_t/P_a)/ N_{60} = 8.5 (1 – I_c/4.6)$ $(q_t/P_a)/ N_{60} = 10^{(1.1268 – 0.2817Ic)}$ P_a being atmospheric pressure	3, 5 15, 31
$(N_1)_{60}I_c$	SPT N_{60} value corrected for overburden pressure (using N_{60} I_c). User has 3 options.	1) $(N_1)_{60}Ic = C_n \bullet (N_{60} I_c)$ 2) $q_{c1n} / (N_1)_{60}I_c = 8.5 (1 – I_c/4.6)$ 3) $(Q_{tn}) / (N_1)_{60}I_c = 10^{(1.1268 – 0.2817Ic)}$	4 5 15, 31
S_u or S_u (N_{kt})	Undrained shear strength based on q_t S_u factor N_{kt} is user selectable.	$Su = \frac{qt - \sigma_v}{N_{kt}}$	1, 5
S_u or S_u (N_{du}) or S_u ($N_{\Delta u}$)	Undrained shear strength based on pore pressure S_u factor $N_{\Delta u}$ is user selectable.	$Su = \frac{u_2 - u_{eq}}{N_{\Delta u}}$	1, 5
D_r	Relative Density determined from one of the following user selectable options: 1) Ticino Sand 2) Hokksund Sand 3) Schmertmann (1978) 4) Jamiolkowski (1985) - All Sands 5) Jamiolkowski et al (2003) (various compressibilities, K_o)	See reference (methods 1 through 4) Jamiolkowski et al (2003) reference	5 14
PHI ϕ	Friction Angle determined from one of the following user selectable options (methods 1 through 4 are for sands and method 5 is for silts and clays): 1) Campanella and Robertson 2) Durgunoglu and Mitchel 3) Janbu 4) Kulhawy and Mayne 5) NTH method (clays and silts)	See appropriate reference	5 5 5 11 23
Delta U/q_t $\Delta u/q_t$ du/q_t	Differential pore pressure ratio (older parameter used before B_q was established)	$= \frac{\Delta u}{qt}$ *where:* $\Delta u = u - u_{eq}$ *and* u *= dynamic pore pressure* u_{eq} *= equilibrium pore pressure*	39

Calculated Parameter	Description	Equation	Ref
B_q	Pore pressure parameter	$Bq = \frac{\Delta u}{qt - \sigma_v}$ where: $\Delta u = u - u_{eq}$ *and* u *= dynamic pore pressure* u_{eq} *= equilibrium pore pressure*	1, 2, 5
Net q_t or qtNet	Net tip resistance (used in many subsequent correlations)	$qt - \sigma_v$	36
q_e or qE or q_E	Effective tip resistance (using the dynamic pore pressure u_2 and not equilibrium pore pressure)	$q_t - u_2$	36
qeNorm	Normalized effective tip resistance	$\frac{qt - u_2}{\sigma'_v}$	36
Q_t or Norm: Qt or Q_{t1}	Normalized q_t for Soil Behavior Type classification as defined by Robertson (1990) using a linear stress normalization. Note this is different from Q_{tn}. This parameter was renamed to Q_{t1} in Robertson, 2009. Without normalization limits this parameter calculates to very high unrealistic values at low stresses.	$Qt = \frac{qt - \sigma_v}{\sigma'_v}$	2, 5, 15
F_r or Norm: Fr	Normalized Friction Ratio for Soil Behavior Type classification as defined by Robertson (1990)	$Fr = 100\% \cdot \frac{fs}{qt - \sigma_v}$	2, 5
$Q(1\text{-}B_q)$ $Q(1\text{-}B_q) + 1$	$Q(1\text{-}B_q)$ grouping as suggested by Jefferies and Davies for their classification chart and the establishment of their I_c parameter. Later papers added the +1 term to the equation.	$Q \cdot (1 - Bq)$ $Q \cdot (1 - Bq) + 1$ *where Bq is defined as above and Q is the same as the normalized tip resistance,* Q_{t1}*, defined above*	6, 7, 34
q_{c1}	Normalized tip resistance, q_{c1}, using a fixed stress ratio exponent, n (this method has stress units)	$q_{c1} = q_t \cdot (Pa/\sigma_v')^{0.5}$ where: P_a = atmospheric pressure	21
q_{c1} (0.5)	Normalized tip resistance, q_{c1}, using a fixed stress ratio exponent, n (this method is unit-less)	$q_{c1}\ (0.5) = (q_t/P_a) \cdot (Pa/\sigma_v')^{0.5}$ where: P_a = atmospheric pressure	5
q_{c1} (C_n)	Normalized tip resistance, q_{c1}, based on C_n (this method has stress units)	$q_{c1}(Cn) = C_n * q_t$	5, 12
q_{c1} (C_q)	Normalized tip resistance, q_{c1}, based on C_q (this method has stress units)	$q_{c1}\ (Cq) = C_q * q_t$ (some papers use q_c)	5, 12
q_{c1n}	normalized tip resistance, q_{c1n}, using a variable stress ratio exponent, n (where n=0.0, 0.70, or 1.0) (this method is unit-less)	$q_{c1n} = (q_t / P_a)(P_a/\sigma_v')^n$ where: P_a = atm. Pressure and n varies as described below	3

Calculated Parameter	Description	Equation	Ref
I_c or Ic (RW1998)	Soil Behavior Type Index as defined by Robertson and Wride (1997, 1998) for estimating grain size characteristics and providing smooth gradational changes across the SBTn chart. Ic(RW1998) is different from that of Jefferies and Davies (7) and is different from Ic(PKR2009).	$I_c = [(3.47 - \log_{10}Q)^2 + (\log_{10} Fr + 1.22)^2]^{0.5}$ *Where:* $Q = \left(\frac{qt - \sigma_v}{P_a}\right)\left(\frac{P_a}{\sigma_v'}\right)^n$ *Or* $Q = q_{c1n} = \left(\frac{qt}{P_a}\right)\left(\frac{P_a}{\sigma_v'}\right)^n$ *depending on the iteration in determining* I_c *And Fr is in percent* P_a *= atmospheric pressure* *n has the following distinct values:* *0.5, 0.75 and 1.0* *and is determined in an iterative manner based on the resulting* I_c *in each iteration* *Note that NCEER replaced 0.75 with 0.70*	3, 4, 5 10
I_c (PKR 2009)	Soil Behavior Type Index, I_c (PKR 2009) is based on a variable stress ratio exponent n, which itself is based on I_c (PKR 2009). An iterative calculation is required to determine I_c (PKR 2009) and its corresponding n (PKR 2009).	I_c (PKR 2009) = $[(3.47 - \log_{10}Q_{tn})^2 + (1.22 + \log_{10}F_r)^2]^{0.5}$	15
n (PKR 2009)	Stress ratio exponent n, based on I_c (PKR 2009). An iterative calculation is required to determine n (PKR 2009) and its corresponding I_c (PKR 2009).	$n \text{ (PKR 2009)} = 0.381 (I_c) + 0.05 (\sigma_v'/P_a) - 0.15$	15
Q_{tn} (PKR 2009)	Normalized tip resistance using a variable stress ratio exponent based on I_c (PKR 2009) and n (PKR 2009). An iterative calculation is required to determine Q_{tn} (PKR 2009).	$Q_{tn} = [(qt - \sigma_v)/P_a](P_a/\sigma_v')^n$ *where* P_a *= atmospheric pressure (100 kPa)* *n = stress ratio exponent described above*	15
FC	Apparent fines content (%)	$FC=1.75(Ic^{3.25}) - 3.7$ $FC=100$ for $I_c > 3.5$ $FC=0$ for $I_c < 1.26$ $FC = 5\%$ if $1.64 < I_c < 2.6$ AND $F_r<0.5$	3
I_c Zone	This parameter is the Soil Behavior Type zone based on the I_c parameter (valid for zones 2 through 7 on SBTn or SBT Qtn charts)	$I_c < 1.31$ Zone = 7 $1.31 < I_c < 2.05$ Zone = 6 $2.05 < I_c < 2.60$ Zone = 5 $2.60 < I_c < 2.95$ Zone = 4 $2.95 < I_c < 3.60$ Zone = 3 $I_c > 3.60$ Zone = 2	3
CD	The contractive / dilative boundary on Robertson's Modified SBTn (contractive/dilative) Chart shown in Figure 6 above. The boundary is marked as CD = 70 on the chart in the relevant paper. Similar to the $Q_{tn,cs} = 70$ line in Figure 4.	$CD = 70 = (Q_{tn} - 11) (1 + 0.06F_r)^{17}$ lower bound of CD = 60: $CD = 60 = (Q_{tn} - 9.5) (1 + 0.06F_r)^{17}$	30

Calculated Parameter	Description	Equation	Ref
I_B	Hyperbolic fit defining the boundary between SBT soil types proposed by Schneider as a better fit than the I_c circles. I_B = 32 represents the boundary for most sand like soils. I_B = 22 represents the upper boundary for most clay like soils. The region between I_B=22 and I_B=32 is the "transitional soil" zone.	$I_B = 100\ (Q_{tn} + 10) / (70 + Q_{tn}\ F_r)$	30
State Param or State Parameter or ψ	The state parameter index, ψ, is defined as the difference between the current void ratio, e, and the critical void ratio, e_c. Positive ψ - contractive soil Negative ψ - dilative soil This is based on the work by Been and Jefferies (1985) and Plewes, Davies and Jefferies (1992) This method uses mean normal stresses based on a uniform value of K_0 or a calculated K_0 using methods described elsewhere in this document	See reference	6, 8
Yield Stress σ_p'	Yield stress is calculated using the following methods 1) General method 2) 1st order approximation using q_tNet (clays) 3) 1st order approximation using Δu_2 (clays) 4) 1st order approximation using q_e (clays) 5) Based on Vs	All stresses in kPa 1) $\sigma_p' = 0.33\cdot(q_t - \sigma_v)^{m'}\ (\sigma_{atm}/100)^{1-m'}$ where $m' = 1 - \frac{0.28}{1+(I_c/2.65)^{25}}$ 2) $\sigma_p' = 0.33\cdot(q_t - \sigma_v)$ 3) $\sigma_p' = 0.54\cdot(\Delta u_2)$ $\Delta u_2 = u_2 - u_0$ 4) $\sigma_p' = 0.60\cdot(q_t - u_2)$ 5) $\sigma_p' = (Vs/4.59)^{1.47}$	19 20 20 20 18
OCR OCR(JS1978) YSR(Mayne2014) YSR (qtNet) YSR (deltaU) YSR (qe) YSR (Vs) OCR (PKR2015)	Over Consolidation Ratio based on 1) Schmertmann (1978) method involving a plot plot of $S_u/\sigma_v' / (S_u/\sigma_v')_{NC}$ and OCR  2) based on Yield stresses described above 3) approximate version based on qtNet 4) approximate version based on Δu 5) approximate version based on effective tip, q_e 6) approximate version based on shear wave velocity, V_s and σ_v' 7) based on Qt	1) requires a user defined value for NC Su/P_c' ratio 2 through 5) *based on yield stresses* 6) YSR (Vs) = $\sigma_p'(Vs) / \sigma_v'$ 7) OCR = $0.25\cdot(Qt)^{1.25}$	9 19 20 20 20 18 32
Es/qt	Intermediate parameter for calculating Young's Modulus, E, in sands. It is the Y axis of the reference chart. Note that Figured 5.59 from reference 5, Lunne, Robertson and Powell, (LRP) has an error. The X axis values are too high by a factor of 10. The plot is based on Baldi's (not Bellotti as cited in	Based on Figure 5.59 in the reference	5, 37

