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Seismic Vulnerability Assessment
Phase 1

Phillips 66 Terminal
Portland, Oregon

for

Phillips 66 Pipeline LLC

May 30, 2024

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GeoEngineers

Seismic Vulnerability Assessment Phase 1

Phillips 66 Terminal
Portland, Oregon

File No. 20545-035-02
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1.0 Introduction

This report summarizes the preliminary results of the seismic vulnerability assessments (Phase 1) performed to date for the Phillips 66 Terminal and Lubricants facility located at 5528 NW Doane Avenue in Portland, Oregon (herein referred to as the "Facility" or "Site"). The seismic vulnerability assessments were requested by Phillips 66 Pipeline LLC (Phillips 66) in response to the 81st Oregon Legislative Assembly passed Senate Bill 1567 that requires owners or operators of large capacity fuel terminals located in Columbia, Multnomah, or Lane Counties to conduct and submit to the Oregon Department of Environmental Quality (DEQ) seismic vulnerability (a.k.a., risk) assessments by June 1, 2024.

The Facility's assessment team includes GeoEngineers, Inc. (GeoEngineers), Reid Middleton, Inc. (RM) and Alpha Technology Group, Inc. (ATG). GeoEngineers is leading the risk assessment and providing environmental engineering, geotechnical engineering and seismic risk and hazard analyses under the design earthquake with corresponding ground-shaking. RM has been tasked with performing structural analyses for the tanks, buildings and concrete containment walls, while ATG is performing structural and operational analyses of the Facility's dock, control equipment, piping, loading racks and safety systems.

The Facility is located in the northwest portion of the City of Portland on the west bank of the Willamette River at river mile marker 7.8 and southeast of the St. John's Bridge. The terminal consists of a bulk storage and distribution terminal for finished petroleum products and lubricant oils. The lubricants plant also performs blending and packaging operations at the Site. The Site is shown relative to surrounding physical features in Figure 1, Vicinity Map.

This report was prepared as the initial assessment report following the roadmap developed by the Oregon DEQ for facilities to use to develop seismic vulnerability assessments to comply with the Oregon DEQ's Fuel Tank Seismic Stability (FTSS) Program rules per Oregon Administrative Rules Chapter 340 Division 300 (OAR 340-300). The rules state that their purpose is to protect public health, life safety and environmental safety against release of fuel products and fires. The FTSS Program rules were adopted on September 14, 2023, and the roadmap with corresponding checklists was issued by DEQ in mid-March 2024. **Since the rules and corresponding guidance documents are relatively new, we opted for a phased seismic vulnerability study after finding the ground deformations predicted for the site using simplified procedures were too inaccurate to effectively evaluate the structural integrity of the individual infrastructure components. The simplified procedures for estimating earthquake induced ground deformations are known to over-predict large strain deformations. Thus, more refined analyses will be required to establish realistic ground deformations, which can then better support the seismic vulnerability assessment of the individual site components.**

The purpose of this report is to provide the preliminary results of the seismic vulnerability assessment performed to date for the Facility and include a summary of remaining work to be completed and the corresponding proposed schedule for the next phase.

1.1 ASSESSMENT PERFORMANCE CRITERIA

OAR 340-300-0001 defines the performance objective for the rules and includes a limiting performance level and a definition for Maximum Allowable Uncontained Spill (MAUS). It is understood that the intent of the rules is to assess the potential for a spill greater than the MAUS emanating from individual terminal components. Components must be evaluated using the seismic ground motions consistent with the ASCE 7 Design Level Earthquake. In addition, spills that are adequately contained do not count towards the "uncontained" spill. The MAUS is defined as a volume of petroleum product equal to one barrel.

2.0 General Site Information

The Phillips 66 Portland Terminal generally consists of product storage tanks, product transfer systems, a marine dock, a lubricant blending and packaging facility and related maintenance facilities. The Site's petroleum product storage consists of 116 aboveground storage tanks (ASTs), including 85 lube oil ASTs, 17 gasoline and diesel ASTs and 14 ASTs that are cleaned and currently out of service. These tanks are located in six tank farms; three of which consist primarily of light oil products and additive tanks (Tank Farm No. 1, No. 2 and No. 3), and three which are lube oil tank farms (Upper and Lower Lube Cells and F-Row Tank Farm). All ASTs are constructed of welded and/or riveted steel (mostly dependent on age). The Facility also has two registered underground storage tanks (USTs) associated with the lubricants plant. Several other USTs were removed from the Site in 1997. A list of the Site's ASTs, USTs and their contents is presented in the SPCC Plan. The facility layout is depicted in Figure 2, Site Map.

2.1 SURFACE CONDITIONS

The ground surface across the terminal is primarily covered by asphalt and/or gravel surfacing. The existing site grades are relatively consistent at an approximate elevation of +35 to +40 feet in the North American Vertical Datum of 1988 (NAVD88).

2.2 SUBSURFACE CONDITIONS

Our understanding of the subsurface conditions across the terminal is based on our review of the available geotechnical information (e.g., existing subsurface explorations, etc.) within the Site vicinity and three cone penetration tests (CPTs) that we completed as part of this first phase of the seismic vulnerability study. The site soils underlying the existing site grade generally consist of fill, alluvium and Columbia River Basalt (CRB).

- **Fill** (Engineering Soil Unit [**ESU-1**]) was observed below the existing site grade across the terminal and extended to approximately 5 feet below ground surface (bgs). It generally consists of sand with varying amounts of fines content (silt and clay).
- **Alluvium (ESU-2)** was observed below the fill across the terminal and extended to approximately 42 to 79 feet bgs. The alluvial layer was delineated into three units:
  - **Upper Alluvium (sand and silt):** Upper Alluvium consists mostly of sand and silt and was observed at relatively shallower depths within the alluvium to approximately 14 to 29 feet bgs. The thickness of Upper Alluvium ranges from 9 to 24 feet.
  - **Intermediate Alluvium (silt and clay):** Intermediate Alluvium consisting mostly of silt and clay extends to approximately 35 to 55 feet bgs. The thickness of Intermediate Alluvium ranges from 14 to 28 feet.
  - **Lower Alluvium (silt and sand):** Lower Alluvium consists mostly of silt and sand and was observed to approximately 42 to 79 feet bgs. The thickness of Lower Alluvium ranges from 5 to 25 feet.
- **CRB (ESU-3)** was estimated to be at the depth where refusal was encountered in CPTs, based on our previous experience from the nearby projects and published geologic information.

The site plan shown in Figure 3 includes the approximate locations of the explorations completed for this phase of the study and previously completed subsurface explorations that we reviewed. The logs of the current explorations are included in Appendix A.1. The logs from the historical explorations are presented in Appendix A.2.

2.3 GROUNDWATER CONDITIONS

Based on the pore pressure dissipation data reported from the existing subsurface explorations and previous subsurface explorations report, the groundwater table is estimated to be on the order of 9 to 15 feet bgs. Groundwater conditions will vary as a function of season, precipitation and other factors. Considering that the terminal is on the west bank of the Willamette River, the groundwater table will be heavily influenced by the water level in the Willamette River.

2.4 SITE-SPECIFIC SEISMIC CRITERIA

The site is a seismic Site Class F per American Society of Civil Engineers (ASCE) 7-16 Section 20.3 due to the presence of potentially liquefiable soils on site (as discussed in more detail in Section 3.1 and Appendix A, Section A.4); therefore, site response analysis is required to determine the seismic design parameters for this site, which are presented in Section 3.1 and Appendix A.

3.0 Seismic Vulnerability Assessment Checklists

The Oregon DEQ's roadmap includes nine **(9)** checklist forms that provide detailed guidance for the seismic vulnerability assessments on various components, that include: geotechnical assessment, tanks, pipelines, piers and wharves, liquefied natural gas (LNG) tanks and pipelines, berms and dikes, building and building structures, fire detection and suppression and control systems. This section summarizes the preliminary results of seismic vulnerability assessments completed to date for various components following the nine checklist forms.

3.1 GEOTECHNICAL ASSESSMENT

The general site conditions including surface, subsurface and groundwater conditions were summarized in Section 2. One representative cross section was developed across the terminal as shown in Figure 4 for use in the geotechnical assessment. The location of the cross section is shown in Figure 3. Our interpretation of the subsurface conditions are also depicted in Figure 4. Considering the water level in the Willamette River, a design groundwater table was assumed at 9 feet bgs.

3.1.1 Seismic Design Parameters

A site-specific ground motion analysis was completed to develop the site-specific risk-targeted maximum-considered earthquake (MCRR) horizontal response spectrum. Table 1 and Figure 5 present our recommended site-specific MCER response spectrum. Please refer to Appendix A, Section A.4, for additional details regarding the development of the recommended site-specific MCER response spectrum.

TABLE 1. RECOMMENDED SITE-SPECIFIC MCE_R RESPONSE SPECTRUM

PERIOD (SEC)	5% DAMPED SPECTRAL ACCELERATION (G)
0.010	0.634
0.020	0.674
0.030	0.699
0.050	0.732
0.075	0.886
0.100	1.015
0.150	1.227
0.200	1.405
0.250	1.467
0.300	1.520
0.400	1.501
0.500	1.423
0.750	1.148
1.000	0.939
1.500	0.623
2.000	0.465
3.000	0.281
4.000	0.203
5.000	0.163
7.500	0.108
10.000	0.081

3.1.2 Liquefaction Potential

Liquefaction is a phenomenon where soils experience a rapid loss of internal strength as a consequence of strong ground shaking. Ground settlement, lateral spreading and sand boils may result from liquefaction. In general, the soil that is susceptible to liquefaction includes very loose to medium dense, clean to silty sands and some silts that are below the groundwater level.

The structures supported on liquefied soils could suffer foundation settlement, downdrag loads, or lateral movement that could be severely damaging to the structures. The evaluation of liquefaction potential is complex and dependent on numerous parameters, including soil type, grain size distribution, soil density, depth to groundwater, in-situ static ground stresses, earthquake-induced ground stresses and excess pore water pressure generated during seismic shaking.

The evaluation of liquefaction potential is a complex procedure and depends on numerous site parameters. Typically, the liquefaction potential of a site is evaluated by comparing the cyclic stress ratio (CSR), which is the ratio of the cyclic shear stress induced by an earthquake to the initial effective overburden stress, to the cyclic resistance ratio (CRR), which is the soils resistance to liquefaction.

Our liquefaction potential evaluation was performed under a maximum-considered earthquake (MCE) event with a 2 percent probability of exceedance in 50 years (2,475-year return period) per ASCE 7-16. Liquefaction analysis of the CPTs were completed using the analytical software CLiq (GeoLogismiki 2018) and the semi-empirical approach by Boulanger and Idriss (2014). We completed these analyses using the MCER peak ground acceleration (PGA) of 0.634g and crustal earthquake magnitude of 6.9 derived from site-specific MCE_R horizontal response spectra, and a second analysis with a Cascadia Subduction Zone (CSZ) earthquake deterministic 84^{th} percentile PGA value of 0.62g and a magnitude of 9.0. Ground settlement due to liquefaction-induced settlement was estimated using empirical procedures based on Boulanger and Idriss (2014) correlations for CPTs. Our analyses indicate that the site under existing condition could experience a liquefaction-induced free-field ground settlement on the order of 5 to 14 inches.

3.1.3 Lateral Spreading

Lateral spreading involves lateral displacements of large volumes of liquefied soil. Lateral spreading can occur on near-level ground as blocks of surface soil are displaced relative to adjacent blocks. It also occurs as blocks of surface soils are displaced towards a nearby slope or free face, such as the bank of the Willamette River, by movement of underlying liquefied soil. In the case of this project site, lateral spreading could occur during and/or after earthquakes resulting in excessive movement of terminal facilities.

Figure 6 presents the slope stability analyses results for post-earthquake and during seismic conditions. In the post-seismic condition, the factor of safety (FOS) was estimated less than 1.05 for the slip surfaces going through the existing tank farm, which indicates a flow failure and an unstable condition. As shown in Figure 6, in the seismic condition, the yield acceleration was estimated as 0.027g within 1,000 feet from the free face. The yield acceleration is the horizontal seismic coefficient that results in a factor of safety of 1.0 computed from limit equilibrium slope stability analysis. The corresponding earthquake-induced lateral ground deformation was estimated to be greater than 2 feet using the simplified displacement approach developed by Bray and Travasarou (2007) and Bray et al. (2018).

If they were to occur, flow failures and large ground deformations would likely damage most infrastructure components subject to such deformations. However, we do not believe these deformations are realistic. The simplified procedures for estimating lateral deformation, such as conventional slope stability analysis and Newmark sliding block analyses (see Appendix A, Section A.5) breakdown and are typically not accurate when large strains are predicted. These simplified methods provide reliable results when confirming an issue does not exist or the estimated deformations are relatively small. However, these simple models do not capture the softening and strain-hardening effects that occur as liquified soils move and excess pore water pressure is redistributed. Nor do the simple models capture the significant damping that occurs as the soil displaces.

To better understand the likely ground deformations, numerical modeling using constitutive soil models based on detailed laboratory testing to determine the model input parameters will be required.

3.2 TANKS

3.2.1 Tank Characteristics

The Facility has over a hundred fixed steel ASTs that contain different grades of gasoline, No. 2 diesel, biodiesel, lube oil and lube blend/base stocks, additives and ethanol. Tank Farm 1, located adjacent to NW Doane Avenue between the Refined Products Loading Rack and the Lube Blending and Packaging Warehouse, includes 15 ASTs with shell volume (capacity) ranging between 464 and 39,644 barrels (19,488 and 1,665,048 gallons). Tank Farm 2, located at the corner of NW Doane Avenue and NW Front Street, includes 12 ASTs with shell volumes ranging between 5,483 and 84,600 barrels (230,286 and 3,553,200 gallons). Tank Farm 3, located at the southeast portion of the Facility, includes 27 ASTs with shell volume ranging between 243 and 80,571 barrels (10,206 and 3,383,982 gallons). The Upper and Lower Lube Cells (Tank Farms 4 and 5, respectively) are adjoining and located next to the southwest corner of the Lube Blending and Packaging Warehouse. Tank Farm 4 includes eighteen 20,370-gallon ASTs and Tank Farm 5 includes 23 ASTs with shell volumes ranging between 289 and 699 barrels (12,138 and 29,358 gallons). Although not identified on Figure 2, there are 14 fixed ASTs inside the Lube Blending and Packaging Warehouse with shell volumes ranging between 23 and 157 barrels (966 and 6,594 gallons). The F-Row (Tank Farm 6) is located between Tank Farm 3 and the Upper and Lower Lube Cells and includes eight (8) ASTs with shell volumes ranging between 162 and 485 barrels (6,804 and 20,370 gallons).

In addition to the fixed ASTs, there are two 5,000-gallon metal single wall, cathodically protected, registered USTs below the Lubricants testing lab. Mobile or portable soil storage containers, 55-gallon drums and 275 gallon totes (poly and stainless steel) are located throughout the Facility. Most of these mobile containers are stored within the Lube Blending and Packaging Warehouse. A complete list of Facility's tanks and their contents is presented in the terminal's SPCC Plan.

At this time, a comprehensive catalog of the Site's tank foundations and the seismic-relevant tank properties has yet to be prepared. This Facility has existed in some capacity for over one hundred years. Many of the facility's oldest tanks are currently out of service. The facility has many tanks that were built throughout the 20th century. Many of the facility's older, larger, field-constructed AST's do not have anchors connecting the tank to a concrete foundation and are, therefore, classified as unanchored. While some of the newer (i.e., post-1970s) and smaller ASTs (e.g., less than 50,000 gallons) do appear to have anchors connecting the tank to a concrete foundation (i.e., anchored). Some of the tanks located in the lube cell areas are steel tanks that are slightly elevated on steel leg pedestals with anchors.

3.2.2 Tank Risk Categories Per OSSC, IBC and ASCE 7

Every structure designed in accordance with the Oregon Structural Specialty Code (OSSC), International Building Code (IBC) and ASCE 7 must be assigned a Risk Category. These code documents describe risk categories in relation to the risk to human life, health and welfare that would be caused by a structure's damage or failure. Therefore, the nature of a structure's use dictates its Risk Category. These code documents have four risk category levels defined as Risk Category I, Risk Category II, Risk Category III and Risk Category IV. Higher risk categories reflect structures with more relative risk. For example, Risk Category IV structures include "essential facilities" like hospitals and emergency shelters, whereas Risk Category I structures include generally unoccupied structures like agriculture barns or minor storage facilities. Most typical structures (such as office buildings, apartment buildings, homes, restaurants, retail stores, etc.) are assigned to Risk Category II.

For the seismic design and analysis of structures, the Risk Category dictates the Seismic Importance Factor used for a structure. Risk Category I and II structures are assigned a Seismic Importance Factor of 1.0, Risk Category III structures are assigned a Seismic Importance Factor of 1.25 and Risk Category IV structures are assigned a Seismic Importance Factor of 1.5. In the determination of design seismic forces on a structure, the Seismic Importance Factor has the effect of causing structures with a higher importance factor to be designed for larger forces.