Calculated Parameter	Description	Equation	Ref
	LRP) original Figure 3 where the X axis is: $\frac{q_c}{\sqrt{\sigma'_v}}$ (both in kPa) with a range of 200 to 3000. Figure 5.59 from LRP shows a dimensionless form of the equation, q_{c1}, displaying the same range of values. Figure 5.59's X axis uses $q_{c1} = \left(\frac{q_c}{P_a}\right)\left(\frac{P_a}{\sigma'_v}\right)^{0.5}$ The two expressions are not the same: they differ by a factor of $\frac{\sqrt{P_a}}{P_a}$. With P_a taken to be 100 kPa the factor is 1/10. Substituting typical values of 200 bar (20000 kPa) for q_c and 225 kPa for σ_v' one gets: 20000 / 15 = 1333.33 for Bellotti's axis and $(200/1)(100/225)^{0.5}$ = 200 * (10/15) = 133.3 for LRP's axis (noting that P_a = 1 bar) showing a factor of 10 difference.		
Es or E_s Young's Modulus E	Young's Modulus based on the work done in Italy. There are three types of sands considered in this technique. The user selects the appropriate type for the site from: a) OC Sands b) Aged NC Sands c) Recent NC Sands Each sand type has a family of curves that depend on mean normal stress. The program calculates mean normal stress and linearly interpolates between the two extremes provided in the E_s/q_t chart. E_s is evaluated for an axial strain of 0.1%.	Mean normal stress is evaluated from: $\sigma'_m = \frac{1}{3}(\sigma'_v + \sigma'_h + \sigma'_h)$ where σ_v'= vertical effective stress σ_h'= horizontal effective stress *and* $\sigma_h = K_o \cdot \sigma_v'$ with K_o assumed to be 0.5	5
Delta U/TStress $\Delta u / \sigma_v$	Differential pore pressure ratio with respect to total stress	$= \frac{\Delta u}{\sigma_v}$ *where:* $\Delta u = u - u_{eq}$	39
Delta U/EStress, P Value, Excess Pore Pressure Ratio $\Delta u/\sigma_v'$	Differential pore pressure ratio with respect to effective stress. Key parameter (P, Normalized Pore Pressure Parameter, Excess Pore Pressure Ratio) in the Winckler et. al. static liquefaction method.	$= \frac{\Delta u}{\sigma'_v}$ *where:* $\Delta u = u - u_{eq}$	25, 25a
Su/EStress S_u/σ_v'	Undrained shear strength ratio with respect to vertical effective overburden stress using the S_u (N_{kt}) method	$= Su(N_{kt}) / \sigma_v'$	9, 23
Vs or V_s	Recorded shear wave velocities (not estimated). The shear wave velocities are typically collected over 1 m depth intervals. Each data point over the relevant depth range is assigned the same V_s value.	recorded data	27
Vp or V_p	Recorded compression wave (or P wave) velocities (not estimated). The P wave velocities are typically collected over 1 m depth intervals. Each data point over the relevant depth range is assigned the same V_p value.	recorded data	27

Calculated Parameter	Description	Equation	Ref
V_{s30} V_{s100}	The average shear wave velocity of the near surface materials to a depth of 30 m (100 ft). It is based on the sum of all travel times through all layers in the top 30m (100 ft). V_{s100} is the same calculation as V_{s30} except down to a depth of 100 feet.	$V_{s30} = \frac{total\ thickness\ of\ all\ layers\ to\ 30\ m}{\Sigma\left(\frac{layer\ thickness}{layer\ shear\ wave\ velocity}\right)}$ $V_{s30} = \frac{total\ thickness\ of\ all\ layers\ to\ 30\ m}{\Sigma\ (layer\ travel\ times)}$	38
G_{max}	G_{max} determined from SCPT shear wave velocities (not estimated values). Note that seismic data (V_s) is collected over set depth intervals (typically 1 meter). Each data point over the test segment is assigned the same V_s value. Since soil density changes with depth, slightly different G_{max} values may be calculated over the test depth interval.	$G_{max} = \rho V_s^2$ where ρ is the mass density of the soil determined from the estimated unit weights at each test depth	27
qtNet/G_{max}	Net tip resistance ratio with respect to the small strain modulus G_{max} determined from SCPT shear wave velocities (not estimated values)	$= (qt - \sigma_v) / G_{max}$ where $G_{max} = \rho V_s^2$ and ρ is the mass density of the soil determined from the estimated unit weights at each test depth	15, 28, 30
qUlt	A site specific and client specific parameter for estimating the limiting stress for “crane walk” accessibility	$q_{ult} = CraneWalkFactor \cdot S_u$ Where: *CraneWalkFactor* is client provided	U*
Estimated G_o	Estimated value for small strain shear modulus	$G_o = 0.0188[10^{(0.55Ic + 1,68)}](q_t - \sigma_v)$	15
Estimated E_{25}	Estimated value for Young’s Modulus, E, at a 25% working load	$E_{25} = \alpha_E$ (qtNet) where $\alpha_E = 0.015[10^{(0.55Ic + 1,68)}]$	15
k_{SBT}	Estimated soil permeability derived from Soil Behavior Type (SBT) Chart I_c values.	For $1.0 < I_c \leq 3.27$: $k = 10^{(0.952 - 3.04Ic)}$ in m/s For $3.27 < Ic < 4.0$: $k = 10^{(-4.52 - 1.37Ic)}$ in m/s	35
M or D’ Constrained Modulus	Constrained Modulus based on 1) Robertson, M 2) Mayne, D’	1) Robertson $M = \alpha_M (q_t - \sigma_v)$ $I_c > 2.2$ (fine grained) $\alpha_M = Qt$ when $Qt < 14$ $\alpha_M = 14$ when $Qt > 14$ $Ic < 2.2$ (coarse grained) $\alpha_M = 0.0188\ [10^{(0.55Ic + 1.68)}]$ $D' = \alpha_D (qt - \sigma_v)$ where $\alpha_D = 5$	32 23

Table 1b. CPT Parameter Calculation Methods – Liquefaction Parameters

Calculated Parameter	Description	Equation	Ref
K_{SPT} or K_s	Equivalent clean sand factor for $(N_1)60$	$K_{SPT} = 1 + ((0.75/30) \bullet (FC - 5))$	10
K_{CPT} or K_c (RW1998)	Equivalent clean sand correction for q_{c1N}	$K_{cpt} = 1.0$ for $I_c \leq 1.64$ $K_{cpt} = f(I_c)$ for $I_c > 1.64$ (see reference) $K_c = -0.403\ I_c^4 + 5.581\ I_c^3 - 21.63 I_c^2 + 33.75\ I_c - 17.88$	3, 10
K_c (PKR 2010)	Clean sand equivalent factor to be applied to Q_{tn}	$K_c = 1.0$ for $I_c \leq 1.64$ $K_c = -0.403\ I_c^4 + 5.581\ I_c^3 - 21.63 I_c^2 + 33.75\ I_c - 17.88$ for $I_c > 1.64$	16
$(N_1)_{60cs}I_c$	Clean sand equivalent SPT $(N_1)_{60}I_c$. User has 3 options.	1) $(N_1)_{60cs}Ic = \alpha + \beta((N_1)_{60}I_c)$ 2) $(N_1)_{60cs}Ic = K_{SPT} * ((N_1)_{60}I_c)$ 3) $(q_{c1ncs})/ (N_1)_{60cs}I_c = 8.5\ (1 - I_c/4.6)$ FC ≤ 5%: $\alpha = 0,\ \beta=1.0$ FC ≥ 35% $\alpha = 5.0,\ \beta=1.2$ 5% < FC < 35% $\alpha = \exp[1.76 - (190/FC^2)]$ $\beta = [0.99 + (FC^{1.5}/1000)]$	10 10 5
q_{c1ncs}	Clean sand equivalent q_{c1n}	$q_{c1ncs} = q_{c1n} \bullet K_{cpt}$	3
$Q_{tn,cs}$ (PKR 2010)	Clean sand equivalent for Q_{tn} described above - Q_{tn} being the normalized tip resistance based on a variable stress exponent as defined by Robertson (2009)	$Q_{tn,cs} = Q_{tn} \cdot K_c$ (PKR 2016)	16
Su(Liq)/ESv or $S_u(Liq)/\sigma_v'$	Liquefied shear strength ratio as defined by Olson and Stark	$\frac{S_u(Liq)}{\sigma_v'} = 0.03 + 0.0143(q_{c1})$ Note: σ_v' and s_v' are synonymous	13
Su(Liq)/ESv or $S_u(Liq)/\sigma_v'$ (PKR 2010)	Liquefied shear strength ratio as defined by Robertson (2010)	$\frac{S_u(Liq)}{\sigma_v'}$ Based on a function involving $Q_{tn,cs}$	16
S_u (Liq) (PKR 2010)	Liquefied shear strength derived from the liquefied shear strength ratio and effective overburden stress	$S_u(Liq) = \sigma_v' \cdot \left(\frac{S_u(Liq)}{\sigma_v'}\right)$	16
Cont/Dilat Tip	Contractive / Dilative q_{c1} Boundary based on $(N_1)_{60}$	$(\sigma_v')_{boundary} = 9.58 \times 10^{-4}\ [(N_1)_{60}]^{4.79}$ q_{c1} is calculated from specified q_t(MPa)/N ratio	13
CRR	Cyclic Resistance Ratio (for Magnitude 7.5)	$q_{c1ncs} < 50$: $CRR_{7.5} = 0.833\ [q_{c1ncs}/1000] + 0.05$ $50 \leq q_{c1ncs} < 160$: $CRR_{7.5} = 93\ [q_{c1ncs}/1000]^3 + 0.08$	10
Kg or K_g	Small strain Stiffness Ratio Factor, K_g	$[G_{max}/q_t]/[q_{c1n}^{-m}]$ m = empirical exponent, typically 0.75	26