For tanks that store petroleum products and other associated products, the primary determinant of the assigned risk category is whether the product stored in the tank is defined as “toxic”, “highly toxic” or “explosive”. The OSSC and IBC have specific definitions for “toxic”, “highly toxic” and “explosive”. “Toxic” substances are defined as chemicals with a median lethal dose (LD_{50}) or median lethal concentration (LC_{50}) above certain thresholds. The thresholds depend on whether the chemical is ingested orally, comes in contact with skin, or is inhaled through the air. “Highly toxic” substances are defined similarly to “toxic” substances but with more strict thresholds (smaller LD_{50} and LC_{50} values). “Explosive” substances are defined as chemical compounds, mixtures, or devices, the primary or common purpose of which are to function by explosion such as dynamite, black powder, pellet powder, etc. Most common petroleum products, such as gasoline and diesel fuels, are categorized as flammable or combustible but not toxic, highly toxic, or explosive.

The products stored at the Phillips 66 Portland Terminal will be evaluated for whether they are classified as toxic, highly toxic or explosive. This will determine what risk category each tank is assigned. Tanks that store typical petroleum products such as gasoline, diesel, lubricants, etc., are expected to not be classified as toxic, highly toxic, or explosive as defined by the OSSC, IBC and ASCE 7 and will be assigned to Risk Category II.

3.2.3 Tank Vulnerability Assessment

Seismic design requirements for new building and non-building structures changed significantly throughout the 20th century. In general, seismic design requirements have become more strict over time. In addition, when seismic design regulations were first adopted in Oregon, the state was identified as a moderate seismicity region. However, the current adopted building code identifies most of Western Oregon as a high seismicity region. In addition, the general seismic design force stipulated in the current adopted building code for Western Oregon (including Portland) is approximately double the seismic design force when seismic regulations were first adopted (Oregon, 2012).

As a result, almost no building or non-building structures built during the 20th century in Western Oregon comply with the current adopted seismic building code provisions for new structures. Given this, older structures are potentially more vulnerable to earthquakes than brand new structures. However, the seismic behavior of structures is complex. Just because a structure is older does not mean it should be automatically expected to fail or collapse in an earthquake. In fact, most earthquake events see the majority of structures not collapse even for vulnerable structures that experience strong shaking (FEMA, 2020).

Building code documents recognize that it is not possible to eliminate all seismic risk for structures. The intent of the building code is to reduce the probability of structural failure to levels deemed acceptable to regulators, the engineering community and the general public.

An important aspect of existing structure seismic evaluations is to identify seismic deficiencies that will have an appreciable deleterious effect on seismic performance. In addition, the OAR 340-300 performance objective is to limit the spill volume to be less than the MAUS. The Oregon Administrative Rules imply that seismic structural damage to the fuel tanks is acceptable as long as the MAUS is not expected to be exceeded.

3.2.4 Tank Vulnerability Discussion

The geotechnical engineer has identified the Site as having potentially liquefiable soil. In addition, lateral spreading is noted as being a possibility in a seismic event especially at site locations nearest the bank of the Willamette River. Please see Section 3.1 for additional information. In a seismic event, these phenomena have the potential to cause the soil to move laterally and for the site to experience uniform and differential vertical soil settlement. For the Site's steel storage tanks, the magnitude of uniform and differential vertical soil settlement, as well as the magnitude of the soil's lateral movement will have a significant effect on their seismic performance.

Most structures are not designed to accommodate large ground deformations. At this time, only high-level empirical formulations have been used to estimate the ground deformation in a seismic event. Additional geotechnical engineering work, including the collection of additional soil samples, is required to more-precisely estimate the expected ground deformations in a seismic event. Accordingly, a detailed assessment of the ability of the site's tanks to accommodate ground deformations has not been conducted at this time. Once the geotechnical engineer conducts additional engineering work for the ground deformations, the ability of the tanks to accommodate the ground deformation will be evaluated in more detail.

In addition, one significant thing that has changed over the years in the way steel storage tanks are designed is the anchorage requirements to prevent uplift due to seismic overturning forces. Tanks with low seismic overturning forces will have a low potential for uplift and may be permitted to be unanchored. However, tanks with high seismic overturning forces are required to be anchored to prevent tank uplift. The requirements for the conditions under which tanks must be anchored have become more strict over the years.

However, just because a tank that is anticipated to have high seismic overturning forces is unanchored does not mean that it automatically has a high probability of leaking. Rather it indicates that additional analysis may be justified to investigate the probability of leaking.

An assessment of the Facility's tanks in response to a design earthquake event is pending development of more accurate and realistic estimates on the depth of liquefaction, which will help further refine the corresponding liquefaction-induced ground deformation. The Form 2 Checklist for Tanks has been initiated by RM and is included in Appendix B. Following a site-specific seismic ground motion analysis, RM will complete its analysis of the Facility's tank structures and evaluate their probability of exceeding the MAUS.

3.3 PIPELINES

The Site contains the following product transfer facilities with inter-terminal above and below piping connected to the Facility's ASTs (referred to as Terminal Product Piping Systems):

- Refined Products Loading Rack
- Lane 4 Diesel Loading Rack
- Lube Oil Loading Rack
- Railcar Loading/Unloading Rack
- Tank Truck Unloading Station
- Gasoline Additive Unloading Station
- Marine Vessel Loading/Unloading and Out of Service Tug Fueling Stations
- Offloading Stations
  - Ethanol Offloading
  - Biodiesel Offloading
  - Trans-mix Pump-Off
  - Refined Products Additive Offloading
  - Diesel Pump-Off

In addition to the Terminal Product Piping Systems, the Olympic Pipe Line Company owns and operates two 12-inch pipelines which supply the majority of gasoline and No. 2 diesel to the Facility. Similarly, two 10-inch inter-company pipelines, running between the Kinder Morgan, Chevron and Phillips 66 terminals are also used to transfer gasoline and No. 2 diesel to the Site.

The product pipes and general routing of Terminal Product Piping Systems are shown on drawing 0608-P-1100 in Appendix C. In general, the terminal has specific product and transfer facilities for gasoline, diesel, biodiesel, ethanol, transmix and lubricants. These piping systems are described in more detail in Appendix C, which also includes reference drawings and pipe stress model drawings.

Detailed seismic vulnerability analyses are not included for these components in this phase of the study. Analyses will be completed by ATG for the next phase of the study after the estimated seismic ground deformations are better understood.

3.4 DOCK

Some of the Facility's refined products (e.g., diesel, gasoline and lube oil) is received and exported by marine vessel at the terminal's dock located on the Willamette River northeast of Tank Farm No. 2 across NW Front Street. The Marine Vessel Loading/Unloading Station is located near the center of the dock. All transfers at the Marine Vessel Loading/Unloading Station are conducted using flexible cargo hoses between the vessels and the piping risers. The risers are connected to the storage tanks within the tank farms through a combination of above and below ground piping. Transfers to the terminal are conducted using pumps on board the tanker or barge whereas transfers to barges are conducted using pumps at the Facility.

The Site's dock was originally constructed in approximately 1961, and repairs were designed, permitted (by the City of Portland) and constructed in 2007 and 2012. The structure is supported by pile-supported bents at approximately 17.5 feet on center. The bents include vertical and batter piles to provide resistance to lateral loads (berthing, mooring and seismic loading). Most of the structural members are wood, and these are typically replaced with steel when new repairs are made.

The most recent inspection report was completed in 2021, and the main dock structure was graded as being in “Fair” condition. The inspection report, site plan, cross sections and drawings are presented in Appendix D.

Detailed seismic analyses are not included for dock structural elements in this phase of the study. Structural analyses will be completed by ATG for the next phase of the study after the estimated seismic ground deformations are better understood.

3.5 LIQUFIED NATURAL GAS TANKS AND PIPELINES

The facility does not contain liquefied natural gas components; thus, this checklist is not included in this phased report.

3.6 BERMS AND DIKES

The Facility is separated into six outdoor containment areas. These containment areas are Tank Farm 1, Tank Farm 2, Tank Farm 3, Tank Farm 4 (Upper Lube Cell), Tank Farm 5 (Lower Lube Cell) and Tank Farm 6 (F-Row).

Secondary containment for the tank farms and lube cells is provided by concrete blocks, cast-in-place concrete walls or asphalt covered berms. A portion of Tank Farm 3 is surrounded by an earthen berm. The tops of these containment walls vary in height above finished grade in the range of 2 to 12 feet. At this time, as-built record drawings of the existing walls have not been able to be viewed. The precise geometry of the concrete walls and the amount and spacing of internal reinforcement is not known. The configurations of the wall foundations are also not known. It is understood that these walls are many decades old and were likely not designed considering the current design seismic loads. Additional investigation of the existing walls is necessary, including the determination of the geometry of the existing walls, determination of foundation configuration and determination of internal reinforcing amount and spacing. The floor/base in Tank Farms 1, 2 and 3 consists primarily of silty sand fill overlain by gravel in most areas, whereas the Upper and Lower Lube Cells and F-Row are equipped with concrete floors. The containment capacity of each tank farm is designed to contain the maximum capacity of the largest storage tank plus the rainfall from a 25- year, 24-hour storm event and 20-minutes of fire protection water. The actual containment volumes and calculations for each tank farm are provided in the Facility’s SPCC plan.

At this time, it is understood that the site’s liquefaction potential has only been evaluated by the geotechnical engineer using high-level empirical formulations. Additional engineering work must be done to more precisely understand the magnitude of total and differential settlement and lateral deformation caused by seismic soil liquefaction. Once the precise magnitude of the site’s total and differential seismic settlement and lateral deformation is understood, the concrete and earthen secondary containment walls can be evaluated for their ability to accommodate the settlement and lateral deformation. The Form 6 Checklist for Berms and Dikes Surrounding Tank Farms has been initiated by RM and is included in Appendix E. Following a site-specific seismic ground motion analysis, RM will complete its analysis of the secondary containment systems.

3.7 BUILDING AND BUILDING STRUCTURES

The Facility includes the following buildings and structures :

- Office/Lube Blending and Packaging Warehouse;
- Lubricants Testing Lab;
- Boiler House;
- Warehouse Annex;
- Office Trailers;
- Maintenance Shop/Garage; and
- Dock Warehouse.

The lube oil blending/packaging area and warehouse is situated on the first floor of the Main Office/Warehouse Building as shown in Figure 2. The east end of the warehouse contains a blending facility with a multitude of piping and pumps used to blend various lube oil base stocks and additives to produce finished lube products. The Warehouse Annex is located at the southern end of the Site. Finished lube oil products or additives in drums or totes are stored in this building. The Lubricants Testing Lab is located behind the main Warehouse in an elevated structure. Various solvents and lubricant samples are present in the lab. The Boiler House contains the boiler that provides steam to the terminal and houses the product quality room for the light oil terminal. The Maintenance Shop/Garage is situated along NW Doane Avenue adjacent to Tank Farm 2 and is used to conduct maintenance work on the terminal's miscellaneous equipment. Insignificant quantities of virgin and used lubricating oils, paint, etc., are stored in the garage in association with the normal maintenance and repair operations. The Dock Office and Office Trailer are small buildings that are not used to store petroleum products or hazardous materials. The Dock Office is situated on the dock platform between the Vessel Loading/Unloading. Station. The Office Trailer is located in the main terminal area adjacent to the Lube Warehouse in the entry alley. The Dock Warehouse (a.k.a., Asphalt Warehouse) is located along NW Front Street and is adjacent to the dock as shown in Figure 2; it is used to store terminal supplies and equipment (e.g., pumps, spill response equipment, excess furniture). The Facility's Hazardous Waste Storage Area is also located in the Dock Warehouse. Drums of petroleum products or other potential contaminants are periodically stored within this warehouse.

In accordance with OAR 340-300-0003(1)(f), it is anticipated that the building assessment will evaluate the potential for a spill greater than the MAUS during or after the Design Level Earthquake. With the exception of some piping that penetrates the envelope of some of the buildings, the building superstructures generally do not support product or product storage. The storage of product is only supported by the ground floor systems of the buildings. As such, it is anticipated that the collapse potential for the buildings will be evaluated in accordance with the OAR 340-300-0003(1)(f) to evaluate whether the performance objective is met.

The Form 7 Checklist for Buildings and Building-Like Structures has been initiated by RM and is included in Appendix F. Additional engineering work must be undertaken to review existing building structural drawings, understand the building construction types and construction details, and understand the potential vulnerabilities of these buildings. Following a site-specific seismic ground motion analysis, RM will complete its risk assessment of the Facility's buildings that serve as product storage or handling function.

3.8 FIRE DETECTION AND SUPPRESSION

The main Terminal Petroleum Fire Pump/Foam System provides protection for products stored at Tank Farms 1 and 2 and the Main Truck Load Rack located on Doane Avenue. A separate Fire Water/Sprinkler System protects the warehouse that stores lubricant products. A significant update to the terminal fire protection system was designed, permitted (City of Portland) and constructed in 2008. Reference drawings for the fire protection system are presented in Appendix G.

Detailed seismic vulnerability analyses are not included for the fire protection components in this phase of the study. Analyses will be completed by ATG for the next phase of the study after the estimated seismic ground deformations are better understood.

3.9 CONTROL SYSTEMS

The terminal has many control systems that allow for automated control of various pumps, valves and related equipment. Programmable logic controllers (PLCs) are connected to various instrumentation devices throughout the terminal. These devices include level transmitters on tanks and pressure/temperature transmitters on product piping systems. The PLCs are programmed to shut off pumps and related equipment in the event of an upset condition. The control system reference drawings are presented in Appendix H.

Detailed seismic vulnerability analyses are not included for the control system components in this phase of the study. Analyses will be completed by ATG for the next phase of the study after the estimated seismic ground deformations are better understood.

4.0 Remaining Work and Proposed Schedule

This section summarizes the remaining work and the proposed schedule.

4.1 ADDITIONAL GEOTECHNICAL ASSESSMENT

For the additional geotechnical assessment, we propose additional subsurface explorations and analysis to refine our preliminary results, particularly estimated ground deformations, as presented in this report.

4.1.1 Additional Subsurface Explorations and Laboratory Testing

We propose to complete additional subsurface explorations to obtain supplemental information to refine the subsurface conditions across the site and to obtain relatively undisturbed soil samples for laboratory testing. The proposed additional subsurface explorations mainly include geophysical survey and geotechnical borings.

Geophysical survey is a non-intrusive subsurface testing to measure shear wave velocity (Vs). We propose to complete:

- **Two (2)** one-dimensional (1D) MASW soundings to capture the site-specific Vs to approximately 100 feet bgs. The results will be compared with the Vs measured through the previous seismic CPT locations and used to refine the design Vs profiles for use in our site-specific GRA.

We proposed to complete **four (4)** geotechnical borings at the site up to 100 feet bgs or practical refusal. Soil samples will be retrieved using standard penetration test (SPT) sampling technique during drilling for soil classification and laboratory testing. Relatively undisturbed samples will also be collected using Shelby tubes for use in cyclic direct simple shear tests (CyDSS), where appropriate.

We propose to complete geotechnical laboratory testing on the selected soil samples obtained from the additional borings. The laboratory testing includes moisture content, percent fines, sieve analysis and Atterberg Limit tests (plasticity characteristics). We also propose to conduct up to **six (6)** CyDSS on the selected relatively undisturbed samples to calibrate the advanced soil constitutive models for the potentially liquefiable/cyclic-softening layers that will be used to refine our site-specific GRA and numerical modeling.

4.1.2 Additional Geotechnical Engineering Analysis

Based on the results of additional subsurface explorations and laboratory testing, we will refine soil profiles, Vs profiles, and soil engineering properties for use in our geotechnical engineering analysis.

We will develop time histories for use in the nonlinear time history structural analysis for the tanks. We assume that up to two spectra will be developed according to the tank periods, which will either be amplitude scaled or spectrally matched. The spectra will either be the risk-targeted maximum-considered earthquake (MCER) or Conditional Mean Spectrum (CMS), depending on the tank structural analyses period range of interest.

One-dimensional (1D) total and effective stress site-specific GRA will be completed to assess the design response spectrum and the liquefaction potential evaluation. The advanced soil constitutive models (e.g., PM4Sand and PM4Silt) used in the effective stress GRA will be calibrated based on the results of the CyDSS.

Two-dimensional (2D) numerical modeling will be completed on **one (1)** representative cross section to evaluate and refine the earthquake-induced ground deformations (e.g., lateral spreading and vertical settlements) under a Mw 9.0 earthquake event with MCE_R ground shaking intensity.

Upon the development of more reliable ground deformation estimates, detailed analyses, consistent with methodologies described in DEQ's road map and associated checklists, will be completed for:

- Tanks
- Pipelines
- Piers and wharves (Dock)
- Berms and dikes
- Buildings and building structures.
- Fire detection and suppression systems.
- Control systems.