Calculated Parameter	Description	Equation	Ref
K_g*	Revised K_g factor extended to fine grained soils (Robertson).	$K_g^* = (G_o / q_n)(Q_{tn})^{0.75}$ where q_n is the net tip resistance = $q_t - \sigma_v$	30
SP Distance	State Parameter Distance, Winckler static liquefaction method	Perpendicular distance on Q_{tn} chart from plotted point to state parameter Ψ = -0.05 curve	25
URS NP Fr	Normalized friction ratio point on Ψ = -0.05 curve used in SP distance calculation		25
URS NP Q_{tn}	Normalized tip resistance (Q_{tn}) point on Ψ = -0.05 curve used in SP Distance calculation		25

Table 2. References

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CERTIFICATE OF CALIBRATION

Calibration Information			
Cone Serial Number	EC859	Model	A15 T1500 F15 U35
Date	2023-12-13		
Technician Performing Calibration	Richard Chen	Signature	Chen
Calibration Approved By	Vishrut Khunt	Signature	Vishrut

Lab Condition	As Found	As Left		
Lab Temperature	N/A	24˚C		
Lab Humidity	N/A	29%	Reason for Calibration	Repair

Cone Information				
Tip Stress Limit	1500	bar	Tip End Area	15 cm^2
Friction Stress Limit	15	bar	Friction Surface Area	225 cm^2
Pressure Limit	35	bar	RTD Location	Pressure Carrier
X-Inclinometer Limit	30	degrees	Geophone	X and Z
Y-Inclinometer Limit	30	degrees	Temperature Range	-20°C to 60°C

Baseline Summary: (For Reference Only)

Channel	Units	As Found	As Left
Tip	bar	1.072	0.481
Sleeve	bar	0.002	-0.021
Pressure	bar	1.014	1.013
X-Inclinometer	degrees	0.396	0.014
Y-Inclinometer	degrees	-0.250	0.000
Temperature	˚C	23.239	23.782

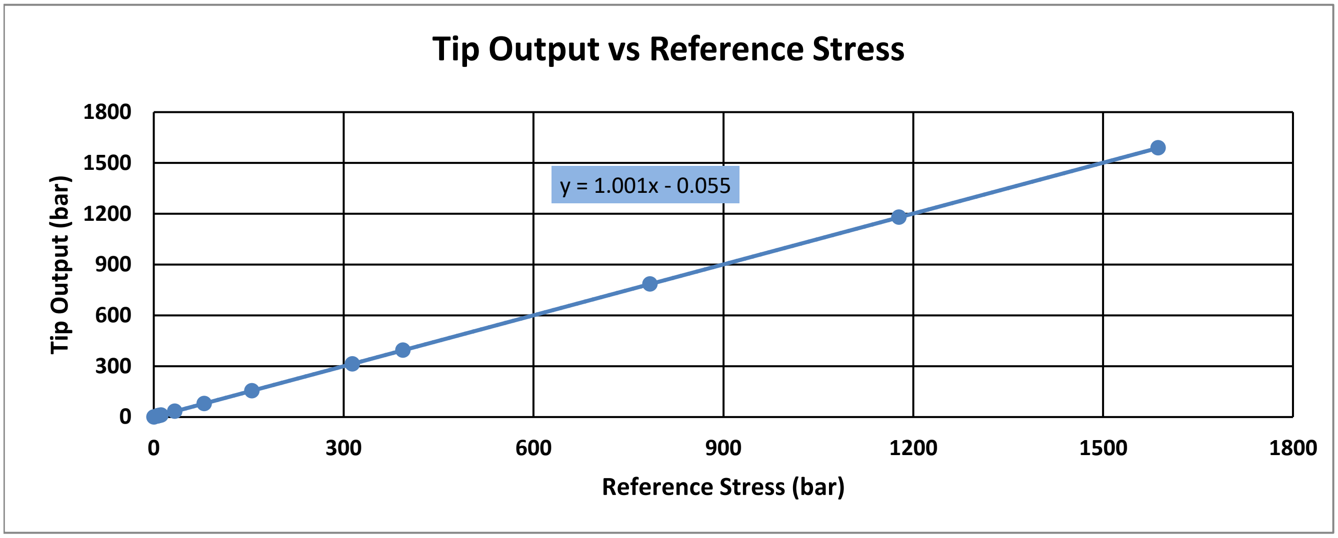
Classified in accordance with ISO 22476-1:2012 Class 1
Classified in accordance with ISO 22476-1:2012 Class 2

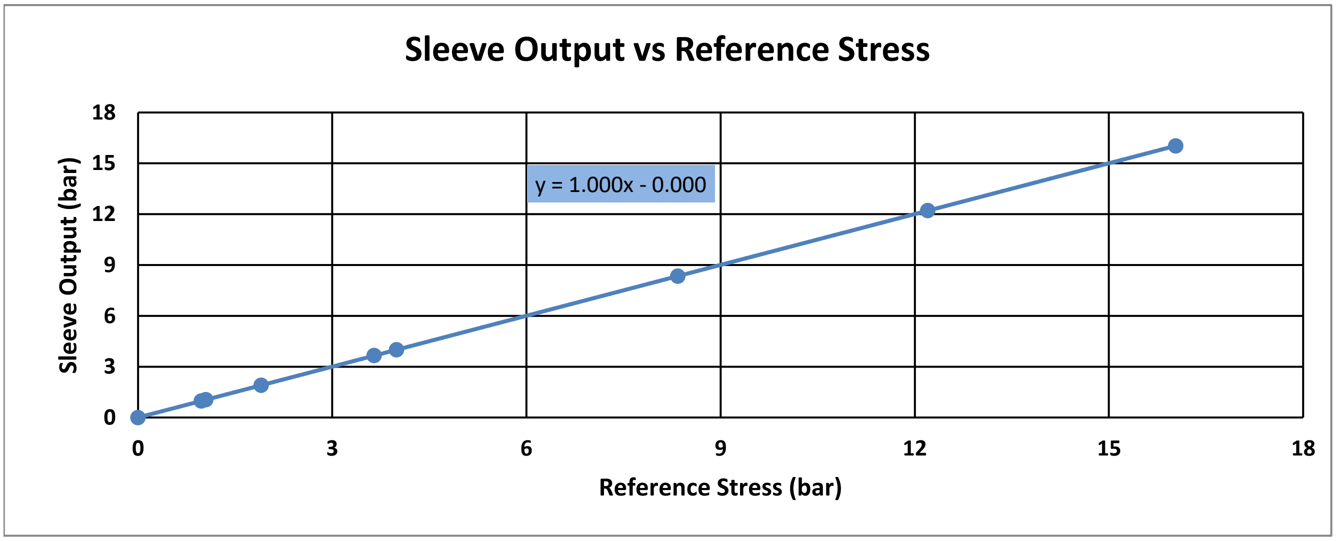
Calibrated in general accordance with the ASTM D5778-20 and D7400-08 standards

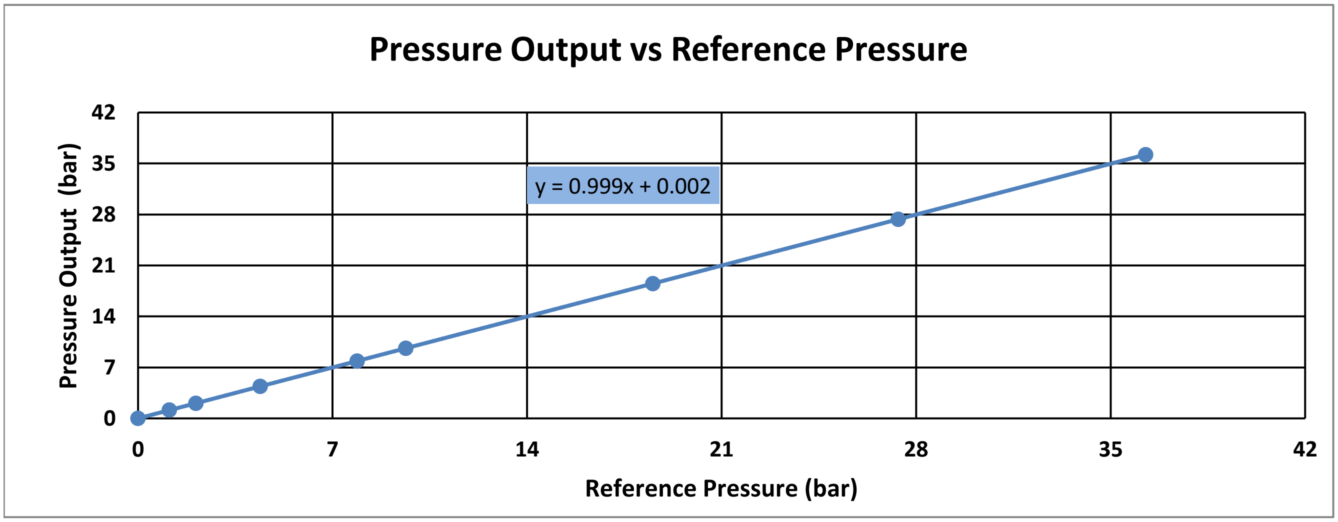
Calibrated with Adara calibration procedure EC_CPTCAL-2.1

Collective uncertainty of the measurement standards conforms to a test uncertainty ratio (TUR) of 3:1 for tip and sleeve measurement and 4:1 for pressure measurement with a confidence level k=2

Cone Output vs Reference Stress/Pressure Plots







Calibration Results

Tip Calibration

As Found			As Left		
Max. Non Linearity	0.07%	PASS	Max. Non Linearity	0.09%	PASS
Calibration Error	0.07%	PASS	Calibration Error	0.10%	PASS

Sleeve Calibration

As Found			As Left		
Max. Non Linearity	0.14%	PASS	Max. Non Linearity	0.03%	PASS
Calibration Error	0.41%	PASS	Calibration Error	0.06%	PASS

Pressure Calibration

As Found			As Left		
Max. Non Linearity	0.03%	PASS	Max. Non Linearity	0.12%	PASS
Calibration Error	0.08%	PASS	Calibration Error	0.12%	PASS