4.2 PROPOSED SCHEDULE

The total duration for completing the remaining work may take up to 16 months upon receiving approval from Oregon DEQ to continue with the phased approach. The proposed schedule for the subsequent phase is presented below:

- Complete additional geotechnical subsurface explorations and laboratory testing (Section 4.1.1) – 4 to 5 months upon approval.
- Complete additional geotechnical engineering analysis (Section 4.1.2) – 8 to 9 months upon approval.
- Complete structural and system assessments (Sections 4.2 through 4.9) – 13 to 14 months upon approval.
- Complete a final seismic vulnerability assessment report – 15 to 16 months upon approval.

5.0 Analysis, Assessment and Report Limitations

Seismic design in the United States is conducted based on concepts of probability of structural failure. This means that both new and existing structures are designed to target certain probabilities of damage given certain levels of earthquake shaking. In addition to uncertainties around structural performance, there is significant uncertainty about the magnitude, duration, frequency content and return period of earthquake shaking. Earthquakes pose inherent risks for structures and often cause the highest internal forces a structure will experience in its lifetime. Building code documents recognize that it is not possible to eliminate all seismic risk for structures. The intent of the building code is to reduce the probability of structural failure to levels deemed acceptable to regulators, the engineering community and the general public. Even structures designed to Risk Category IV requirements that are in compliance with building code standards still have a certain probability of failure in a large earthquake.

In addition, the professional services described in this report were performed based on limited information available at this time. No detailed investigation or destructive testing was performed to qualify as built conditions or verify the quality of materials and workmanship. No other warranty is made as to the professional advice included in this report. This report provides an overview of a preliminary seismic vulnerability assessment report. This report does not address any portions of structures, buildings, equipment, or systems other than those mentioned, nor does it provide any warranty, either expressed or implied, for any portion of the facility. This report has been prepared for the exclusive use of Phillips 66 Pipeline LLC and is not intended for use by other parties, as it may not contain sufficient information for other parties' purposes or their uses.

Please refer to Appendix I, Report Limitations and Guidelines for Use, for additional information pertaining to the use of this report.

6.0 References

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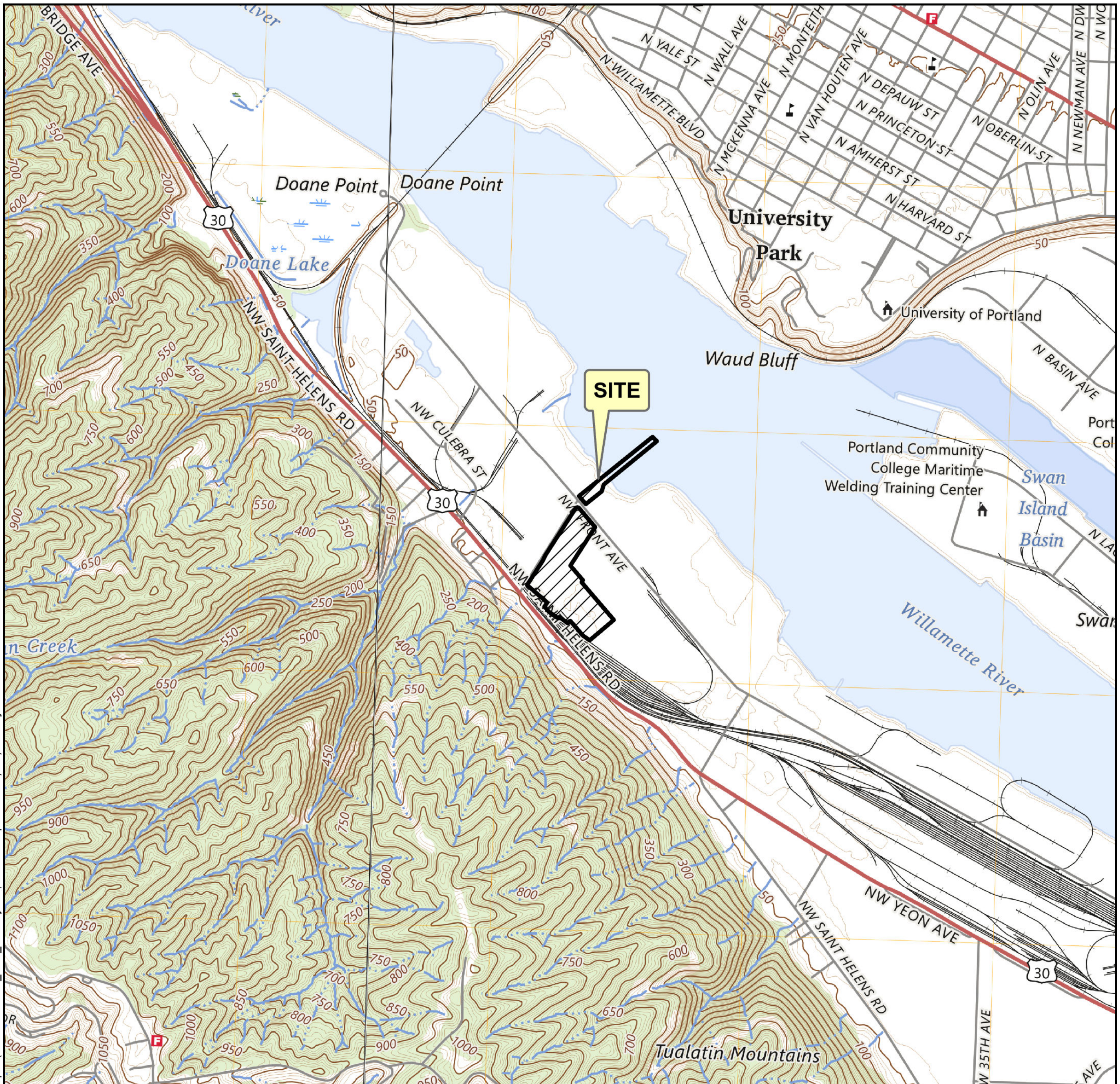
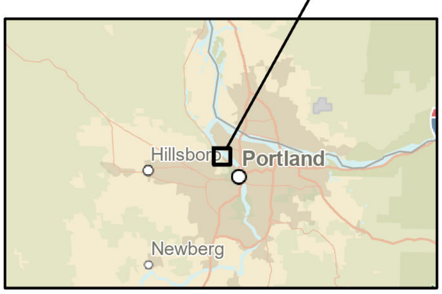
Figures





N

0 2,000

Feet

Source(s):

- ESRI
- USGS

Coordinate System: NAD 1983 StatePlane Oregon North FIPS 3601 Feet

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Vicinity Map

Phillips 66 Portland Terminal
Facility Improvement,
Portland, Oregon

GEOENGINEERS

Figure 1

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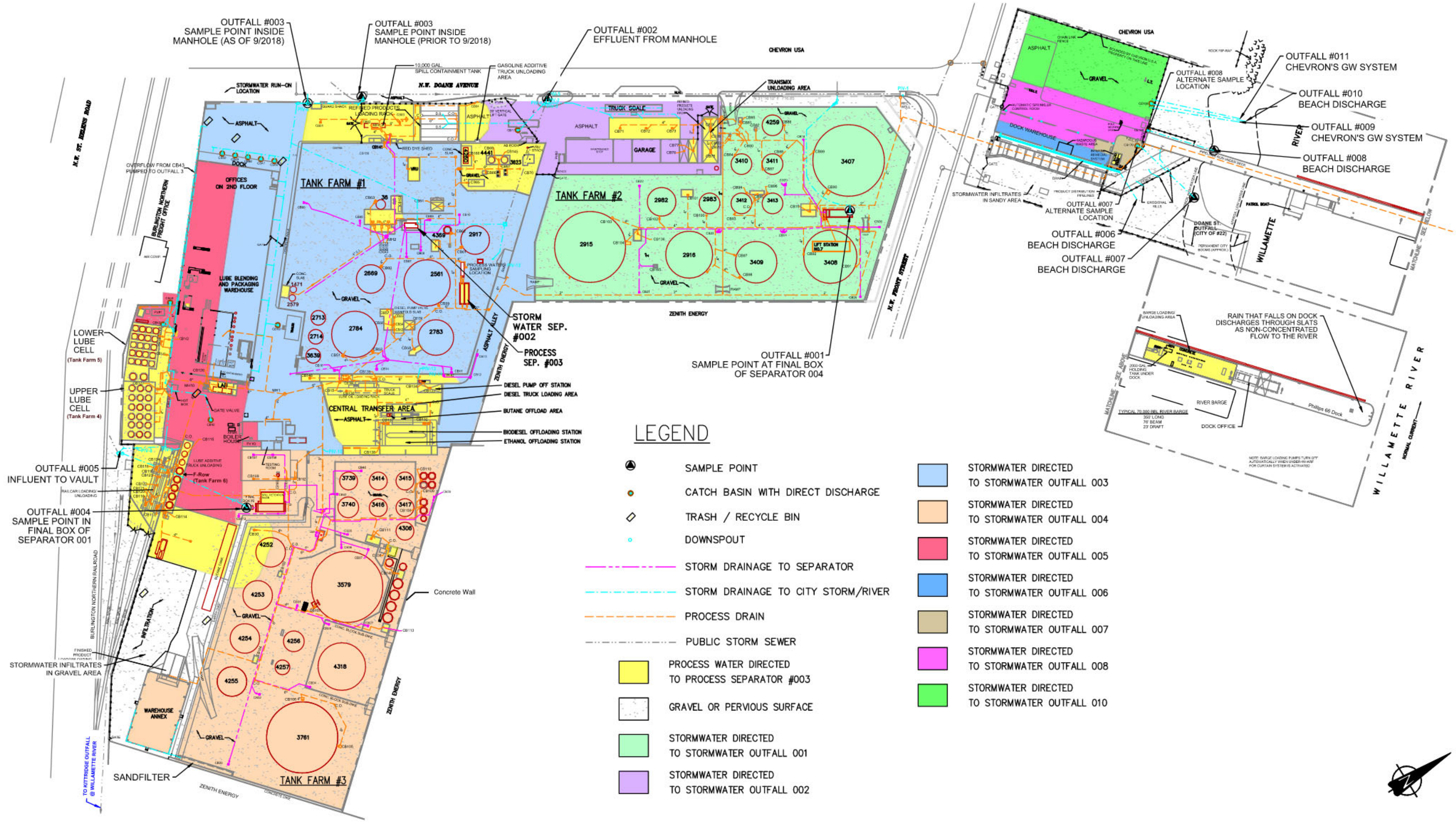

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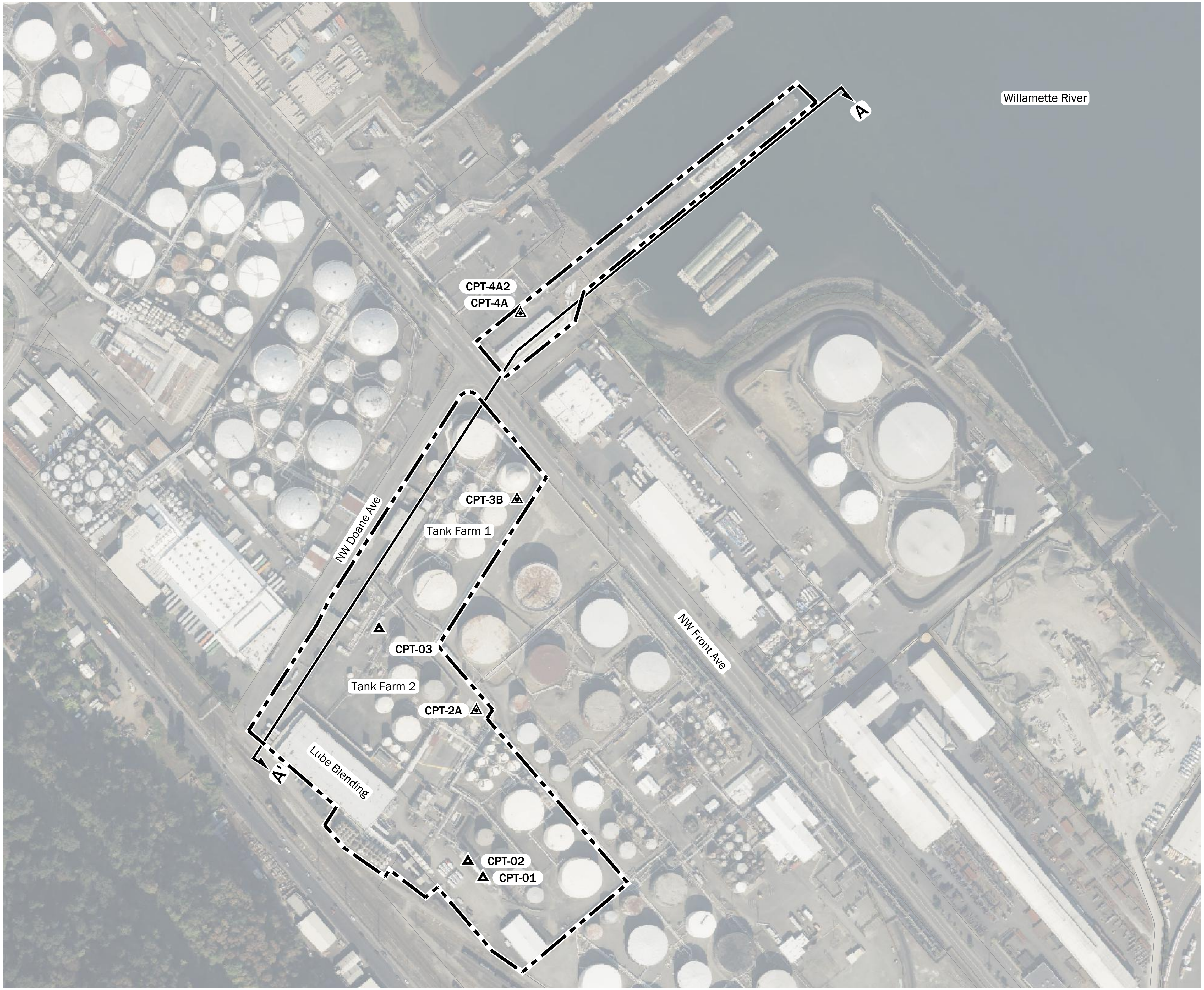
Site Map	
Philips 66 Portland Terminal Portland, Oregon	
GeoEngineers	Figure 2

20545-035-01 Date: 5/13/2024



Legend

Site Boundary

CPT-4A Cone Penetration Test by GeoEngineers, 2024

CPT-01 Cone Penetration Test by Intertek PSI, 2016

Source(s):
- Aerial from Microsoft Bing

Projection: OR State Plane, North Zone, NAD83, US Foot

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N

0 300

Feet

Site Plan

Phillips 66 Portland Terminal
Facility Improvement,
Portland, Oregon

GeoEngineers

Figure 3

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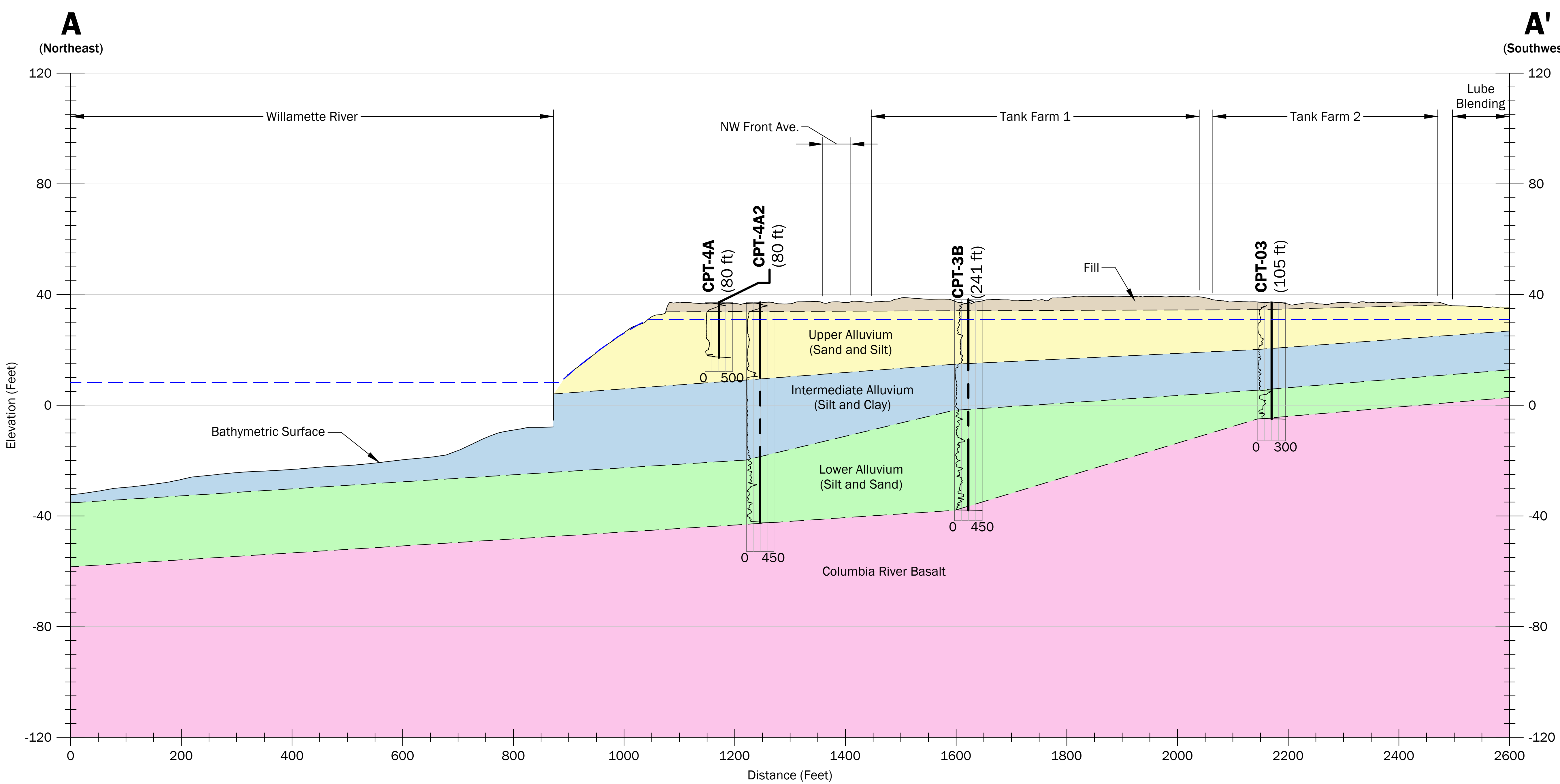

Note(s):

1. The subsurface conditions shown are based on interpolation between widely spaced explorations and should be considered approximate; actual subsurface conditions may vary from those shown.

Datum: NAVD88

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Legend

CPT ID (Offset)
Tip Resistance
0 300 in Square Foot (tsf)
Centerline

Inferred Soil Contact
Groundwater Table

Fill
Upper Alluvium (Sand and Silt)
Intermediate Alluvium (Silt and Clay)
Lower Alluvium (Silt and Sand)
Columbia River Basalt

0 200
Horizontal Feet
0 40
Vertical Feet
Vertical Exaggeration = 5X

Cross Section A-A'

Phillips 66 Portland Terminal
Facility Improvement,
Portland, Oregon

GeoEngineers

Figure 4

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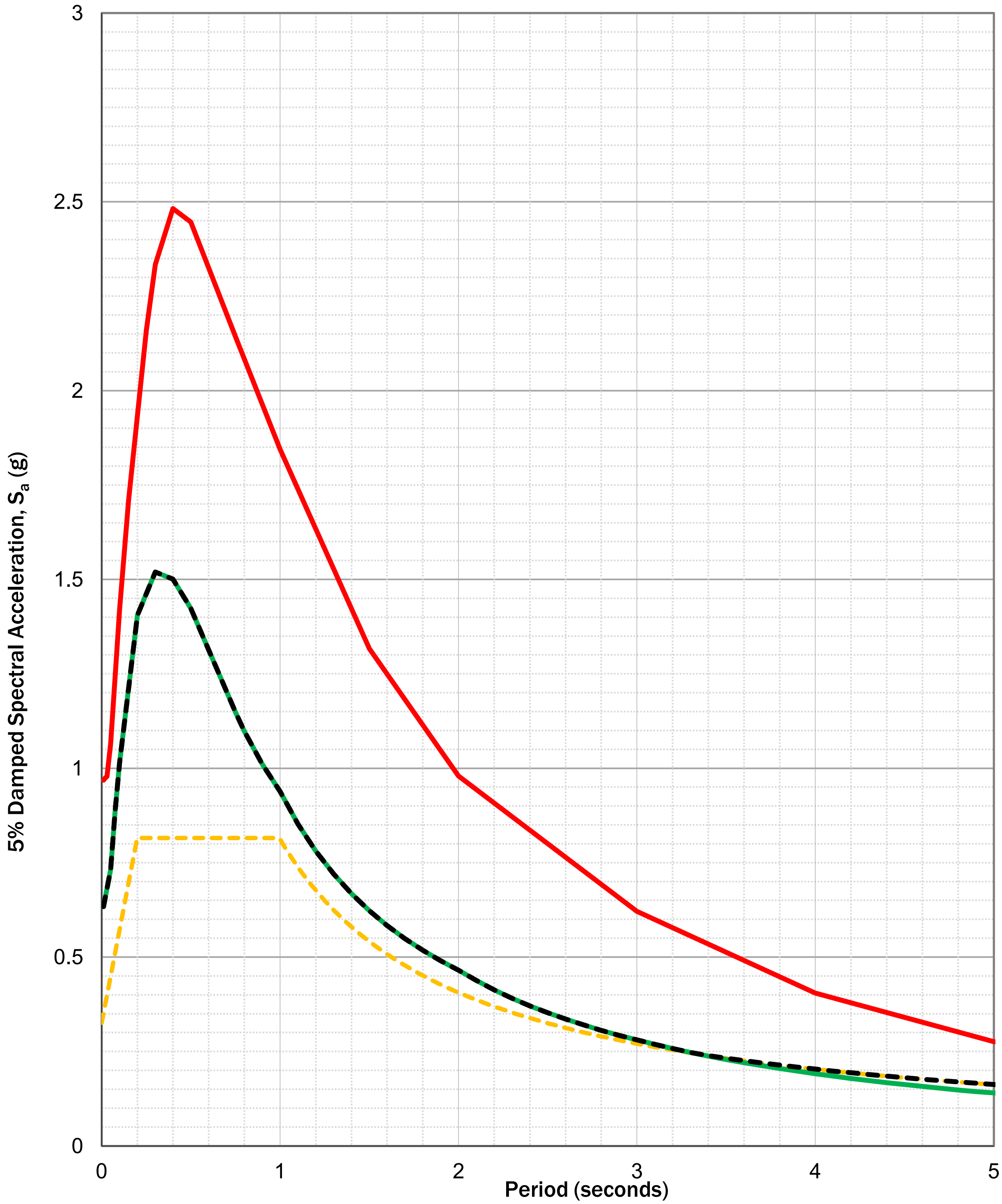


ASCE 7-16 Section 21.2.3 Site Class D Minimum

Site-Specific Deterministic (MCEr)

Site-specific Probabilistic (MCEr)

Recommended Site-Specific MCEr

Recommended Site-Specific MCE_R Response Spectrum

Phillips 66 Terminal
Portland, Oregon

GEOENGINEERS

Figure 5

25045-035-02 Date:04/02/2024

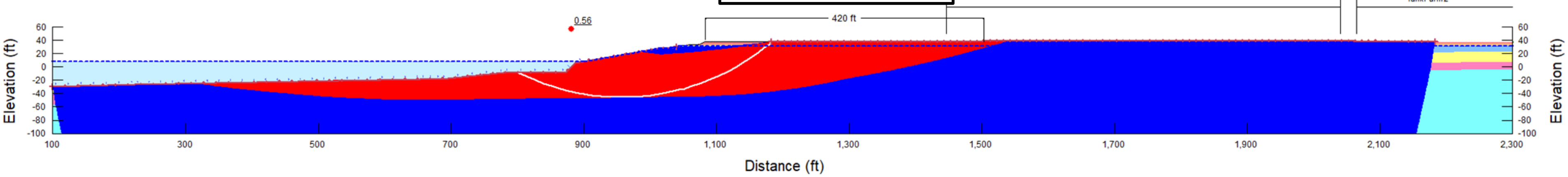
Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	ESU-1 Fill (80%)	Mohr-Coulomb	120	0	29
	ESU-2 Intermediate Alluvium (Silt and Clay) (80%)	Mohr-Coulomb	105	400	0
	ESU-2 Lower Alluvium (Silt and Sand) (Residual)	Mohr-Coulomb	115	0	6
	ESU-2 Upper Alluvium (Sand and Silt) (80%)	Mohr-Coulomb	115	0	26
	ESU-2 Upper Alluvium (Sand and Silt) (Residual)	Mohr-Coulomb	115	0	6
	ESU-3 Columbia River Basalt	Mohr-Coulomb	125	0	37

Factor of Safety
≤ 0.55 - 1.05
≥ 1.05

Post-Earthquake Condition

- **Liquefaction in ESU-2 and ESU-4.**
- **Flow failure within approximately 420' behind.**

FS<1.05 within 420' behind.



Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	ESU-1 Fill (80%)	Mohr-Coulomb	120	0	29
	ESU-2 Intermediate Alluvium (Silt and Clay) (80%)	Mohr-Coulomb	105	400	0
	ESU-2 Lower Alluvium (Silt and Sand) (Residual)	Mohr-Coulomb	115	0	6
	ESU-2 Upper Alluvium (Sand and Silt) (80%)	Mohr-Coulomb	115	0	26
	ESU-2 Upper Alluvium (Sand and Silt) (Residual)	Mohr-Coulomb	115	0	6
	ESU-3 Columbia River Basalt	Mohr-Coulomb	125	0	37

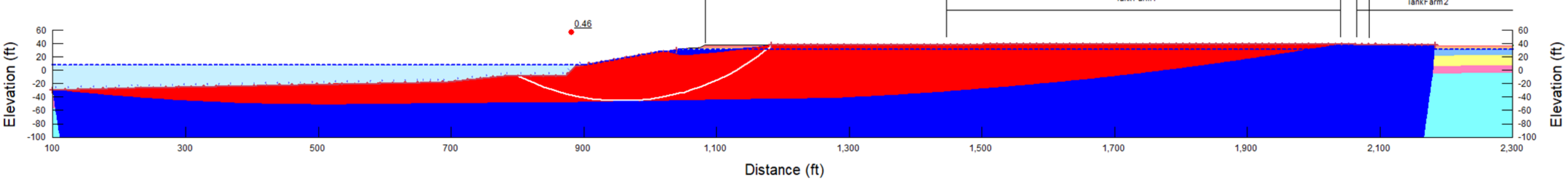
Factor of Safety
≤ 0.45 - 1.00
≥ 1.00

Horz Seismic Coef.: 0.027

Seismic Condition

- **Liquefaction in ESU-2 and ESU-4.**
- **Lateral spreading is estimated to be greater than 2 feet within 1,000 feet of the river.**

FS<1.01 within 1000' behind.



Slope Stability Analysis Results

Philips 66 Portland Terminal
Portland, Oregon

GeoEngineers

Figure 6

20545-035-01 Date: 3/27/2024

Appendices

Appendix A
Geotechnical Assessment

Appendix A
Geotechnical Assessment

This appendix presents the available geotechnical data and geotechnical assessment checklist per OAR 340-300-0003(6)(a). It also summarizes the results of our preliminary seismic hazard analysis performed to date for the McCall Terminal.

A.1 Subsurface Investigations

The logs of the subsurface investigations completed for this initial phase, which consisted of cone penetration tests (CPTs), are presented in this Appendix A.1.

A.2 Historical Geotechnical Reports

The available historical geotechnical reports are presented in Appendix A.2.

A.3 Geotechnical Assessment Checklist

The following is a checklist to satisfy the Oregon DEQ requirements for the geotechnical component of the seismic vulnerability assessment OAR 340-300-0003(6)(a):

1. *Provide a scale plan or plot drawing of the entire facility, including all tanks, berms, marine terminals, loading racks, pipelines, etc. [GEO1].*

   Response: A site map including the above items is shown in Figure 2 in the main body of this report.

2. *Provide all available soil data, boring logs, and geotechnical reports developed for the site since the original design and as-build properties of the facility. [GEO2].*

   Response: All available soil data, boring logs, and geotechnical reports developed for this site were included in this report. Please refer to Appendices A.1 through A.3 for details.

3. *Provide locations of all existing boreholes or CPTs on the plan or plot drawings. [GEO3]*

   Response: The approximate locations of all existing Cone Penetration Tests (CPTs) are presented in Figure 3 in the main body of this report.

4. *Do the borings, CPTs, and other geotechnical investigational tools meet the following criteria and conform to Oregon Structural Specialty Code 2022 ed. [GEO4].*

   a. *Boring or CPT depth shall be minimum 100 feet.*

      Response: The existing CPTs terminated at approximately 20 to 80 feet below ground surface (bgs). CPTs terminated at practical refusal, likely upon encountering the underlying Columbia River Basalt.

b. *Borings are to be onshore and offshore (if any marine structures).*

Response: All the existing CPTs are onshore. We propose offshore explorations near the dock in the next phase.

c. *Spacing of boreholes or CPTs along the berms shall not be more than 200 feet.*

Response: Spacing between the existing CPTs are more than 200 feet. We propose four additional geotechnical borings to be completed in the next phase, as discussed in Section 4.1.1 in the main body of this report, to obtain supplemental subsurface information. We also proposed to do geophysical survey across the site, as discussed in Section 4.1.1 in the main body of this report, that includes one-dimensional (1D) multi-channel analysis of surface waves (MASW) to capture the site-specific Vs down to 100 feet approximately. The combination of additional geotechnical borings and geophysical survey should be adequate to capture and refine the subsurface conditions across the site, in our opinion.

d. *If CPTs are used, a few cases of verification of results should be compared with those from adjacent borings. Relationships between the SPTs, CPTs, and full borings should be provided, using the latest geotechnical references and procedures.*

Response: Geotechnical borings were not completed in the recently completed and previous subsurface explorations. We propose four geotechnical borings to be completed in the next phase, as discussed in Section 4.1.1 in the main body of this report. The comparison between CPTs and adjacent borings will be incorporated in our development of representative soil engineering properties in the next phase.

e. *Provide geologic cross sections (color) of the facility to provide stratigraphy of the site, and to establish the site classification (A-F).*

Response: Provided in Figure 4 in the main body of this report.

f. *If any other geotechnical data (other than CPT, SPT or borings) was available, provide details and dates.*

Response: All geotechnical data available in this phase were included in this report.

g. *Employ contemporary standards of practice for all new soil investigations.*

Response: We will employ contemporary standards of practice for all new soil investigations in the next phase since no new subsurface investigations were done during this phase.

h. *Verify compliance with items (i) through (v) of OAR340-300-0003(6)(a).*

Response: Verified.

5. *The following consideration must be addressed in the geotechnical design report [GEO5]*

a. Liquefaction potential in “sand-like” soil and cyclic degradation in “clay-like” soil. How was cyclic resistance ratio (CRR) evaluated (simplified or site-specific)?

Response: Simplified liquefaction analysis was used to evaluate the liquefaction potential of the site soils. Please refer to Section A.4 for detailed information.

b. If a site-*specific response analysis has been performed, was it one or two dimensional?*

Response: One-dimensional (1D) site-specific response analysis has not been performed.

c. *What ground motion parameters were used?*

Response: We used MCER peak ground acceleration (PGA) of 0.634g and crustal magnitude of 6.9 derived from site-specific MCE_R horizontal response spectra, and a second analysis with a Cascadia Subduction Zone (CSZ) earthquake deterministic 84th percentile PGA value of 0.62g and a magnitude of 9.0 in the simplified liquefaction analysis. Please refer to Section A.4 for detailed information.

d. *What methodology was used to calculate residual shear strength?*

Response: We developed the residual shear strengths for the potentially liquefiable soils based on correlations with the CPTs. The CPT-based correlations were completed using a commercial software, CPeT-IT, developed by GeoLogismiki. Please refer to Table A-5 in Section A.5 for the developed residual shear strengths.

e. *What safety factor for liquefaction in sand (CRR/CBR)?*

Response: We performed the simplified liquefaction analysis to evaluate the liquefaction potential of the site soils. Please refer to Appendix A.4 for detailed information.

f. *If using a simplified procedure, what current methodology has been used? Is the safety factor less than 1.4, what reduction factor has been applied to the initial shear strength of the soil?*

Response: We performed simplified liquefaction analysis, as discussed in Section A.4.

g. *If the safety factor is 1.0 < SF <1.2, how have the seismically induced ground movements been evaluated?*

Response: We evaluated the seismically induced ground movements (e.g., lateral movement and vertical settlements) based on the simplified liquefaction analysis. Please refer to Section 3.1.2 in the main body of the report and Section A.5 regarding the seismically induced ground movements.

h. *If the safety factor < 1.0, what is the residual shear strength?*

Response: Please refer to Table A-5 in Section A.5 for the developed residual shear strengths.

6. *Provide evaluations for other geotechnical hazards, if applicable [GEO6]*

    a. *Slope movement.*

    b. *Lateral spreading.*

    c. *Ground settlement.*

    d. *Other surface manifestations.*

Response: Please refer to Sections 3.1.2 and 3.1.3 in the main body of the report and Sections A.4 and A.5 for related information.