X-Inclinometer Calibration

As Found			As Left		
Max. Non Linearity	N/A	N/A	Max. Non Linearity	0.04%	PASS
Calibration Error	N/A	N/A	Calibration Error	-0.08%	PASS

Y-Inclinometer Calibration

As Found			As Left		
Max. Non Linearity	N/A	N/A	Max. Non Linearity	-0.25%	PASS
Calibration Error	N/A	N/A	Calibration Error	0.50%	PASS

Seismic Calibration

As Found			As Left		
Trigger Delay Error	N/A	N/A	Trigger Delay Error	0.00%	PASS

Temperature Calibration

Full Scale Error	0.27%	PASS	

Channel	Cold	Room	Hot	Units
Ref_Temp	5.3	23.3	43.3	˚C
Tip	3.954	0.275	-1.338	bar
Sleeve	0.051	-0.011	-0.016	bar
Pressure	1.074	1.061	1.052	bar
Temperature	5.340	23.106	43.189	˚C

Tip Temperature Coefficient	-0.138 bar/˚C	PASS
Sleeve Temperature Coefficient	-0.002 bar/˚C	PASS
Pressure Temperature Coefficient	-0.001 bar/˚C	PASS

Testing Equipment Details

Testing Machines	Model Number	Serial Number	Calibration Number	Due Date
Tip Load Cell	Precision	P-10289	100490	2025-09-18
Sleeve Load Cell	Precision	P-10868	100579	2025-10-01
Digital Loadcell Indicator	4215	62140	100490	2024-07-18
Fluke Reference Pressure Monitor	RPM4 A10Ms	3061	100214	2024-01-05
Tektronix Function Generator	AFG3021B	C030955	100751	2024-10-20
Thermometer	THS-222-555	D23255834	100410	2024-07-11
Thermometer	THS-222-555	D23255829	100410	2024-07-11
Thermometer	THS-222-555	D20345575	100565	2024-07-14

Adara Error Definitions

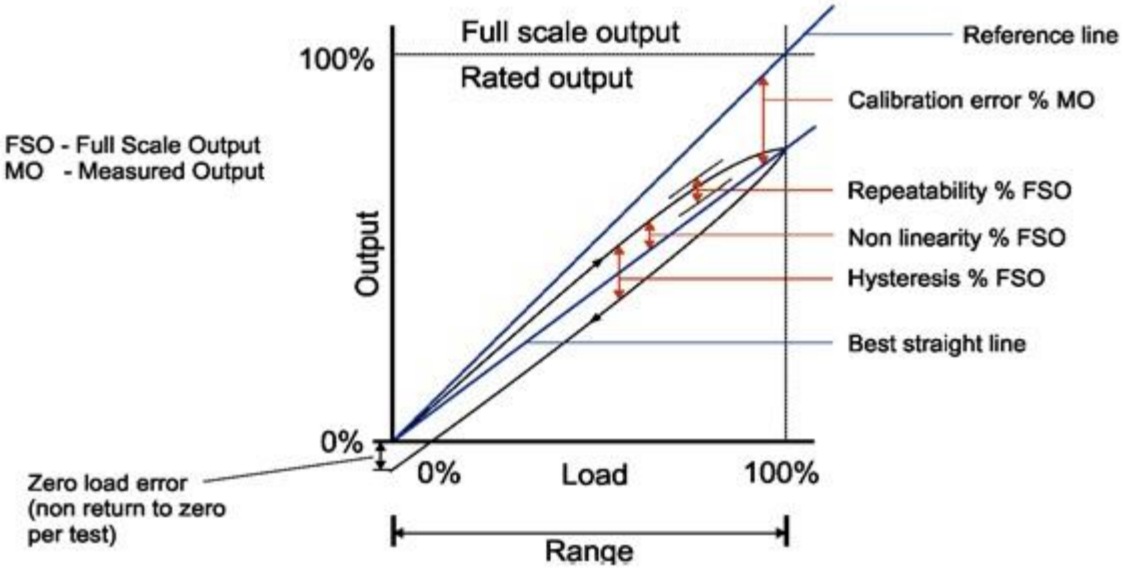


Figure 1: Definition of Calibration Terms for Load Cell and Transducers (Adapted from [1])

Actual Sensitivity	The slope of the best fit line through all data points starting at zero load.
Slope Error	The error in the best fit line compared to the ideal linear calibration in % . Slope Error = (Best Fit Slope - Ideal Slope) / Ideal Slope
Maximum Non Linearity	This value represents the maximum error (absolute value) relative to the best fit line considering each calibration point starting at loads greater than approximately 10% of FSO. The reported errors are a percent error of FSO. Adara's Pass/Fail criteria is 0.5% of FSO (ASTM is 0.5% of FSO at loads > 20% FSO).
Calibration Error	This value represents the maximum error (absolute value) in the recorded load value as compared to the actual load value for each calibration point for loads greater than approximately 10% of FSO. Adara's Pass/Fail criteria for the tip and sleeve is 0.5% of MO and 1.0% of MO for the pore pressure (ASTM for the tip and sleeve is 1.5% and 1.0% of MO respectively at loads greater than 20% of FSO)

Temperature Check Passing Criteria

Tip Temperature Coefficient	<0.200	bar/˚C
Sleeve Temperature Coefficient	<0.005	bar/˚C
Pressure Temperature Coefficient	<0.0196	bar/˚C

ASTM D5778-20 Annex A Summary [1]

A1.4 Force Transducer Calibration Requirements

A1.4.1 states the following limits:

Non Linearity	Tip	≤ +0.5% of FSO
	Sleeve	≤ +1.0% of FSO
Calibration Error	Tip	≤ +1.5% of MO at loads > 20% FSO
	Sleeve	≤ +1.0% of MO at loads > 20% FSO

A1.5 Pressure Transducer Calibrations

A1.5.1 limits:

Non Linearity	Pore Pressure	≤ +1.0% of FSO
Calibration Error	Pore Pressure	not specified

ISO 22476 -1:2012 Summary [2]

Section 5.2 states the following allowable minimum accuracy

Class 1	Cone Resistance	35 kPa or 5%
	Sleeve Friction	5 kPa or 10%
	Pore Pressure	10 kPa or 2%
Class 2	Cone Resistance	100 kPa or 5%
	Sleeve Friction	15 kPa or 15%
	Pore Pressure	25 kPa or 3%

Note: ISO Compliance is based on low end calibration only.

References

[1] ASTM D5778-20. "Standard Test Method for Electronic Friction Cone and Piezocone Penetration Testing of Soils". ASTM, West Conshohocken, PA, USA.

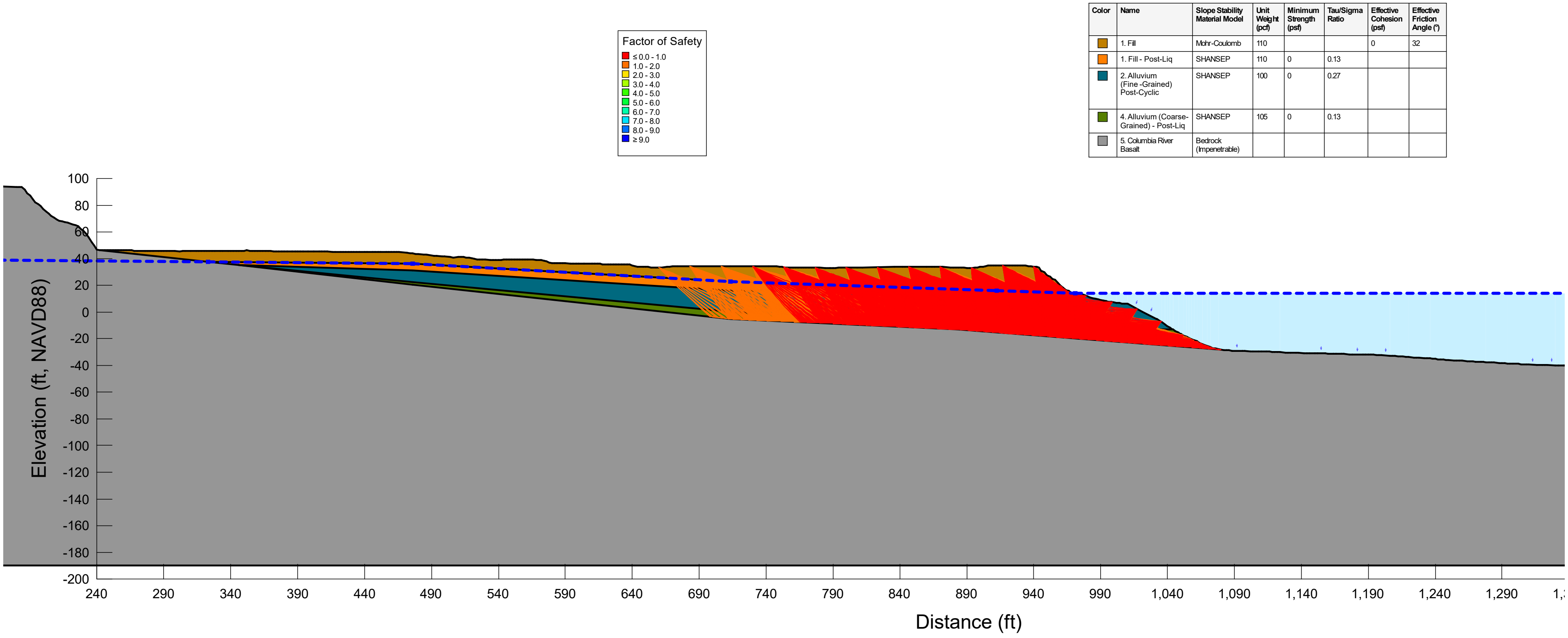
[2] ISO 22476-1:2012. "Geotechnical investigation and testing - Field Testing - Part 1: Electrical cone and piezocone penetration test". ISO, Geneva, Switzerland.

ASTM D7400-08. "Standard Test Methods for Downhole Seismic Testing". ASTM, West Conshohocken, PA, USA.