7. *Slope Stability [GEO7]*

    a. *Any possibility that a slope failure that could affect any component of the facility?*

    Response: Yes.

b. *If a slope failure is possible, has a stability analysis been performed?*

Response: Yes, slope stability analysis was performed to evaluate the slope failure.

c. *Are seismically induced ground movements considered?*

Response: Yes, seismically induced ground movements were considered.

d. *If there are ground movements considered, what method have been used to analyze?*

Response: We performed slope stability analysis and simplified Newmark analysis to evaluate the earthquake-induced lateral ground deformations, as discussed in Section A.5. We estimated the liquefaction-induced settlement using empirical procedures based on Boulanger and Idriss (2014) correlations for CPTs, as discussed in Section 3.1.2 in the main body of this report.

e. *Is the expected seismic (DE) displacement greater than 0.1ft?*

Response: Yes, the earthquake-induced lateral ground deformations are anticipated to be greater than 0.1 feet, as discussed in Section A.5.

8. *Soil Structure Interaction [GEO8]*

   a. *What aspects of dynamic SSI have been evaluated (e.g., piles, pipelines, tanks, earth retention system, or other)?*

   b. *What assumptions and procedures have been used to assess SSI?*

   Response: The dynamic SSI hasn't been evaluated in this phase.

9. *The geotechnical design report documents design requirements, assumptions, calculation processes and results. This document should present a complete set of information that allows for thorough review of all calculations and data analyzed to develop design recommendations and provide input into the determination of the seismic demand (ref. 4) [GEO9]*

   a. *Description of the local geologic and geomorphologic setting of the facility.*

   b. *Include any and all historical geotechnical data, reports, or boring information.*

   c. *Present subsurface profiles in graphical cross sections.*

   d. *Describe groundwater levels and possible artesian or sub-artesian conditions.*

   e. *Identify main subsurface units, based on material type, strength, and deformability.*

   f. *Assess lateral variability of subsurface units.*

   g. *Summarize main soil and rock parameters, for each of the identified subsurface units.*

   h. *Describe the lateral variability to top of rock, where rock is present within the depth of concern.*

   i. *Likelihood of encountering rock or cobbles that might be present within the soil matrix.*

   j. *Provide justification for the "site classification" (A-F) for this facility.*

   k. *Anay additional requirements per Oregon Specialty Code Section 1803.6?*

Response: Most of the above items were covered in this report. We will refine the geotechnical design report in the next phase by including the results from the proposed work as discussed in Section 4.1 in the main body of this report.

A.4 Site-Specific Ground Response Analysis

We used the following procedure to develop the recommended site-specific MCE_R response spectrum and evaluate the liquefaction susceptibility of the subsurface soils:

1. Determine the ASCE 7-16 Site Class and mapped seismic parameters.
2. Complete a site-specific probabilistic seismic hazard analysis (PSHA) to compute a rock outcrop uniform hazard response spectrum (UHS) for the maximum considered earthquake (MCE) event (i.e., 2 percent probability of exceedance in 50 years, 2,475-year return period ground motion).
3. Develop maximum component adjustment (MCA) factors and risk coefficients.
4. Develop a site-specific probabilistic MCE_R response spectrum by scaling the rock outcrop MCE UHS by MCA factors and risk coefficients.
5. Develop deterministic MCE_R response spectra as the envelope of the 84th percentile maximum-direction response spectra computed for the Portland Hills fault, CSZ interface, and CSZ intraslab sources.
6. Develop a recommended site-specific MCE_R response spectrum as the lesser of the probabilistic MCE_R response spectrum and the deterministic MCE_R response spectrum, but not less than the ASCE 7-16 Section 21.3 minimum.
7. Evaluate the liquefaction susceptibility of the subsurface soils based on the simplified liquefaction analysis results.

SITE CLASS AND MAPPED SEISMIC PARAMETERS

Based on the three recently completed seismic CPTs (CPT-2A, CPT-4A and CPT-4A2) (locations as shown in Figure 3 in the main body of the report), the site was classified as Site Class D based on the Vs measurements. We used this Vs30-based site class only for deriving code minimum ground motions for the site-specific GRA per ASCE 7-16 Section 21.3. This evaluation does not consider the ASCE 7-16 Ch. 20 criteria that could result in Site Class F. See discussion in Section 3.1.1 in the main body of the report and following section in this appendix regarding site liquefaction risk and resulting Site Class F classification.

Table A-1 presents the site-specific minimum parameters used for the derivation of the ASCE 7-16 Section 21.3 minimum for site-specific ground motions, which has 80 percent of spectral accelerations determined in accordance with Section 11.4.6 and the site-specific minimum parameters provided in Table A-1. Since the site is in Portland, Oregon, the site-specific minimum parameters were also adjusted based on 2019 Oregon Structural Specialty Code (OSSC) 1613.2.3.1 Modification.

TABLE A-1. ASCE 7-16 AND 2019 OSSC MAPPED SEISMIC PARAMETERS

PARAMETER	VALUE
Site Class	D[1]
Short-period mapped MCE_R spectral response acceleration, S_S (g)	0.892
Long-period mapped MCE_R spectral response acceleration, S_1 (g)	0.406
Short-period site coefficient, F_a	1.143
Long-period site coefficient, F_v	1.89
Short-period MCE_R spectral response acceleration adjusted for site class, S_{MS} (g)	1.02
Long-period MCE_R spectral response acceleration adjusted for site class, S_{M1} (g)	1.15
Short-period design spectral response acceleration adjusted for site class, S_{DS} (g)	0.68
Long-period design spectral response acceleration adjusted for site class, S_{D1} (g)	0.77
Long-period transition period, T_L (s)	16

Notes:

[1] The site is classified as Site Class F due to the presence of potentially liquefiable soils. Site Class D is used to determine site-specific minimum ground motions only.

SITE-SPECIFIC PROBABILISTIC SEISMIC HAZARD ANALYSIS

Near fault effects

Near-fault effects from the Portland Hills fault were not considered for this project. In previous years, we have implemented directivity in the hazard runs, and there has been no change in the hazard. We believe this occurs because the directivity effects are minimal, and the additional epistemic uncertainty in the ground motion model (GMM) we included for the shallow crustal sources envelopes these effects. Based on this information and recognizing there are large differences (i.e., epistemic uncertainty) between the directivity models for dip-slip faults (Donahue et al. 2019), it is our opinion that it is appropriate to not include directivity effects in the seismic hazard.

Sedimentary Basin Effects

Incorporation of sedimentary basin effects was considered into the probabilistic MCE (Section PROBABILISTIC [MCER] HORIZONTAL RESPONSE SPECTRA) because the USGS 2023 national seismic hazard maps have already considered basin effects for the Portland region (Peterson et al., 2023). Sedimentary basin effects were not included in the deterministic MCE because the recommended site-specific MCE_R is based on the probabilistic MCE_R and as such the deterministic MCE should not be used in any follow-on analyses (Refer to Section RECOMMENDED SITE-SPECIFIC MCER HORIZONTAL RESPONSE SPECTRA PER ASCE 7-16).

DETERMINISTIC (MCER) HORIZONTAL RESPONSE SPECTRA

We evaluated 84th percentile deterministic (MCE_R) horizontal response spectra per ASCE 7-16 Section 21.2.2. We first computed maximum-direction deterministic response spectra for the Portland Hills fault, CSZ intraslab, and CSZ interface sources using the best estimate time-averaged Vs in the upper 30 meters (m) (100 feet) (VS30) value for each GMM listed in Table A-2 for its respective source-type. Table A-3 summarizes the magnitude and distance parameters used in the deterministic analyses, which corresponds to the disaggregation magnitude and distance for the given source at the 2475-year return period using the National Seismic Hazard Model (NSHM) Tool United States Geological Survey (USGS) 2023 (Beta Version). Figure A-1 presents the MCE PGA deaggregation, showing the magnitude and distance distribution that is bi-model; moderate magnitude crustal events with Mw of 6.5 occurring at short distances (< 10 km) and large CSZ magnitudes of Mw 9.0 occurring at moderate distances (60 km to 130 km).

We next computed weighted deterministic scenarios using the GMM weights listed in Table 3 and developed the deterministic MCE_R spectra by scaling the deterministic MCE response spectra computed by the MCA factors and risk coefficients. Figure A-2 presents our recommended deterministic MCE_R response spectrum, which is taken as the envelope of the deterministic response spectra from the three sources. The ASCE 7-16 Section 21.3 Site Class D minimum response spectrum are also shown in Figure A-2. The Portland Hills fault scenario produces the largest ground motion across all periods. This is expected given the short rupture distance and large magnitude of the Portland Hill scenario. Figure A-2 presents the deterministic MCE_R and Table A-4 lists the MCE_R values.

TABLE A-2. GROUND MOTION MODELS AND WEIGHTS

EARTHQUAKE SOURCE	GROUND MOTION MODEL	WEIGHT
Shallow crustal	Abrahamson et al. (2014) [ASK14]	0.25
	Boore et al. (2014) [BSSA14]	0.25
	Campbell and Bozorgnia (2014) [CB14]	0.25
	Chiou and Youngs (2014) [CY14]	0.25
CSZ Intraslab	Abrahamson and Gülerce (2020; 2022) – Cascadia [AG20-C]	0.40
	Kuehn et al. (2020) – Cascadia [KBCG20-C]	0.40
	Parker et al. (2020; 2021) – Cascadia [PSBAH20-C]	0.20
CSZ Interface	Abrahamson and Gülerce (2020; 2022) – Cascadia [AG20-C]	0.34
	Kuehn et al. (2020) – Cascadia [KBCG20-C]	0.33
	Parker et al. (2020; 2021) – Cascadia [PSBAH20-C]	0.33

TABLE A-3. DETERMINISTIC EARTHQUAKE SCENARIOS

PARAMETER	PORTLAND HILLS FAULT	CSZ INTRASLAB	CSZ INTERFACE
Mw	6.9	6.9	9.1
V_{S30} (m/s)	260	260	260
R_{RUP} (km)	1.5	52.6	73
R_{JB} (km)	1.5	-	-
R_X (km)	1.5	-	-
Style of Faulting	Strike-slip	-	-
Dip (degrees)	90	-	-
Z_{TOR} (km)	1	58	5
Z_{HYP} (km)	7	58	-
W (km)	17	-	-

PROBABILISTIC (MCE_R) HORIZONTAL RESPONSE SPECTRA

We developed the probabilistic (MCE_R) horizontal response spectra per ASCE 7-16 Section 21.2.2. The probabilistic MCE_R is the USGS 2023 (Beta Version) horizontal response spectra from the NSHM Tool Website for site class D at the 2475-year return period for the site coordinates, latitude 45.5642 and longitude 122.7397. Similarly to the deterministic MCE_R, we calculated the probabilistic MCE_R spectra by scaling the probabilistic MCE_R response spectra by the MCA factors and risk coefficients. Figure A-2 presents the probabilistic MCE_R and Table A-4 lists the MCE_R values.

RECOMMENDED SITE-SPECIFIC MCE_R HORIZONTAL RESPONSE SPECTRA PER ASCE 7-16

Per ASCE 7-16 Section 21.2.3, the site-specific MCE_R ground motions are taken as the lesser of the probabilistic MCE_R and deterministic MCE_R ground motions. The recommended site-specific MCER spectra is the greater of the site-specific MCER spectrum and ASCE 7-16 code minimum spectrum. Figure A-2 presents our recommended site-specific MCE_R spectra and Table A-4 lists the MCE_R values.

TABLE A-4. DETERMINISTIC, PROBABILISTIC, RECOMMENDED SITE-SPECIFIC MCE_R

	DETERMINISTIC	PROBABILISTIC	RECOMMENDED
PERIOD (SEC)	MCE_R	MCE_R	MCE_R
0.010	0.968	0.634	0.634
0.020	0.974	0.674	0.674
0.030	0.978	0.927	0.699
0.050	1.065	0.732	0.732
0.075	1.242	0.886	0.886
0.100	1.420	1.015	1.015
0.150	1.707	1.227	1.227
0.200	1.934	1.405	1.405
0.250	2.160	1.467	1.467
0.300	2.334	1.520	1.520
0.400	2.482	1.501	1.501
0.500	2.447	1.423	1.423
0.750	2.143	1.148	1.148
1.000	1.844	0.939	0.939
1.500	1.316	0.623	0.623
2.000	0.980	0.465	0.465
3.000	0.621	0.281	0.281
4.000	0.405	0.191	0.203
5.000	0.276	0.140	0.163
7.500	0.121	0.080	0.108
10.000	0.067	0.058	0.081

LIQUEFACTION EVALUATION

Liquefaction is a phenomenon where soils experience a rapid loss of internal strength as a consequence of strong ground shaking. Ground settlement, lateral spreading and sand boils may result from liquefaction. In general, the soils that are susceptible to liquefaction include very loose to medium dense, clean to silty sands, and some silts that are below the groundwater level.

The structures supported on liquefied soils could suffer foundation settlement, downdrag loads, or lateral movement that could be severely damaging to the structures. The evaluation of liquefaction potential is complex and dependent on numerous parameters, including soil type, grain-distribution, soil density, depth to groundwater, in-situ static ground stresses, earthquake-induced ground stresses, and excess pore water pressure generated during seismic shaking.

Typically, the liquefaction potential of a site is evaluated by comparing the cyclic stress ratio (CSR), which is the ratio of the cyclic shear stress induced by an earthquake to the initial effective overburden stress, to the cyclic resistance ratio (CRR), which is the soil resistance to liquefaction.

Liquefaction analysis of the CPTs were completed using the analytical software CLiq (GeoLogismiki 2018) and the semi-empirical approach by Boulanger and Idriss (2014). We completed these analyses using the MCER peak ground acceleration (PGA) of 0.634g and crustal magnitude of 6.9 derived from site-specific MCE_R horizontal response spectra, and a second analysis with a Cascadia Subduction Zone (CSZ) earthquake deterministic 84^{th} percentile PGA value of 0.62g and a magnitude of 9.0. Ground settlement due to liquefaction-induced settlement was estimated using empirical procedures based on Boulanger and Idriss (2014) correlations for CPTs. The simplified liquefaction analysis results are presented in Appendix C. The ground settlement results were the same for both earthquake scenarios. The results of our simplified analyses indicate that site soils are susceptible to liquefaction to depths of approximately 80 feet bgs during the design seismic events. Our analyses estimate average total free-field liquefaction-induced settlements ranging from 5 to 14 inches.

A.5 Earthquake-Induced Lateral Ground Deformation

Lateral spreading involves lateral displacements of large volumes of liquefied soil. Lateral spreading can occur on near-level ground as blocks of surface soil are displaced relative to adjacent blocks. Lateral spreading can also occur as blocks of surface soils are displaced towards a nearby slope or free face by movement of underlying liquefied soil. In the case of this project site, lateral spreading could occur during and/or after earthquakes resulting in excessive movement of the proposed building.

Earthquake-induced lateral ground deformations were evaluated by performing slope stability analyses and simplified Newmark analyses. Slope stability analyses were completed on Cross Section A-A' using Limit Equilibrium Method (LEM) with commercial software, Slope/W, developed by GEO-SLOPE International, Ltd. The lateral ground deformation of concern is mainly induced by earthquakes; therefore, the seismic and post-earthquake conditions are the two critical situations that were evaluated in our slope stability analyses.

The soil properties that were used in the slope stability analyses are listed in Table A-5. We assumed that liquefaction occurs during the earthquake; therefore, in seismic and post-earthquake conditions, residual strengths were used for the liquefied soils (below the groundwater table and above liquefaction depth); 80 percent of static strengths were used in the soils above the groundwater table; and full static strengths were used in the soils (non-liquefiable) below liquefaction depth.

TABLE A-5. SOIL PROPERTIES IN SLOPE/W ANALYSIS

SOIL UNIT	UNIT WEIGTH (PCF[3])	FRICTION ANGLE (DEG[3])	COHESION (PSF)	RESIDUAL FRICTION ANGLE (DEG)
Fill[1]	120	29	-	-
Upper Alluvium (sand and silt)[1]	115	26	-	6
Intermediate Alluvium (silt and clay)[1]	105	-	400	-
Lower Alluvium (silt and sand) [1]	115	-	-	6
Columbia River Basalt[1]	125	37	-	-

Notes:

[1] The soil properties were estimated based on the correlations on CPTs completed within the site vicinity.

[2] Residual strength is the reduced shear strength of soil after liquefaction.

[3] pcf = pound per cubic foot; deg = degree; psf = pounds per square foot

Figure A-3 presents the slope stability analyses results for post-earthquake and seismic conditions. In the post-seismic condition, the factor of safety (FOS) was estimated less than 1.05 for the slip surfaces going through the existing tank farm, which indicates a flow failure and an unstable condition. As shown in Figure A-3, in the seismic condition, the yield acceleration was estimated as 0.027g within 1,000 feet from the free face. The yield acceleration is the horizontal seismic coefficient that results in a factor of safety of 1.0 computed from limit equilibrium slope stability analysis. The corresponding earthquake-induced lateral ground deformation was estimated to be more than 2 feet using the simplified displacement approach developed by Bray and Travasarou (2007) and Bray et al. (2018).

A.6 References

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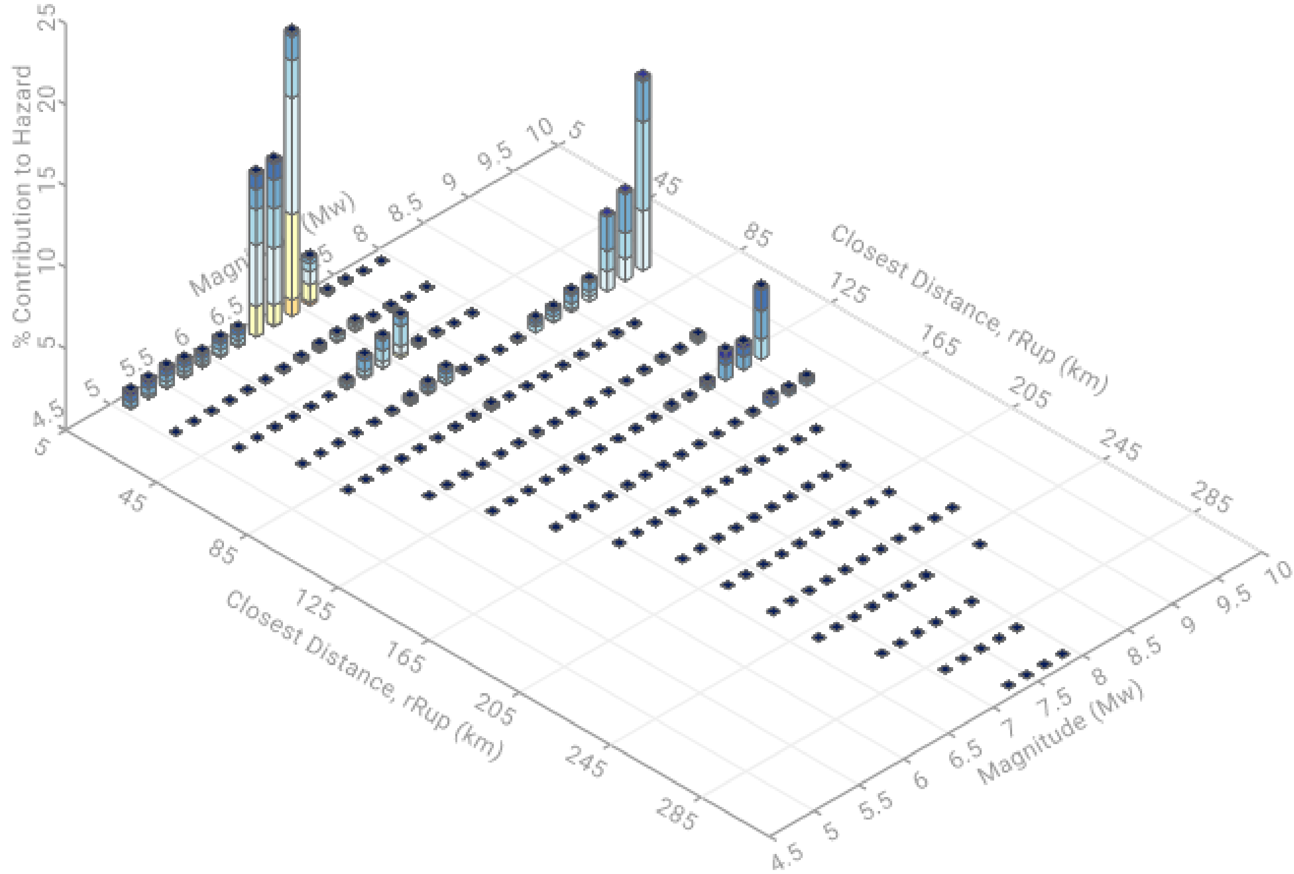
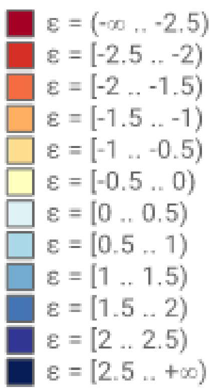




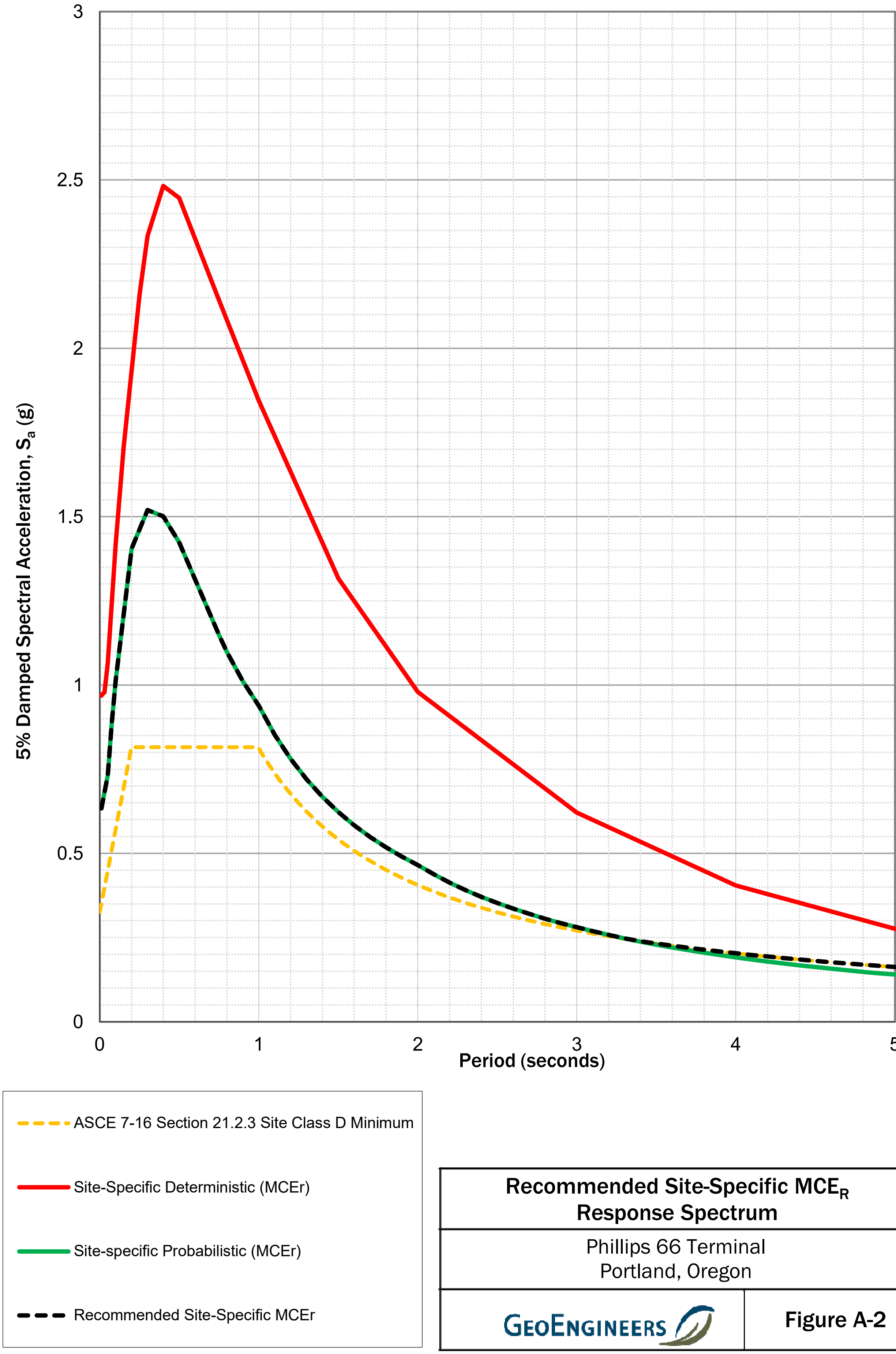
PGA Disaggregation
MCE, Site Class D, V_{S30}=853 ft/s

Phillips 66 Terminal
Portland, Oregon

GeoEngineers

Figure A-1

25045-035-02 Date:04/02/2024



25045-035-02 Date:04/02/2024

Recommended Site-Specific MCE_R Response Spectrum

Phillips 66 Terminal
Portland, Oregon

GeoEngineers

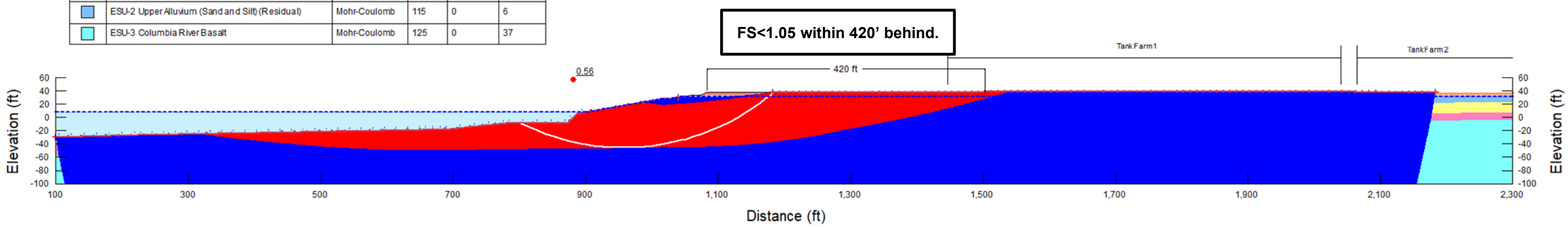
Figure A-2

Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	ESU-1 Fill (80%)	Mohr-Coulomb	120	0	29
	ESU-2 Intermediate Alluvium (Silt and Clay) (80%)	Mohr-Coulomb	105	400	0
	ESU-2 Lower Alluvium (Silt and Sand) (Residual)	Mohr-Coulomb	115	0	6
	ESU-2 Upper Alluvium (Sand and Silt) (80%)	Mohr-Coulomb	115	0	26
	ESU-2 Upper Alluvium (Sand and Silt) (Residual)	Mohr-Coulomb	115	0	6
	ESU-3 Columbia River Basalt	Mohr-Coulomb	125	0	37

Factor of Safety
- ≤ 0.55 - 1.05
- ≥ 1.05

Post-Earthquake Condition

- **Liquefaction in ESU-2 and ESU-4.**
- **Flow failure within approximately 420' behind.**



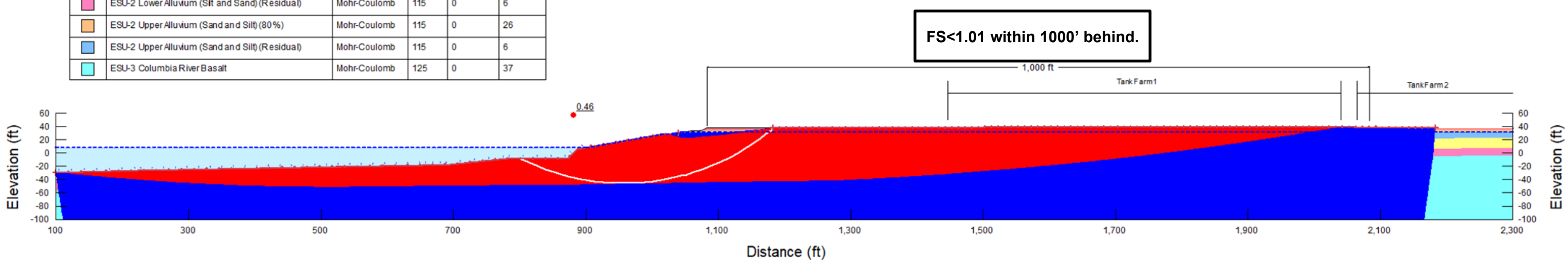
Color	Name	Slope Stability Material Model	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	ESU-1 Fill (80%)	Mohr-Coulomb	120	0	29
	ESU-2 Intermediate Alluvium (Silt and Clay) (80%)	Mohr-Coulomb	105	400	0
	ESU-2 Lower Alluvium (Silt and Sand) (Residual)	Mohr-Coulomb	115	0	6
	ESU-2 Upper Alluvium (Sand and Silt) (80%)	Mohr-Coulomb	115	0	26
	ESU-2 Upper Alluvium (Sand and Silt) (Residual)	Mohr-Coulomb	115	0	6
	ESU-3 Columbia River Basalt	Mohr-Coulomb	125	0	37

Factor of Safety
- ≤ 0.45 - 1.00
- ≥ 1.00

Horz Seismic Coef.: 0.027

Seismic Condition

- **Liquefaction in ESU-2 and ESU-4.**
- **Lateral spreading is estimated to be greater than 2 feet within 1,000 feet of the river.**