APPENDIX C – SLOPE STABILITY ANALYSIS

Cross Section A-A'

Post-Liquefaction Static Stability



Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Minimum Strength (psf)	Tau/Sigma Ratio	Effective Cohesion (psf)	Effective Friction Angle (°)
🟫	1. Fill	Mohr-Coulomb	110			0	32
🟧	1. Fill - Post-Liq	SHANSEP	110	0	0.13		
🟦	2. Alluvium (Fine -Grained) Post-Cyclic	SHANSEP	100	0	0.27		
🟩	4. Alluvium (Coarse-Grained) - Post-Liq	SHANSEP	105	0	0.13		
⬜	5. Columbia River Basalt	Bedrock (Impenetrable)					

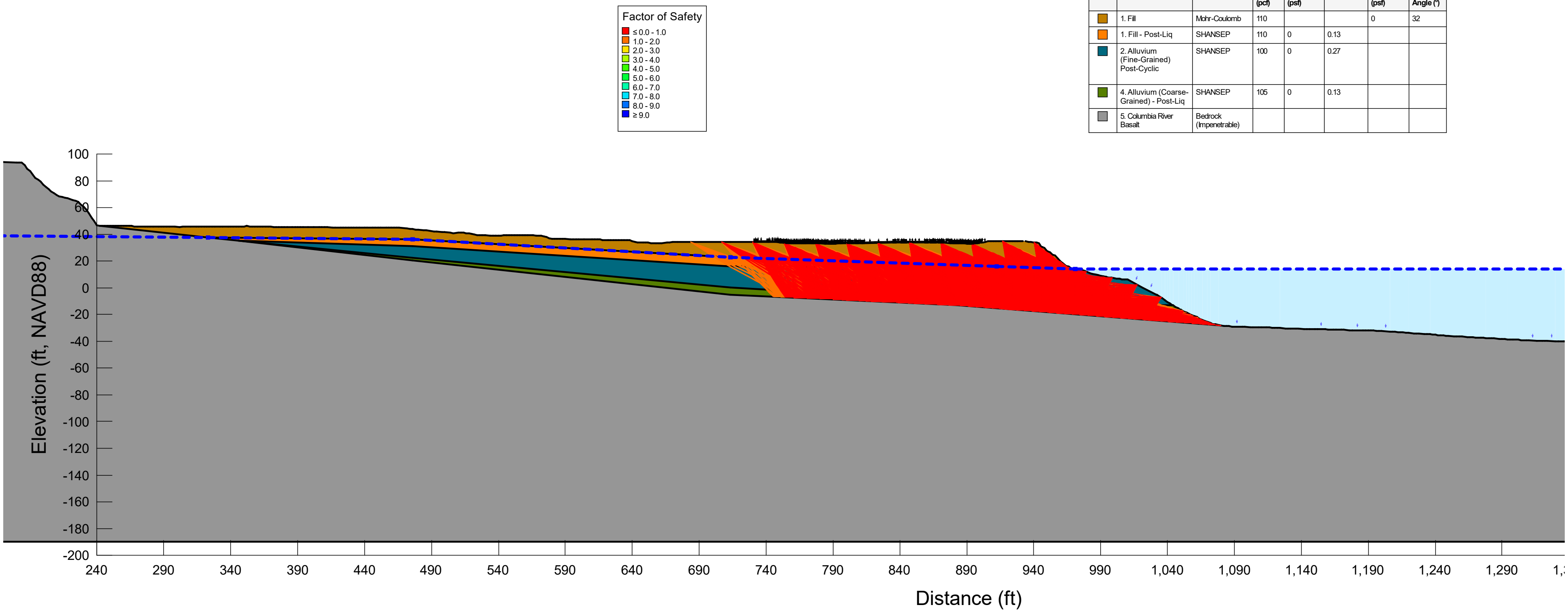
Note: Safety factors less than 1.0 depicted by red slip surfaces, with orange slip surfaces representing safety factors greater than 1.0.

Cross Section A-A'

Post-Liquefaction Static Stability

Tank Surcharge Pressure (2,000 psf)

Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Minimum Strength (psf)	Tau/Sigma Ratio	Effective Cohesion (psf)	Effective Friction Angle (°)
🟫	1. Fill	Mohr-Coulomb	110			0	32
🟧	1. Fill - Post-Liq	SHANSEP	110	0	0.13		
🟦	2. Alluvium (Fine-Grained) Post-Cyclic	SHANSEP	100	0	0.27		
🟩	4. Alluvium (Coarse-Grained) - Post-Liq	SHANSEP	105	0	0.13		
⬜	5. Columbia River Basalt	Bedrock (Impenetrable)					



Note: Safety factors less than 1.0 depicted by red slip surfaces, with orange slip surfaces representing safety factors greater than 1.0.

Appendix C

Risk Assessment

CRITICAL SYSTEMS RISK ASSESSMENT

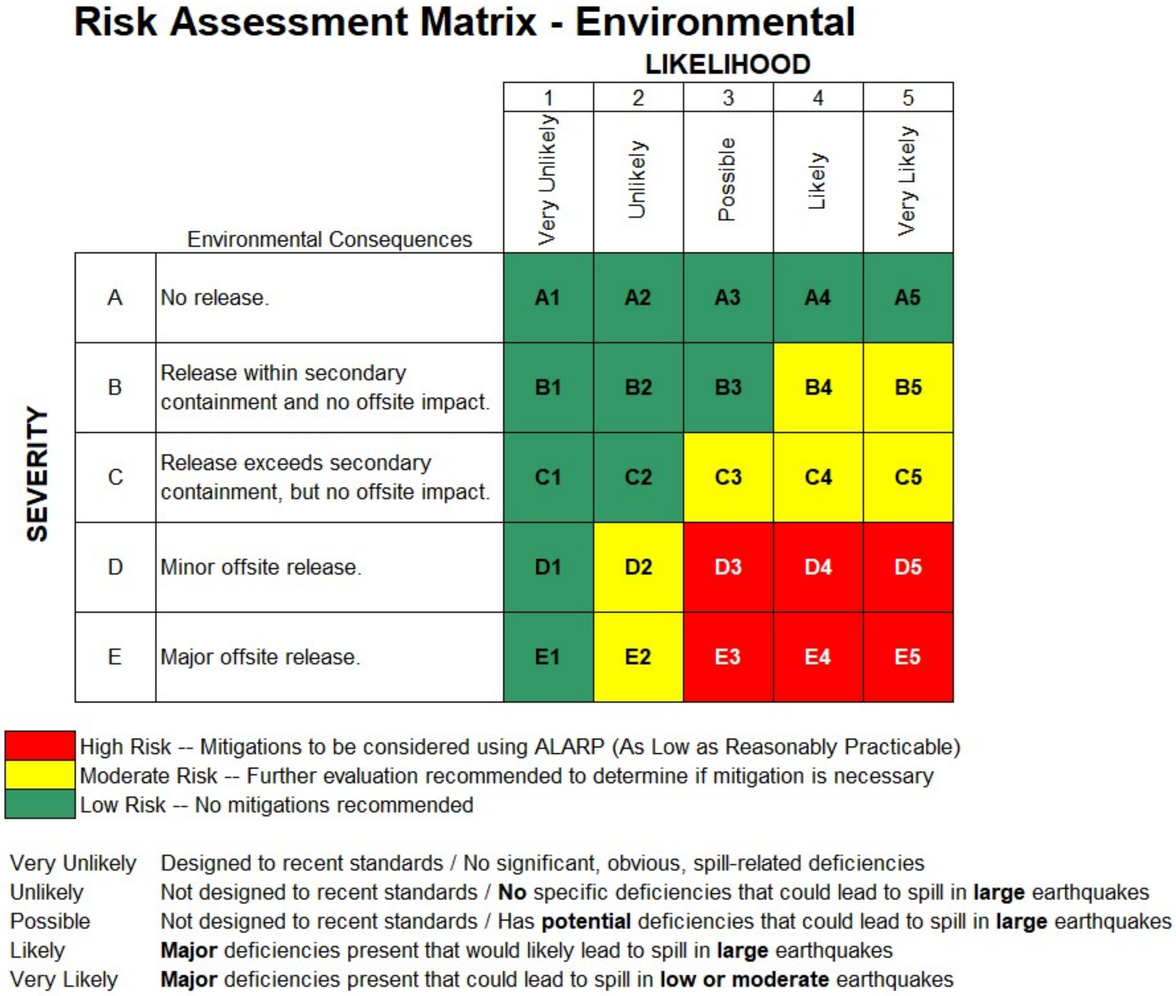
Purpose:	To identify and prioritize critical structures, equipment, tanks, and systems and the performance requirements during and following an earthquake with regards to prevention and containment of oil spills.
Scope:	This study will address all facility components covered by the Rules.
Boundaries:	The team will consider possible scenarios due to earthquakes that may realistically occur and result in an uncontained spill, uncontrolled fire, explosion, or toxic release at the terminal. The following items will be excluded from the scope of this study: • Failures due to non-earthquake related causes • Life-safety considerations that are not directly caused by a spill that occurs due to an earthquake (e.g. life-safety concerns from occupants of a building that collapses)
Process:	***Before the Risk Assessment Session*** • Prepare the charter for the risk assessment. • Prepare a draft assessment based on known industry and terminal practice and knowledge of this specific terminal gained through review of terminal documentation • SGH engineers will perform a structural "walkdown" review of the facility • SGH will prepopulate the risk matrix based on the walkdown review, preliminary geotechnical review, and other factors ***During the Risk Assessment Session*** • Review the risk assessment process and techniques to be used. • Present an overview of the risk assessment matrix. • Review the pre-developed list of systems and components • Identify additional systems and components

- For each physical area of the terminal, identify the following:
    - Key components or systems that require documentation according to the Rules
    - Which components or systems contain hydrocarbons covered by the rules where spill is a concern
    - Safety systems that are being relied on for mitigation or response following an earthquake as related to the scope of the Rules
- For each critical system, identify key components of that system and for each component perform the following:
    - Identify the possible nature of earthquake performance as related to the Rules (e.g. collapse, damage resulting in spill, functional failure)
    - Identify the **likelihood** of possible failure / unacceptable performance, consistent with the risk matrix, based on known properties of the system and visual reviews. (Note: this is subject to revision based on more detailed evaluation or additional data)
    - Identify the **severity** of possible safety or environmental consequences, consistent with the risk matrix
    - Assign a risk level consistent with the risk matrix
- Document team findings

After the Risk Assessment Session

- Update the findings of the risk assessment as appropriate based on further evaluation or additional data
- Use the risk assessment results as needed in development of the facilities mitigation plan, as required by the Rules

Risk Assessment Matrices



Risk Assessment Matrix - Environmental

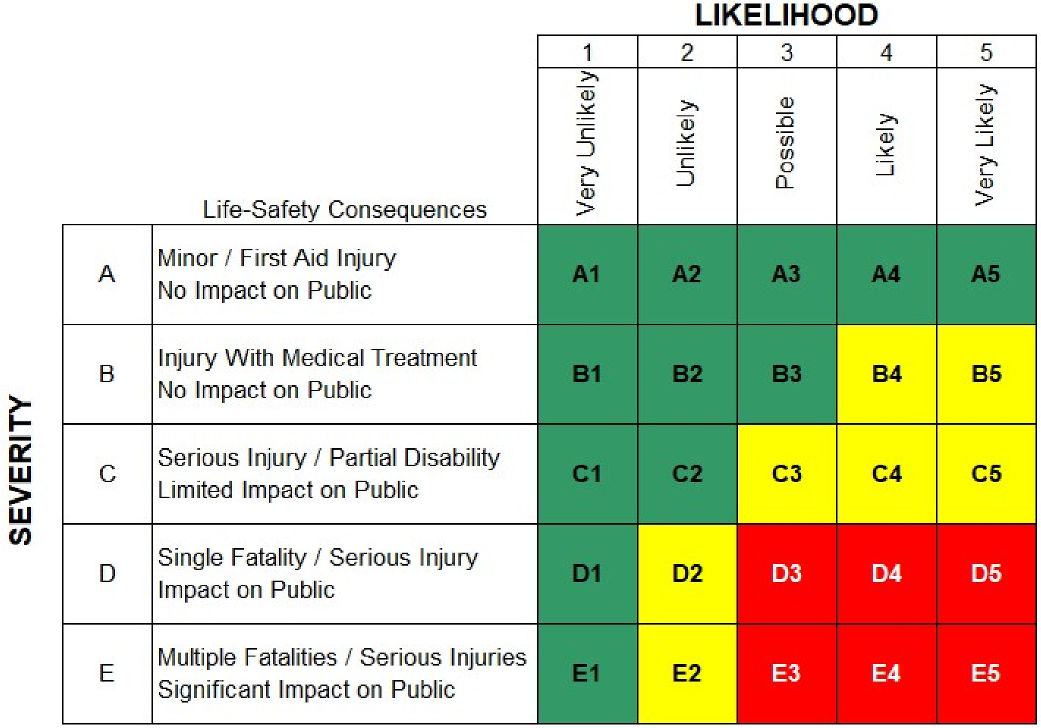
LIKELIHOOD

SEVERITY	Environmental Consequences	1 Very Unlikely	2 Unlikely	3 Possible	4 Likely	5 Very Likely
A	No release.	🟩 **A1**	🟩 **A2**	🟩 **A3**	🟩 **A4**	🟩 **A5**
B	Release within secondary containment and no offsite impact.	🟩 **B1**	🟩 **B2**	🟩 **B3**	🟨 **B4**	🟨 **B5**
C	Release exceeds secondary containment, but no offsite impact.	🟩 **C1**	🟩 **C2**	🟨 **C3**	🟨 **C4**	🟨 **C5**
D	Minor offsite release.	🟩 **D1**	🟨 **D2**	🟥 **D3**	🟥 **D4**	🟥 **D5**
E	Major offsite release.	🟩 **E1**	🟨 **E2**	🟥 **E3**	🟥 **E4**	🟥 **E5**

🟥 High Risk -- Mitigations to be considered using ALARP (As Low as Reasonably Practicable)
🟨 Moderate Risk -- Further evaluation recommended to determine if mitigation is necessary
🟩 Low Risk -- No mitigations recommended

Very Unlikely	Designed to recent standards / No significant, obvious, spill-related deficiencies
Unlikely	Not designed to recent standards / **No** specific deficiencies that could lead to spill in **large** earthquakes
Possible	Not designed to recent standards / Has **potential** deficiencies that could lead to spill in **large** earthquakes
Likely	**Major** deficiencies present that would likely lead to spill in **large** earthquakes
Very Likely	**Major** deficiencies present that could lead to spill in **low or moderate** earthquakes

Risk Assessment Matrix - Life Safety



			LIKELIHOOD				
			1	2	3	4	5
		Life-Safety Consequences	Very Unlikely	Unlikely	Possible	Likely	Very Likely
SEVERITY	A	Minor / First Aid Injury No Impact on Public	A1	A2	A3	A4	A5
	B	Injury With Medical Treatment No Impact on Public	B1	B2	B3	B4	B5
	C	Serious Injury / Partial Disability Limited Impact on Public	C1	C2	C3	C4	C5
	D	Single Fatality / Serious Injury Impact on Public	D1	D2	D3	D4	D5
	E	Multiple Fatalities / Serious Injuries Significant Impact on Public	E1	E2	E3	E4	E5

🟥	High Risk -- Mitigations to be considered using ALARP (As Low as Reasonably Practicable)
🟨	Moderate Risk -- Further evaluation recommended to determine if mitigation is necessary
🟩	Low Risk -- No mitigations recommended

Very Unlikely	Designed to recent standards / No significant, obvious, spill-related, life-safety deficiencies
Unlikely	Not designed to recent standards / **No** specific deficiencies that could lead to spill-related, life-safety concerns in **large** earthquakes
Possible	Not designed to recent standards / Has **potential** deficiencies that could lead to spill-related, life-safety concerns in **large** earthquakes
Likely	**Major** deficiencies present that would likely lead to spill-related, life-safety concerns in **large** earthquakes
Very Likely	**Major** deficiencies present that could lead to spill-related, life-safety concerns in **low or moderate** earthquakes

Risk Assessment Report

Date:
March 28, 2024

Location:
Virtual

Attendees:
Gayle S. Johnson, P.E., SGH, Senior Principal (Facilitator)
William M. Bruin, P.E., SGH, Senior Principal
Julie A. Galbraith, P.E., SGH, Senior Project Manager
Luis H. Palacios, P.E., SGH, Senior Technical Manager
Justin D. Reynolds, P.E., SGH, Senior Project Manager
Jun O. Tucay, P.E., S.E., SGH, Senior Consulting Engineer
Benjamin Serna, P.E., G.E., Gannett Fleming, Principal Geotechnical Engineer
Tim Fluitt, NuStar Energy, Sr. Project Engineer
Ryan Groesbeck, NuStar Energy, Terminal Manager, Portland Terminal
Kyle Bell, NuStar Energy, Operations Supervisor, Portland Terminal
Buch Buchanan, NuStar Energy, Vice President Technical Services
Chris Vratil, NuStar Energy, Senior Director of West Terminal Operations
John Koenig, NuStar Energy, Vice President of Engineering
Tony Valladares, NuStar Energy, Director of HSE, Western Region
Chris Rulon, NuStar Energy, Vice President and Assistant General Council
Jaime White, NuStar Energy, Mgr. Regulatory Affairs and Legal Projects

All Terminal Assets

						Risk Assessment Rankings						
							Severity					
Location	Item Type	Identification	Contents	Out of Service	DOT/PHMSA?	Likelihood WITHOUT Soil Displacements	Environmental	Safety	Risk Score	Likelihood WITH Soil Displacements	Risk Score	Item or Score Notes
Tank Farm 1												
	Tank	2113	Biodiesel			3. Possible	B. Release within secondary containment	B. Injury With Medical Treatment	B3	4.Likely	B4	
	Tank	2511	Distillate products		Yes	3. Possible	B. Release within secondary containment	B. Injury With Medical Treatment	B3	4.Likely	B4	
	Tank	2512	Distillate products		Yes	3. Possible	B. Release within secondary containment	B. Injury With Medical Treatment	B3	4.Likely	B4	
	Tank	3510	Ethanol			3. Possible	B. Release within secondary containment	C. Serious Injury / Limited Impact on Public	C3	3.Possible	C3	Higher flammability / volatility
	Tank	3605	Biodiesel			3. Possible	B. Release within secondary containment	B. Injury With Medical Treatment	B3	4.Likely	B4	
	Tank	5901	Gasoline products		Yes	3. Possible	B. Release within secondary containment	C. Serious Injury / Limited Impact on Public	C3	3.Possible	C3	Higher flammability / volatility
	Tank	5902	Distillate products		Yes	3. Possible	B. Release within secondary containment	B. Injury With Medical Treatment	B3	3.Possible	B3	
	Tank	703	Distillate products		Yes	3. Possible	B. Release within secondary containment	B. Injury With Medical Treatment	B3	3.Possible	B3	
	Tank	1315	N/A	Yes		2. Unlikely	A. No Release	A. Minor / First Aid Injury	A2	3.Possible	A3	Out of service
	Tank	1316	N/A	Yes		2. Unlikely	A. No Release	A. Minor / First Aid Injury	A2	3.Possible	A3	Out of service
	Tank	1104	N/A	Yes		2. Unlikely	A. No Release	A. Minor / First Aid Injury	A2	3.Possible	A3	Out of service
	Tank	17	Vapor Tank			2. Unlikely	A. No Release	A. Minor / First Aid Injury	A2	4.Likely	A4	
		Total										
	Piping (DOT/PHMSA)	Flammable Fuels			Yes	3. Possible	B. Release within secondary containment	C. Serious Injury / Limited Impact on Public	C3	4. Likely	C4	Higher flammability / volatility
	Piping (DOT/PHMSA)	Non-flammable fuels			Yes	3. Possible	B. Release within secondary containment	B. Injury With Medical Treatment	B3	4. Likely	B4	
	Piping	Flammable Fuels				3. Possible	B. Release within secondary containment	C. Serious Injury / Limited Impact on Public	C3	4. Likely	C4	Higher flammability / volatility
	Piping	Non-flammable fuels				3. Possible	B. Release within secondary containment	B. Injury With Medical Treatment	B3	4. Likely	B4	
	Secondary Containment	N/A			Yes	3. Possible	E. Major offsite release	C. Serious Injury / Limited Impact on Public	E3	4. Likely	E4	
Tank Farm 2												
	Tank	2020	Gasoline products		Yes	3. Possible	B. Release within secondary containment	C. Serious Injury / Limited Impact on Public	C3	3.Possible	C3	Higher flammability / volatility
	Tank	2021	Gasoline products		Yes	3. Possible	B. Release within secondary containment	C. Serious Injury / Limited Impact on Public	C3	4.Likely	C4	Higher flammability / volatility
	Tank	2022	Gasoline products		Yes	3. Possible	B. Release within secondary containment	C. Serious Injury / Limited Impact on Public	C3	4.Likely	C4	Higher flammability / volatility
	Tank	3614	Distillate products		Yes	3. Possible	B. Release within secondary containment	B. Injury With Medical Treatment	B3	3.Possible	B3	
	Tank	5919	Distillate products		Yes	3. Possible	B. Release within secondary containment	B. Injury With Medical Treatment	B3	4.Likely	B4	
	Tank	5618	Gasoline products		Yes	2. Unlikely	B. Release within secondary containment	C. Serious Injury / Limited Impact on Public	C2	4.Likely	C4	Higher flammability / volatility
		Total										
	Piping (DOT/PHMSA)	Flammable Fuels			Yes	3. Possible	B. Release within secondary containment	C. Serious Injury / Limited Impact on Public	C3	4. Likely	C4	Higher flammability / volatility
	Piping (DOT/PHMSA)	Non-flammable fuels			Yes	3. Possible	B. Release within secondary containment	B. Injury With Medical Treatment	B3	4. Likely	B4	
	Secondary Containment	N/A			Yes	3. Possible	E. Major offsite release	C. Serious Injury / Limited Impact on Public	E3	4. Likely	E4	