Slope Stability Analysis Results

Philips 66 Portland Terminal
Portland, Oregon

GeoEngineers

Figure A-3

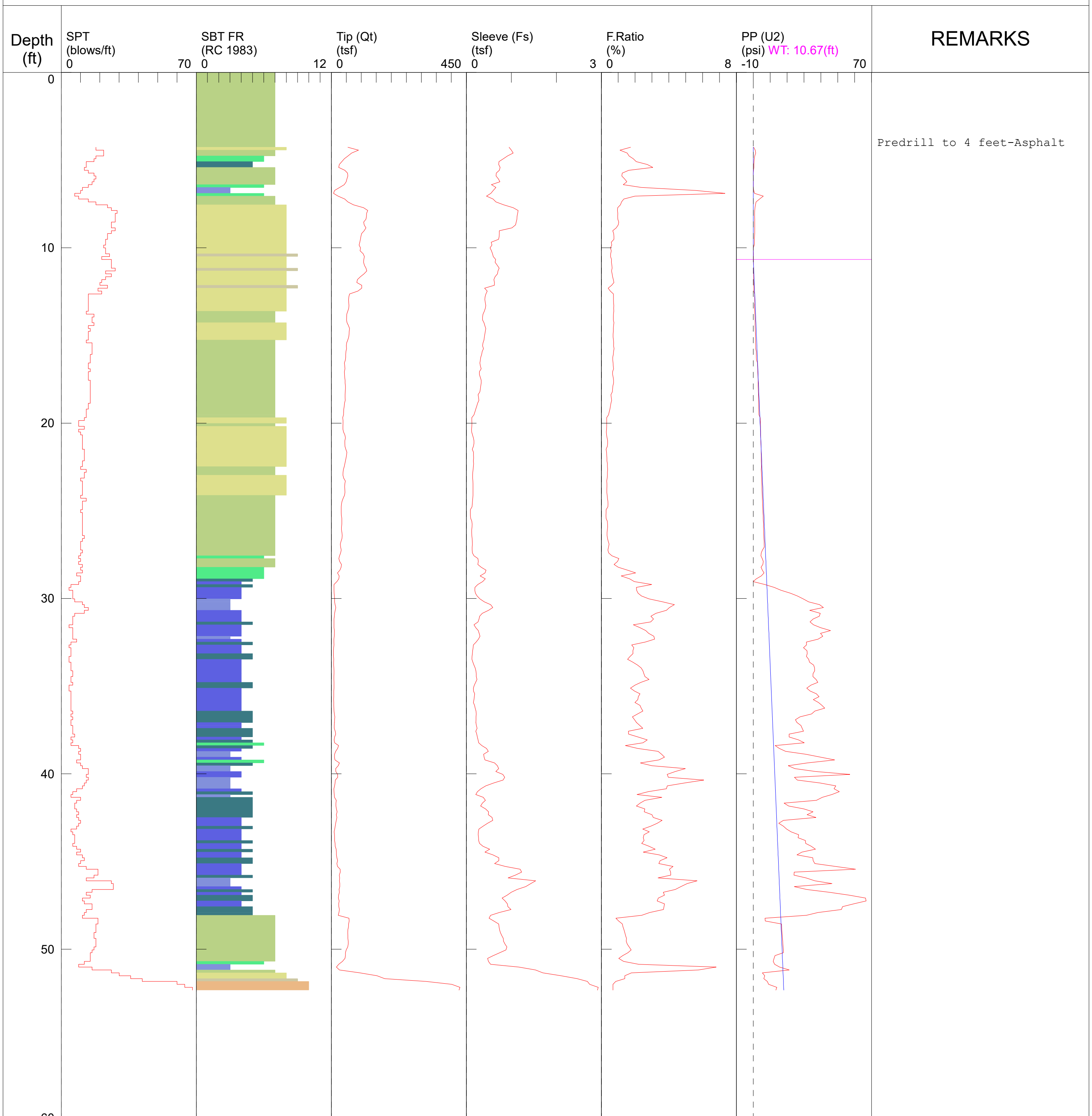
20545-035-01 Date: 3/27/2024

Appendix A.1

Subsurface Exploration Logs

GeoEngineers / CPT-2a / 5528 Doane Ave Portland

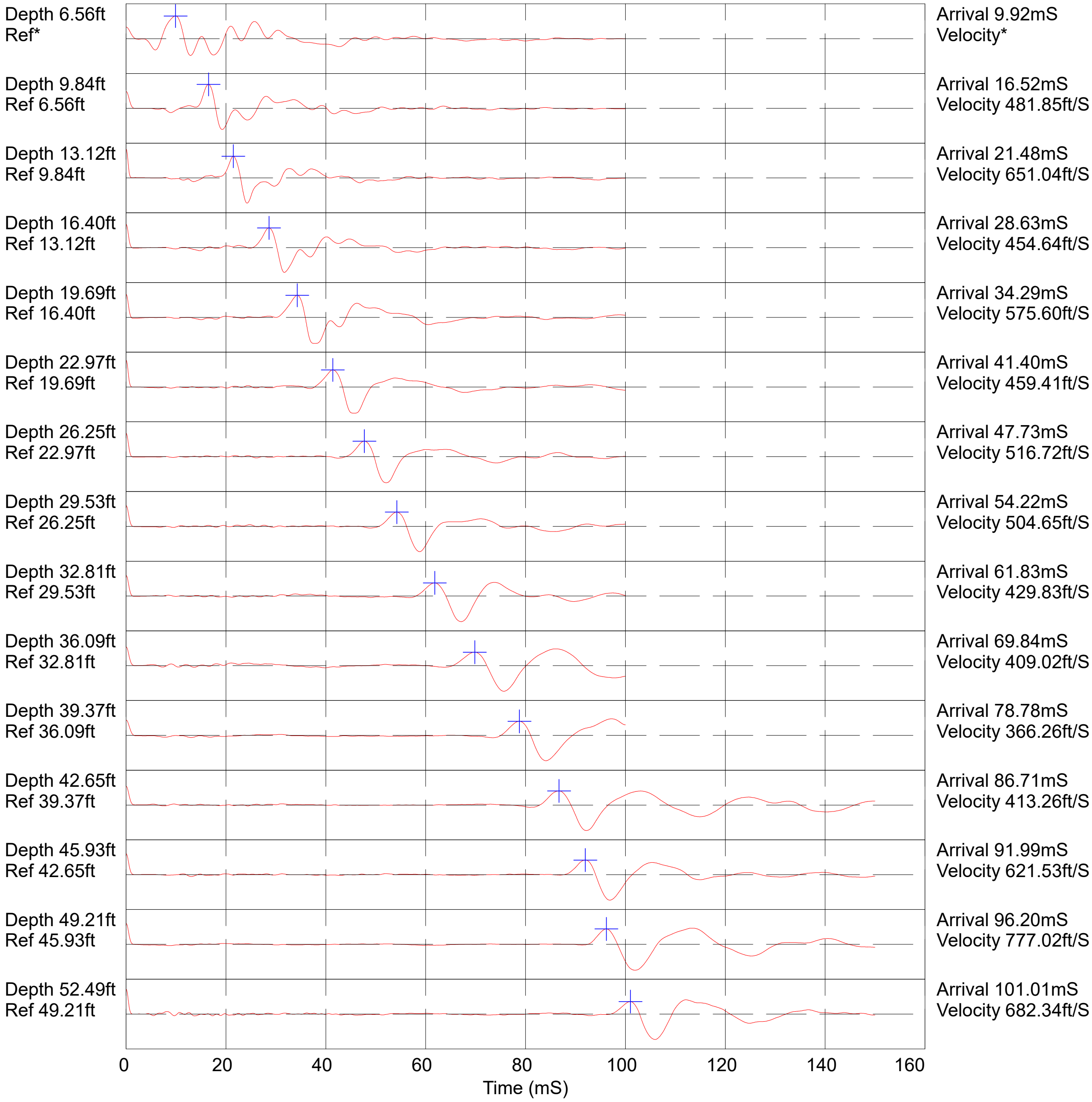
OPERATOR: OGE BAK
CONE ID: DDG1661
TEST DATE: 2/8/2024 11:08:26 AM
TOTAL DEPTH: 52.329 ft



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 (*)

*SBT/SPT CORRELATION: UBC-1983

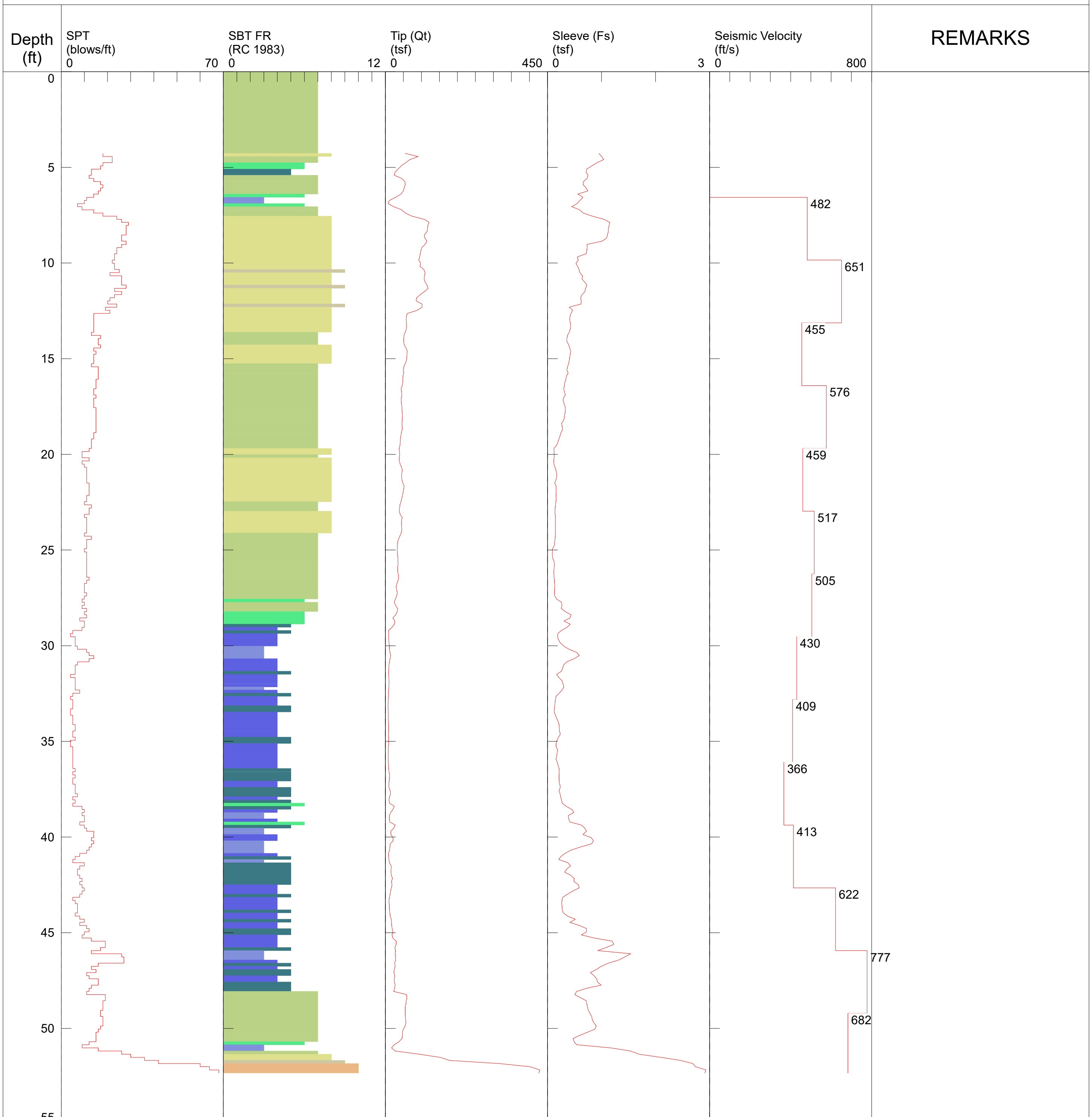
COMMENT: GeoEngineers / CPT-42 / 5528 Doane Ave Portland



Hammer to Rod String Distance (ft): 2.03
* = Not Determined

GeoEngineers / CPT-2a / 5528 Doane Ave Portland

OPERATOR: OGE BAK
CONE ID: DDG1661
TEST DATE: 2/8/2024 11:08:26 AM
TOTAL DEPTH: 52.329 ft

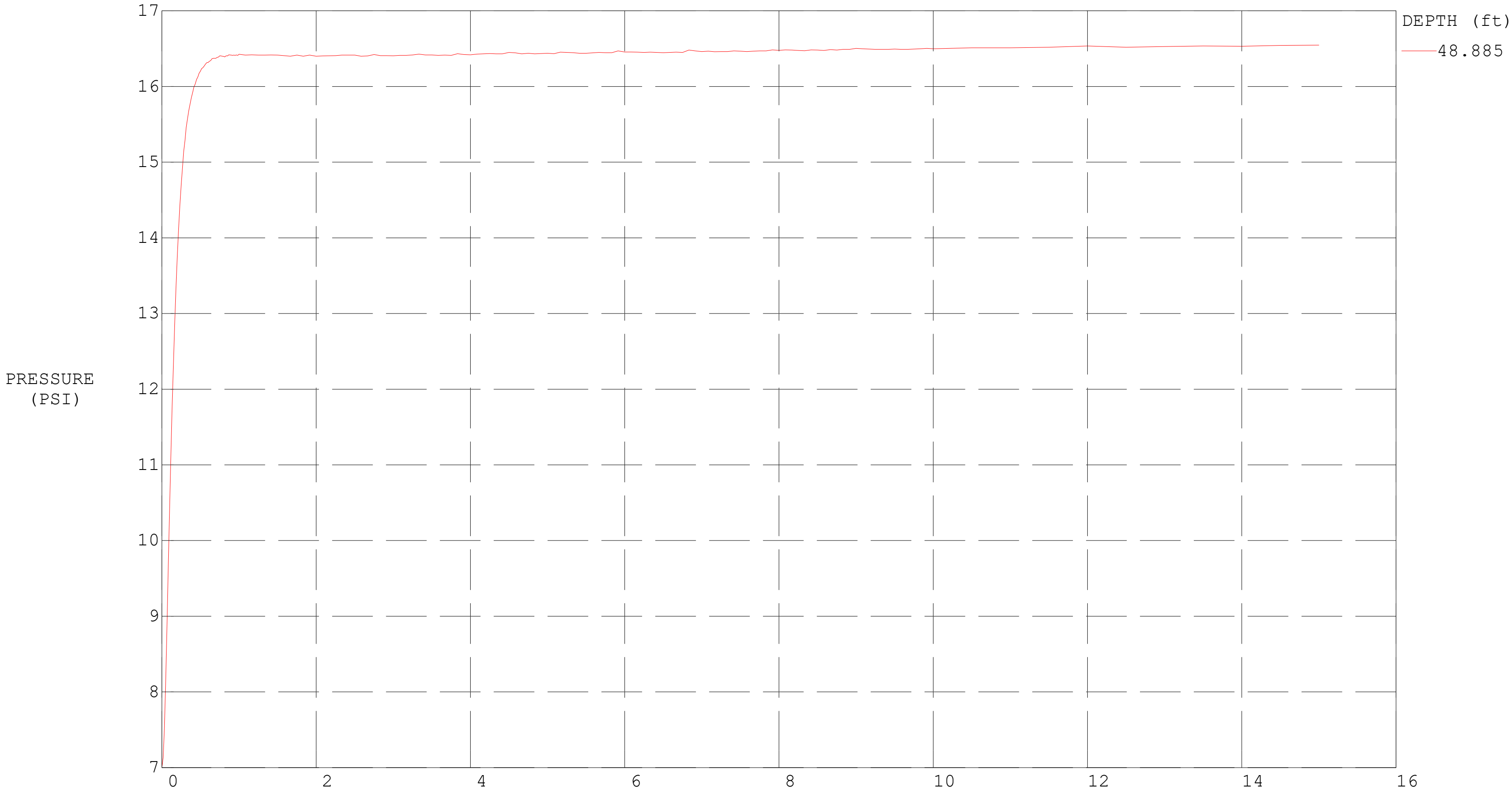


1 sensitive fine grained	4 silty clay to clay	7 silty sand to sandy silt	10 gravelly sand to sand
2 organic material	5 clayey silt to silty clay	8 sand to silty sand	11 very stiff fine grained (*)
3 clay	6 sandy silt to clayey silt	9 sand	12 sand to clayey sand (*)

*SBT/SPT CORRELATION: UBC-1983

COMMENT: GeoEngineers / CPT-2a / 5528 Doane Ave Portland

CONE ID: DDG1296
TEST DATE: 2/8/2024 11:08:26 AM



MAXIMUM PRESSURE = 16.545 (PSI)
HYDROSTATIC PRESSURE = 16.564 (PSI), WATER TABLE: 10.67 ft

GeoEngineers / CPT-2a / 5528 Doane Ave Portland

OPERATOR: OGE BAK
CONE ID: DDG1661
TEST DATE: 2/8/2024 11:08:26 AM
TOTAL DEPTH: 52.329 ft

Depth ft	Tip (Qt) (tsf)	Sleeve (Fs) (tsf)	F.Ratio (%)	PP (U2) (psi)	SPT (blows/ft)	Zone	Soil Behavior Type UBC-1983
4.265	55.04	0.9492	1.725	0.160	18	7	silty sand to sandy silt
4.429	90.61	1.0008	1.105	1.131	22	8	sand to silty sand
4.593	67.74	1.0390	1.534	1.375	22	7	silty sand to sandy silt
4.757	55.78	0.9129	1.636	0.844	18	7	silty sand to sandy silt
4.921	43.69	0.8222	1.882	0.569	17	6	sandy silt to clayey silt
5.085	35.13	0.7236	2.059	0.435	13	6	sandy silt to clayey silt
5.249	26.34	0.7171	2.722	0.361	13	5	clayey silt to silty clay
5.413	24.61	0.7498	3.046	0.337	12	5	clayey silt to silty clay
5.577	45.12	0.7313	1.621	0.148	14	7	silty sand to sandy silt
5.741	53.89	0.6642	1.233	-0.127	17	7	silty sand to sandy silt
5.906	54.91	0.6598	1.202	-0.120	18	7	silty sand to sandy silt
6.070	51.81	0.7119	1.374	-0.079	17	7	silty sand to sandy silt
6.234	49.53	0.7460	1.506	-0.062	16	7	silty sand to sandy silt
6.398	43.16	0.5584	1.294	-0.019	14	7	silty sand to sandy silt
6.562	27.72	0.6559	2.366	0.026	11	6	sandy silt to clayey silt
6.726	10.68	0.5973	5.592	0.230	10	3	clay
6.890	7.46	0.5458	7.317	0.662	7	3	clay
7.054	22.59	0.4475	1.981	5.909	9	6	sandy silt to clayey silt
7.218	44.55	0.5915	1.328	3.683	14	7	silty sand to sandy silt
7.382	55.21	0.6566	1.189	1.638	18	7	silty sand to sandy silt
7.546	73.65	0.8294	1.126	1.263	24	7	silty sand to sandy silt
7.710	107.70	1.0413	0.967	0.964	26	8	sand to silty sand
7.874	121.17	1.1505	0.950	0.837	29	8	sand to silty sand
8.038	117.44	1.1382	0.969	0.789	28	8	sand to silty sand
8.202	116.97	1.1259	0.963	0.796	28	8	sand to silty sand
8.366	115.42	1.1235	0.973	0.818	28	8	sand to silty sand
8.530	107.77	1.1068	1.027	0.842	26	8	sand to silty sand
8.694	109.28	1.0953	1.002	0.849	26	8	sand to silty sand
8.858	115.18	1.0114	0.878	0.825	28	8	sand to silty sand
9.022	109.61	0.7323	0.668	0.835	26	8	sand to silty sand
9.186	100.93	0.7337	0.727	0.835	24	8	sand to silty sand
9.350	98.59	0.7274	0.738	0.660	24	8	sand to silty sand
9.514	96.90	0.7150	0.738	0.610	23	8	sand to silty sand
9.678	94.81	0.5515	0.582	0.581	23	8	sand to silty sand
9.843	92.54	0.5682	0.614	0.552	22	8	sand to silty sand
10.007	97.65	0.5246	0.537	-0.002	23	8	sand to silty sand
10.171	95.79	0.5565	0.581	0.033	23	8	sand to silty sand
10.335	106.29	0.5802	0.546	0.048	25	8	sand to silty sand
10.499	110.39	0.5982	0.542	0.093	21	9	sand
10.663	108.22	0.6503	0.601	0.081	26	8	sand to silty sand
10.827	107.60	0.6369	0.592	0.100	26	8	sand to silty sand
10.991	110.33	0.6863	0.622	0.122	26	8	sand to silty sand
11.155	115.41	0.7286	0.631	0.196	28	8	sand to silty sand
11.319	118.28	0.7014	0.593	0.172	23	9	sand
11.483	108.78	0.6931	0.637	0.167	26	8	sand to silty sand
11.647	96.96	0.6391	0.659	0.175	23	8	sand to silty sand