All Terminal Assets (continued)

						Risk Assessment Rankings						
							Severity					
Location	Item Type	Identification	Contents	Out of Service	DOT/PHMSA?	Likelihood WITHOUT Soil Displacements	Environmental	Safety	Risk Score	Likelihood WITH Soil Displacements	Risk Score	Item or Score Notes
Tank Farm 3 & 4 (shared containment)												
Tank Farm 3	Tank	8006	Gasoline products		Yes	3. Possible	B. Release within secondary containment	C. Serious Injury / Limited Impact on Public	C3	4.Likely	C4	Higher flammability / volatility
	Tank	5209	Distillate products		Yes	2. Unlikely	B. Release within secondary containment	B. Injury With Medical Treatment	B2	4.Likely	B4	
	Tank	8007	Gasoline products		Yes	2. Unlikely	B. Release within secondary containment	C. Serious Injury / Limited Impact on Public	C2	4.Likely	C4	Higher flammability / volatility
	Tank	8308	Distillate products		Yes	2. Unlikely	B. Release within secondary containment	B. Injury With Medical Treatment	B2	4.Likely	B4	
Tank Farm 4	Tank	1009	Distillate products		Yes	3. Possible	B. Release within secondary containment	B. Injury With Medical Treatment	B3	4.Likely	B4	
	Tank	3203	Gasoline products		Yes	3. Possible	B. Release within secondary containment	C. Serious Injury / Limited Impact on Public	C3	4.Likely	C4	Higher flammability / volatility
	Tank	6408	Gasoline products		Yes	3. Possible	B. Release within secondary containment	C. Serious Injury / Limited Impact on Public	C3	4.Likely	C4	Higher flammability / volatility
	Tank	1010	Distillate products		Yes	2. Unlikely	B. Release within secondary containment	B. Injury With Medical Treatment	B2	4.Likely	B4	
	Tank	1011	Ethanol			2. Unlikely	B. Release within secondary containment	C. Serious Injury / Limited Impact on Public	C2	4.Likely	C4	Higher flammability / volatility
	Tank	2705	Distillate products		Yes	2. Unlikely	B. Release within secondary containment	B. Injury With Medical Treatment	B2	4.Likely	B4	
	Tank	2706	Gasoline products		Yes	2. Unlikely	B. Release within secondary containment	C. Serious Injury / Limited Impact on Public	C2	4.Likely	C4	Higher flammability / volatility
	Tank	3201	Ethanol			2. Unlikely	B. Release within secondary containment	C. Serious Injury / Limited Impact on Public	C2	4.Likely	C4	Higher flammability / volatility
	Tank	3204	Gasoline products		Yes	2. Unlikely	B. Release within secondary containment	C. Serious Injury / Limited Impact on Public	C2	4.Likely	C4	Higher flammability / volatility
	Tank	4402	Gasoline products		Yes	2. Unlikely	B. Release within secondary containment	C. Serious Injury / Limited Impact on Public	C2	4.Likely	C4	Higher flammability / volatility
	Tank	4507	Gasoline products		Yes	2. Unlikely	B. Release within secondary containment	C. Serious Injury / Limited Impact on Public	C2	4.Likely	C4	Higher flammability / volatility
		Total										
	Piping (DOT/PHMSA)	Flammable Fuels			Yes	3. Possible	B. Release within secondary containment	C. Serious Injury / Limited Impact on Public	C3	4. Likely	C4	Higher flammability / volatility
	Piping (DOT/PHMSA)	Non-flammable fuels			Yes	3. Possible	B. Release within secondary containment	B. Injury With Medical Treatment	B3	4. Likely	B4	
	Piping	Flammable Fuels				3. Possible	B. Release within secondary containment	C. Serious Injury / Limited Impact on Public	C3	4. Likely	C4	Higher flammability / volatility
	Piping	Non-flammable fuels				3. Possible	B. Release within secondary containment	B. Injury With Medical Treatment	B3	4. Likely	B4	
	Secondary Containment	N/A			Yes	3. Possible	E. Major offsite release	C. Serious Injury / Limited Impact on Public	E3	4. Likely	E4	
Tank Farm 5												
	Tank	10026	Distillate products		Yes	3. Possible	B. Release within secondary containment	B. Injury With Medical Treatment	B3	4.Likely	B4	
	Tank	10027	Distillate products		Yes	3. Possible	B. Release within secondary containment	B. Injury With Medical Treatment	B3	4.Likely	B4	
		Total										
	Piping (DOT/PHMSA)	Non-flammable fuels			Yes	3. Possible	B. Release within secondary containment	B. Injury With Medical Treatment	B3	4. Likely	B4	
	Secondary Containment	N/A			Yes	3. Possible	E. Major offsite release	B. Injury With Medical Treatment	E3	4. Likely	E4	
Other Tanks/Liquids (Outside Main Yards)												
	Tank	181	Gasoline Additives			2. Unlikely	B. Release within secondary containment	B. Injury With Medical Treatment	B2	3. Possible	B3	
	Tank	23	Gasoline Additives	Yes		2. Unlikely	A. No Release	A. Minor / First Aid Injury	A2	3. Possible	A3	Out of service
	Tank	4	0	Yes		2. Unlikely	A. No Release	A. Minor / First Aid Injury	A2	4. Likely	A4	Out of service
	Tank	212	0	Yes		2. Unlikely	A. No Release	A. Minor / First Aid Injury	A2	4. Likely	A4	Out of service
	Tank	1015	N/A	Yes		2. Unlikely	A. No Release	A. Minor / First Aid Injury	A2	4. Likely	A4	Out of service
	Tank	1016	N/A	Yes		2. Unlikely	A. No Release	A. Minor / First Aid Injury	A2	4. Likely	A4	Out of service
	Oil Water Separator	01 OWS	Water			2. Unlikely	C. Exceeds secondary containment, but no off	B. Injury With Medical Treatment	C2	4. Likely	C4	
	Drums	Drummed Waste Storage				3. Possible	B. Release within secondary containment	A. Minor / First Aid Injury	B3	4. Likely	B4	South of Yard #2

All Terminal Assets (continued)

						Risk Assessment Rankings						
							Severity					
Location	Item Type	Identification	Contents	Out of Service	DOT/PHMSA?	Likelihood WITHOUT Soil Displacements	Environmental	Safety	Risk Score	Likelihood WITH Soil Displacements	Risk Score	Item or Score Notes
Truck Loading Rack												
	Tank (Underground)	EMS Overfill Tank				2. Unlikely	C. Exceeds secondary containment, but n	A. Minor / First Aid Injury	C2	3. Possible	C3	
	Loading Rack Structure	Truck Loading Racks				2. Unlikely	B. Release within secondary containmer	A. Minor / First Aid Injury	B2	3. Possible	B3	
P-2 Dock (North Wharf)												
	Marine Structure	P-2 Wharf				3. Possible	E. Major offsite release	B. Injury With Medical Treatment	E3	4. Likely	E4	
	Building	Dock Office				2. Unlikely	A. No Release	B. Injury With Medical Treatment	B2	4. Likely	B4	
	Piping	Flammable Fuels				3. Possible	E. Major offsite release	C. Serious Injury / Limited Impact on Public	E3	4. Likely	E4	Higher flammability / volatility
	Piping	Non-flammable fuels				3. Possible	E. Major offsite release	B. Injury With Medical Treatment	E3	4. Likely	E4	
P-3 Dock (South Wharf)												
	Marine Structure	P-3 Wharf				3. Possible	E. Major offsite release	B. Injury With Medical Treatment	E3	4. Likely	E4	
	Building	Dock Office				2. Unlikely	A. No Release	B. Injury With Medical Treatment	B2	4. Likely	B4	
	Piping	Flammable Fuels				3. Possible	E. Major offsite release	C. Serious Injury / Limited Impact on Public	E3	4. Likely	E4	Higher flammability / volatility
	Piping	Non-flammable fuels				3. Possible	E. Major offsite release	B. Injury With Medical Treatment	E3	4. Likely	E4	
Buildings												
	Building	#4 Operations				3. Possible	A. No Release	A. Minor / First Aid Injury	A3	3. Possible	A3	Conc masonry
	Building	#5 Maintenance Shop				3. Possible	A. No Release	A. Minor / First Aid Injury	A3	4. Likely	A4	Near P-2
	Building	#8 Garage				3. Possible	A. No Release	A. Minor / First Aid Injury	A3	4. Likely	A4	Conc masonry
	Building	#10 Foam House				2. Unlikely	A. No Release	C. Serious Injury / Limited Impact on Public	C2	3. Possible	C3	Fire Foam for TLR
	Building	#11				2. Unlikely	A. No Release	A. Minor / First Aid Injury	A2	3. Possible	A3	Next to #10
	Building	#12 Storage				3. Possible	A. No Release	A. Minor / First Aid Injury	A3	4. Likely	A4	Near P-2
	Building	Main Office				2. Unlikely	A. No Release	A. Minor / First Aid Injury	A2	3. Possible	A3	
	Building	#7				2. Unlikely	A. No Release	A. Minor / First Aid Injury	A2	3. Possible	A3	
	Building	#1				2. Unlikely	A. No Release	A. Minor / First Aid Injury	A2	4. Likely	A4	Btwn Yard 4&5; old office
	Building	#13 Foam House				2. Unlikely	A. No Release	C. Serious Injury / Limited Impact on Public	C2	4. Likely	C4	Fire Foam for Yards 1,2,3,5
	Building	#15				3. Possible	A. No Release	A. Minor / First Aid Injury	A3	3. Possible	A3	Conc masonry
	Building	#16				3. Possible	A. No Release	A. Minor / First Aid Injury	A3	3. Possible	A3	Conc masonry
	Building	#17				2. Unlikely	A. No Release	A. Minor / First Aid Injury	A2	4. Likely	A4	East Yard #1
Overall Terminal												
	Emerygency Response	Operator Staffing				1. Very Unlikely	A. No Release	A. Minor / First Aid Injury	A1	1. Very Unlikely	A1	
	Emerygency Response	Communications				1. Very Unlikely	A. No Release	A. Minor / First Aid Injury	A1	1. Very Unlikely	A1	Backup radios; backup lighting
	Power	Transformers				2. Unlikely	A. No Release	B. Injury With Medical Treatment	B2	3. Possible	B3	no backup power for main equipment
	Power	Steam Generator				2. Unlikely	A. No Release	A. Minor / First Aid Injury	A2	2. Unlikely	A2	Main office
	Fire System	Water Main				5. Very Likely	A. No Release	D. Single Fatality / Impact on Public	D5	5. Very Likely	D5	no backup water sources
	Fire System	Foam System				5. Very Likely	A. No Release	D. Single Fatality / Impact on Public	D5	5. Very Likely	D5	unavailable due to no water supply
	Fire System	Fire Pump				5. Very Likely	A. No Release	D. Single Fatality / Impact on Public	D5	5. Very Likely	D5	unavailable due to no water supply
	Fire System	Hydrants				5. Very Likely	A. No Release	D. Single Fatality / Impact on Public	D5	5. Very Likely	D5	unavailable due to no water supply

Assets Not Exempt by DOT / PMHSA Jurisdiction

						Risk Assessment Rankings						
							Severity					
Location	Item Type	Identification	Contents	Out of Service	DOT/PHMSA?	Likelihood WITHOUT Soil Displacements	Environmental	Safety	Risk Score	Likelihood WITH Soil Displacements	Risk Score	Item or Score Notes
Tank Farm 1												
	Tank	2113	Biodiesel			3. Possible	B. Release within secondary containment	B. Injury With Medical Treatment	B3	4.Likely	B4	
	Tank	3510	Ethanol			3. Possible	B. Release within secondary containment	C. Serious Injury / Limited Impact on Public	C3	3.Possible	C3	Higher flammability / volatility
	Tank	3605	Biodiesel			3. Possible	B. Release within secondary containment	B. Injury With Medical Treatment	B3	4.Likely	B4	
	Tank	1315	N/A	Yes		2. Unlikely	A. No Release	A. Minor / First Aid Injury	A2	3.Possible	A3	Out of service
	Tank	1316	N/A	Yes		2. Unlikely	A. No Release	A. Minor / First Aid Injury	A2	3.Possible	A3	Out of service
	Tank	1104	N/A	Yes		2. Unlikely	A. No Release	A. Minor / First Aid Injury	A2	3.Possible	A3	Out of service
	Tank	17	Vapor Tank	Yes		2. Unlikely	A. No Release	A. Minor / First Aid Injury	A2	4.Likely	A4	
		Total										
	Piping	Flammable Fuels				3. Possible	B. Release within secondary containment	C. Serious Injury / Limited Impact on Public	C3	4. Likely	C4	Higher flammability / volatility
	Piping	Non-flammable fuels				3. Possible	B. Release within secondary containment	B. Injury With Medical Treatment	B3	4. Likely	B4	
Tank Farm 3 & 4 (shared containment)												
Tank Farm 4	Tank	1011	Ethanol			2. Unlikely	B. Release within secondary containment	C. Serious Injury / Limited Impact on Public	C2	4.Likely	C4	Higher flammability / volatility
	Tank	3201	Ethanol			2. Unlikely	B. Release within secondary containment	C. Serious Injury / Limited Impact on Public	C2	4.Likely	C4	Higher flammability / volatility
		Total										
	Piping	Flammable Fuels				3. Possible	B. Release within secondary containment	C. Serious Injury / Limited Impact on Public	C3	4. Likely	C4	Higher flammability / volatility
	Piping	Non-flammable fuels				3. Possible	B. Release within secondary containment	B. Injury With Medical Treatment	B3	4. Likely	B4	
Other Tanks/Liquids (Outside Main Yards)												
	Tank	181	Gasoline Additives			2. Unlikely	B. Release within secondary containment	B. Injury With Medical Treatment	B2	3. Possible	B3	
	Tank	23	Gasoline Additives	Yes		2. Unlikely	A. No Release	A. Minor / First Aid Injury	A2	3. Possible	A3	Out of service
	Tank	4	0	Yes		2. Unlikely	A. No Release	A. Minor / First Aid Injury	A2	4. Likely	A4	Out of service
	Tank	212	0	Yes		2. Unlikely	A. No Release	A. Minor / First Aid Injury	A2	4. Likely	A4	Out of service
	Tank	1015	N/A	Yes		2. Unlikely	A. No Release	A. Minor / First Aid Injury	A2	4. Likely	A4	Out of service
	Tank	1016	N/A	Yes		2. Unlikely	A. No Release	A. Minor / First Aid Injury	A2	4. Likely	A4	Out of service
	Oil Water Separator	01 OWS	Water			2. Unlikely	C. Exceeds secondary containment, but no	B. Injury With Medical Treatment	C2	4. Likely	C4	
	Drums	Drummed Waste Storage				3. Possible	B. Release within secondary containment	A. Minor / First Aid Injury	B3	4. Likely	B4	South of Yard #2
Truck Loading Rack												
	Tank (Underground)	EMS Overfill Tank				2. Unlikely	C. Exceeds secondary containment, but no	A. Minor / First Aid Injury	C2	3. Possible	C3	
	Loading Rack Structure	Truck Loading Racks				2. Unlikely	B. Release within secondary containment	A. Minor / First Aid Injury	B2	3. Possible	B3	

Assets Not Exempt by DOT / PMHSA Jurisdiction (continued)

						Risk Assessment Rankings						
							Severity					
Location	Item Type	Identification	Contents	Out of Service	DOT/PHMSA?	Likelihood WITHOUT Soil Displacements	Environmental	Safety	Risk Score	Likelihood WITH Soil Displacements	Risk Score	Item or Score Notes
P-2 Dock (North Wharf)												
	Marine Structure	P-2 Wharf				3. Possible	E. Major offsite release	B. Injury With Medical Treatment	E3	4. Likely	E4	
	Building	Dock Office				2. Unlikely	A. No Release	B. Injury With Medical Treatment	B2	4. Likely	B4	
	Piping	Flammable Fuels				3. Possible	E. Major offsite release	C. Serious Injury / Limited Impact on Public	E3	4. Likely	E4	Higher flammability / volatility
	Piping	Non-flammable fuels				3. Possible	E. Major offsite release	B. Injury With Medical Treatment	E3	4. Likely	E4	
P-3 Dock (South Wharf)												
	Marine Structure	P-3 Wharf				3. Possible	E. Major offsite release	B. Injury With Medical Treatment	E3	4. Likely	E4	
	Building	Dock Office				2. Unlikely	A. No Release	B. Injury With Medical Treatment	B2	4. Likely	B4	
	Piping	Flammable Fuels				3. Possible	E. Major offsite release	C. Serious Injury / Limited Impact on Public	E3	4. Likely	E4	Higher flammability / volatility
	Piping	Non-flammable fuels				3. Possible	E. Major offsite release	B. Injury With Medical Treatment	E3	4. Likely	E4	
Buildings												
	Building	#4 Operations				3. Possible	A. No Release	A. Minor / First Aid Injury	A3	3. Possible	A3	Conc masonry
	Building	#5 Maintenance Shop				3. Possible	A. No Release	A. Minor / First Aid Injury	A3	4. Likely	A4	Near P-2
	Building	#8 Garage				3. Possible	A. No Release	A. Minor / First Aid Injury	A3	4. Likely	A4	Conc masonry
	Building	#10 Foam House				2. Unlikely	A. No Release	C. Serious Injury / Limited Impact on Public	C2	3. Possible	C3	Fire Foam for TLR
	Building	#11				2. Unlikely	A. No Release	A. Minor / First Aid Injury	A2	3. Possible	A3	Next to #10
	Building	#12 Storage				3. Possible	A. No Release	A. Minor / First Aid Injury	A3	4. Likely	A4	Near P-2
	Building	Main Office				2. Unlikely	A. No Release	A. Minor / First Aid Injury	A2	3. Possible	A3	
	Building	#7				2. Unlikely	A. No Release	A. Minor / First Aid Injury	A2	3. Possible	A3	
	Building	#1				2. Unlikely	A. No Release	A. Minor / First Aid Injury	A2	4. Likely	A4	Btwn Yard 4&5; old office
	Building	#13 Foam House				2. Unlikely	A. No Release	C. Serious Injury / Limited Impact on Public	C2	4. Likely	C4	Fire Foam for Yards 1,2,3,5
	Building	#15				3. Possible	A. No Release	A. Minor / First Aid Injury	A3	3. Possible	A3	Conc masonry
	Building	#16				3. Possible	A. No Release	A. Minor / First Aid Injury	A3	3. Possible	A3	Conc masonry
	Building	#17				2. Unlikely	A. No Release	A. Minor / First Aid Injury	A2	4. Likely	A4	East Yard #1
Overall Terminal												
	Emergency Response	Operator Staffing				1. Very Unlikely	A. No Release	A. Minor / First Aid Injury	A1	1. Very Unlikely	A1	
	Emergency Response	Communications				1. Very Unlikely	A. No Release	A. Minor / First Aid Injury	A1	1. Very Unlikely	A1	Backup radios; backup lighting
	Power	Transformers				2. Unlikely	A. No Release	B. Injury With Medical Treatment	B2	3. Possible	B3	no backup power for main equipment
	Power	Steam Generator				2. Unlikely	A. No Release	A. Minor / First Aid Injury	A2	2. Unlikely	A2	Main office
	Fire System	Water Main				5. Very Likely	A. No Release	D. Single Fatality / Impact on Public	D5	5. Very Likely	D5	no backup water sources
	Fire System	Foam System				5. Very Likely	A. No Release	D. Single Fatality / Impact on Public	D5	5. Very Likely	D5	unavailable due to no water supply
	Fire System	Fire Pump				5. Very Likely	A. No Release	D. Single Fatality / Impact on Public	D5	5. Very Likely	D5	unavailable due to no water supply
	Fire System	Hydrants				5. Very Likely	A. No Release	D. Single Fatality / Impact on Public	D5	5. Very Likely	D5	unavailable due to no water supply