Depth ft	Tip (Qt) (tsf)	Sleeve (Fs) (tsf)	F.Ratio (%)	PP (U2) (psi)	SPT (blows/ft)	Zone	Soil Behavior Type UBC-1983
11.811	86.78	0.6106	0.704	0.201	21	8	sand to silty sand
11.975	84.74	0.6238	0.736	0.234	20	8	sand to silty sand
12.139	101.61	0.6200	0.610	0.244	24	8	sand to silty sand
12.303	101.28	0.4046	0.399	0.301	19	9	sand
12.467	88.49	0.4647	0.525	0.352	21	8	sand to silty sand
12.631	60.48	0.4326	0.715	0.806	14	8	sand to silty sand
12.795	58.29	0.4162	0.714	0.801	14	8	sand to silty sand
12.959	57.59	0.4101	0.712	0.827	14	8	sand to silty sand
13.123	58.12	0.4201	0.723	0.885	14	8	sand to silty sand
13.287	58.58	0.4301	0.734	0.760	14	8	sand to silty sand
13.451	58.03	0.4201	0.724	0.792	14	8	sand to silty sand
13.615	54.47	0.4001	0.735	0.835	13	8	sand to silty sand
13.780	51.76	0.3701	0.715	0.909	17	7	silty sand to sandy silt
13.944	50.82	0.3601	0.708	0.959	16	7	silty sand to sandy silt
14.108	50.62	0.3601	0.711	1.000	16	7	silty sand to sandy silt
14.272	52.25	0.3901	0.747	1.043	17	7	silty sand to sandy silt
14.436	56.82	0.4101	0.722	1.074	14	8	sand to silty sand
14.600	60.67	0.4301	0.709	1.114	15	8	sand to silty sand
14.764	59.61	0.4201	0.705	1.174	14	8	sand to silty sand
14.928	59.20	0.4101	0.693	1.248	14	8	sand to silty sand
15.092	57.94	0.4001	0.691	1.272	14	8	sand to silty sand
15.256	54.60	0.3901	0.714	1.294	13	8	sand to silty sand
15.420	50.94	0.3701	0.727	1.327	16	7	silty sand to sandy silt
15.584	49.94	0.3601	0.721	1.365	16	7	silty sand to sandy silt
15.748	50.48	0.3801	0.753	1.392	16	7	silty sand to sandy silt
15.912	48.63	0.3501	0.720	1.719	16	7	silty sand to sandy silt
16.076	48.06	0.3401	0.708	1.779	15	7	silty sand to sandy silt
16.240	47.91	0.3201	0.668	1.815	15	7	silty sand to sandy silt
16.404	46.56	0.3101	0.666	1.894	15	7	silty sand to sandy silt
16.568	43.60	0.3000	0.688	2.473	14	7	silty sand to sandy silt
16.732	44.45	0.3101	0.698	2.516	14	7	silty sand to sandy silt
16.896	45.69	0.3301	0.722	2.580	15	7	silty sand to sandy silt
17.060	43.72	0.2901	0.664	2.633	14	7	silty sand to sandy silt
17.224	43.76	0.2901	0.663	2.697	14	7	silty sand to sandy silt
17.388	44.62	0.3101	0.695	2.769	14	7	silty sand to sandy silt
17.552	45.79	0.3301	0.721	2.819	15	7	silty sand to sandy silt
17.717	45.88	0.3301	0.719	2.838	15	7	silty sand to sandy silt
17.881	47.07	0.3201	0.680	2.884	15	7	silty sand to sandy silt
18.045	46.76	0.3101	0.663	2.939	15	7	silty sand to sandy silt
18.209	46.34	0.3000	0.647	2.972	15	7	silty sand to sandy silt
18.373	45.90	0.2601	0.567	3.018	15	7	silty sand to sandy silt
18.537	46.03	0.2701	0.587	3.061	15	7	silty sand to sandy silt
18.701	46.26	0.2801	0.605	3.106	15	7	silty sand to sandy silt
18.865	44.03	0.2500	0.568	3.233	14	7	silty sand to sandy silt
19.029	42.85	0.2301	0.537	3.257	14	7	silty sand to sandy silt
19.193	41.54	0.2100	0.506	3.281	13	7	silty sand to sandy silt
19.357	40.93	0.1900	0.464	3.307	13	7	silty sand to sandy silt
19.521	40.61	0.1701	0.419	3.326	13	7	silty sand to sandy silt
19.685	37.71	0.1196	0.317	4.046	12	7	silty sand to sandy silt
19.849	39.08	0.1214	0.311	4.094	9	8	sand to silty sand
20.013	39.35	0.1220	0.310	4.158	9	8	sand to silty sand
20.177	38.37	0.1215	0.317	4.221	12	7	silty sand to sandy silt
20.341	38.41	0.1162	0.302	4.276	9	8	sand to silty sand
20.505	40.63	0.1183	0.291	4.328	10	8	sand to silty sand
20.669	44.95	0.1373	0.306	4.383	11	8	sand to silty sand
20.833	46.82	0.1568	0.335	4.419	11	8	sand to silty sand
20.997	45.11	0.1673	0.371	4.441	11	8	sand to silty sand

Depth ft	Tip (Qt) (tsf)	Sleeve (Fs) (tsf)	F.Ratio (%)	PP (U2) (psi)	SPT (blows/ft)	Zone	Soil Behavior Type UBC-1983
21.161	45.00	0.1666	0.370	4.496	11	8	sand to silty sand
21.325	45.71	0.1569	0.343	4.534	11	8	sand to silty sand
21.490	48.25	0.1316	0.273	4.582	12	8	sand to silty sand
21.654	51.89	0.1591	0.307	4.644	12	8	sand to silty sand
21.818	50.52	0.1592	0.315	4.694	12	8	sand to silty sand
21.982	48.82	0.1597	0.327	4.708	12	8	sand to silty sand
22.146	47.13	0.1555	0.330	4.730	11	8	sand to silty sand
22.310	45.28	0.1605	0.354	4.761	11	8	sand to silty sand
22.474	43.25	0.1537	0.355	4.823	10	8	sand to silty sand
22.638	40.33	0.1488	0.369	4.735	13	7	silty sand to sandy silt
22.802	39.04	0.1397	0.358	4.792	12	7	silty sand to sandy silt
22.966	38.65	0.1379	0.357	4.857	12	7	silty sand to sandy silt
23.130	40.83	0.1392	0.341	4.948	10	8	sand to silty sand
23.294	46.53	0.1447	0.311	4.981	11	8	sand to silty sand
23.458	44.83	0.1476	0.329	5.050	11	8	sand to silty sand
23.622	44.35	0.1464	0.330	5.105	11	8	sand to silty sand
23.786	45.31	0.1465	0.323	5.146	11	8	sand to silty sand
23.950	45.63	0.1479	0.324	5.208	11	8	sand to silty sand
24.114	43.73	0.1474	0.337	5.261	10	8	sand to silty sand
24.278	39.69	0.1426	0.359	5.333	13	7	silty sand to sandy silt
24.442	35.77	0.1370	0.383	5.423	11	7	silty sand to sandy silt
24.606	33.70	0.1318	0.391	5.490	11	7	silty sand to sandy silt
24.770	32.91	0.1251	0.380	5.555	11	7	silty sand to sandy silt
24.934	32.76	0.0896	0.274	5.615	10	7	silty sand to sandy silt
25.098	32.90	0.0897	0.273	5.703	11	7	silty sand to sandy silt
25.262	33.54	0.0895	0.267	5.825	11	7	silty sand to sandy silt
25.427	34.82	0.0956	0.275	5.868	11	7	silty sand to sandy silt
25.591	34.80	0.1212	0.348	5.914	11	7	silty sand to sandy silt
25.755	35.06	0.1267	0.361	6.019	11	7	silty sand to sandy silt
25.919	33.72	0.1243	0.369	5.981	11	7	silty sand to sandy silt
26.083	33.74	0.1175	0.348	6.038	11	7	silty sand to sandy silt
26.247	35.35	0.1177	0.333	6.119	11	7	silty sand to sandy silt
26.411	36.27	0.1215	0.335	6.193	12	7	silty sand to sandy silt
26.575	35.36	0.1278	0.361	6.280	11	7	silty sand to sandy silt
26.739	32.22	0.1312	0.407	6.354	10	7	silty sand to sandy silt
26.903	29.80	0.1343	0.451	6.409	10	7	silty sand to sandy silt
27.067	30.71	0.1278	0.416	6.473	10	7	silty sand to sandy silt
27.231	33.11	0.1287	0.389	5.440	11	7	silty sand to sandy silt
27.395	31.03	0.1337	0.431	4.716	10	7	silty sand to sandy silt
27.559	27.40	0.1692	0.617	4.481	9	7	silty sand to sandy silt
27.723	24.85	0.2580	1.038	5.266	10	6	sandy silt to clayey silt
27.887	27.89	0.2658	0.953	5.875	9	7	silty sand to sandy silt
28.051	33.69	0.2538	0.753	5.603	11	7	silty sand to sandy silt
28.215	32.65	0.3276	1.003	4.520	10	7	silty sand to sandy silt
28.379	28.43	0.4383	1.542	5.342	11	6	sandy silt to clayey silt
28.543	20.44	0.4144	2.027	6.296	8	6	sandy silt to clayey silt
28.707	26.01	0.3050	1.173	4.728	10	6	sandy silt to clayey silt
28.871	25.08	0.4201	1.675	1.755	10	6	sandy silt to clayey silt
29.035	18.53	0.3608	1.947	-0.122	9	5	clayey silt to silty clay
29.199	8.58	0.2558	2.979	6.743	5	4	silty clay to clay
29.364	9.13	0.1890	2.070	12.329	4	5	clayey silt to silty clay
29.528	8.96	0.1867	2.084	16.435	6	4	silty clay to clay
29.692	9.26	0.2014	2.176	21.921	6	4	silty clay to clay
29.856	10.03	0.2379	2.373	26.039	6	4	silty clay to clay
30.020	10.66	0.2999	2.812	29.520	7	4	silty clay to clay
30.184	10.98	0.3991	3.635	32.796	11	3	clay
30.348	12.47	0.5388	4.322	39.210	12	3	clay

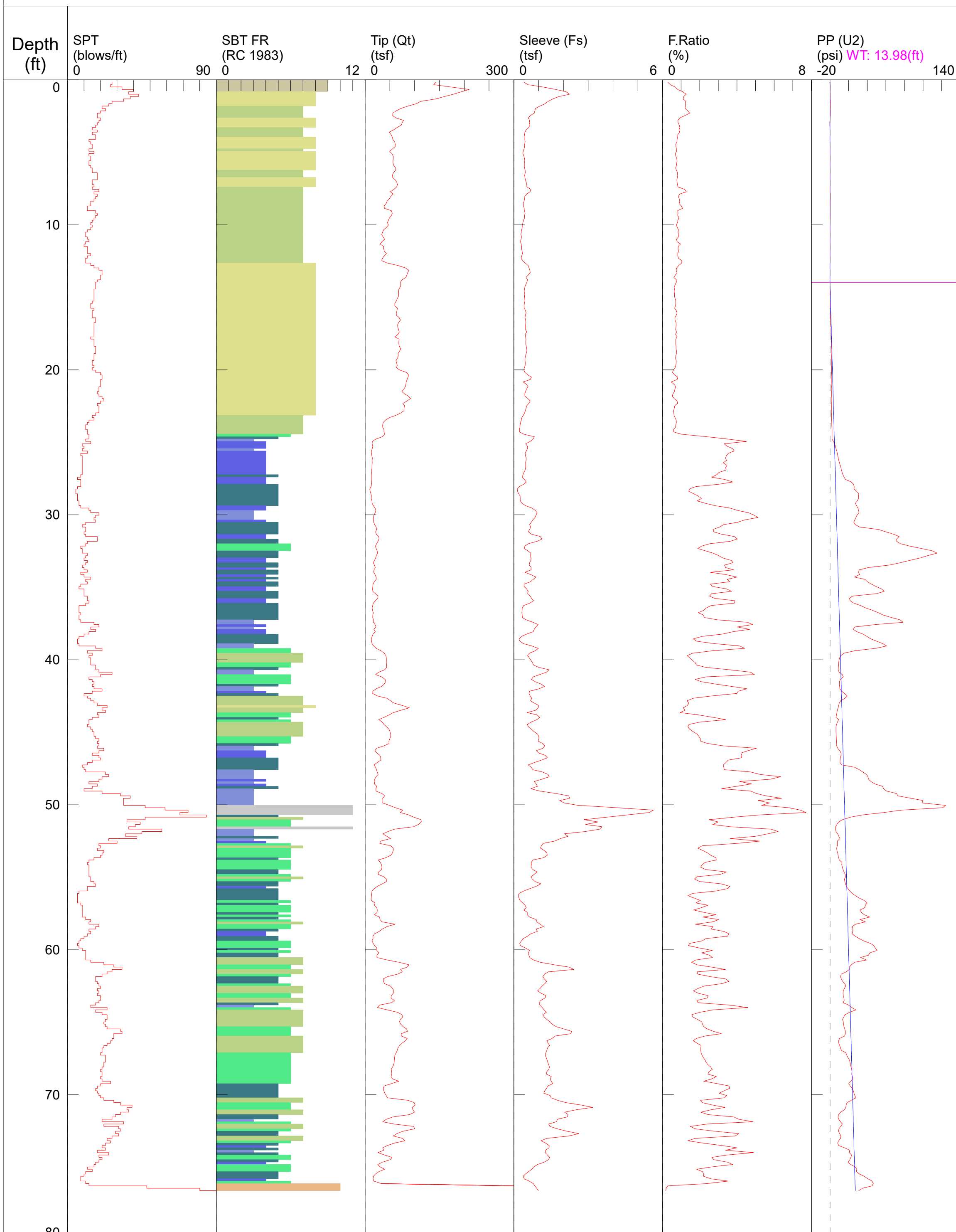
Depth ft	Tip (Qt) (tsf)	Sleeve (Fs) (tsf)	F.Ratio (%)	PP (U2) (psi)	SPT (blows/ft)	Zone	Soil Behavior Type UBC-1983
30.512	14.58	0.5880	4.033	41.525	14	3	clay
30.676	12.82	0.4935	3.850	33.445	12	3	clay
30.840	10.97	0.3538	3.226	39.648	7	4	silty clay to clay
31.004	10.08	0.2954	2.931	39.083	6	4	silty clay to clay
31.168	9.11	0.2808	3.083	35.477	6	4	silty clay to clay
31.332	8.85	0.2587	2.925	33.624	6	4	silty clay to clay
31.496	8.84	0.1674	1.894	35.652	4	5	clayey silt to silty clay
31.660	9.19	0.2100	2.285	39.884	6	4	silty clay to clay
31.824	10.16	0.2669	2.626	45.748	6	4	silty clay to clay
31.988	10.16	0.2842	2.797	39.779	6	4	silty clay to clay
32.152	9.71	0.3021	3.112	41.173	6	4	silty clay to clay
32.316	8.53	0.2687	3.150	38.347	8	3	clay
32.480	8.32	0.2159	2.595	31.402	5	4	silty clay to clay
32.644	8.62	0.1545	1.792	31.493	4	5	clayey silt to silty clay
32.808	7.74	0.1490	1.924	29.740	5	4	silty clay to clay
32.972	7.42	0.1383	1.864	31.751	5	4	silty clay to clay
33.136	7.22	0.1350	1.869	31.960	5	4	silty clay to clay
33.301	7.42	0.1260	1.697	31.395	4	5	clayey silt to silty clay
33.465	7.94	0.1229	1.548	32.794	4	5	clayey silt to silty clay
33.629	7.76	0.1445	1.863	33.294	5	4	silty clay to clay
33.793	7.87	0.1689	2.146	35.707	5	4	silty clay to clay
33.957	8.30	0.1957	2.358	36.266	5	4	silty clay to clay
34.121	8.64	0.2128	2.462	36.357	6	4	silty clay to clay
34.285	8.69	0.2185	2.513	35.336	6	4	silty clay to clay
34.449	8.52	0.2198	2.581	35.276	5	4	silty clay to clay
34.613	8.41	0.2376	2.825	37.204	5	4	silty clay to clay
34.777	8.75	0.2047	2.341	38.280	6	4	silty clay to clay
34.941	8.81	0.1754	1.991	34.155	4	5	clayey silt to silty clay
35.105	9.06	0.1560	1.722	31.641	4	5	clayey silt to silty clay
35.269	8.06	0.1575	1.954	33.650	5	4	silty clay to clay
35.433	8.12	0.1857	2.286	37.345	5	4	silty clay to clay
35.597	8.11	0.1774	2.188	38.992	5	4	silty clay to clay
35.761	7.76	0.1671	2.154	35.482	5	4	silty clay to clay
35.925	7.92	0.1571	1.984	38.591	5	4	silty clay to clay
36.089	7.88	0.1801	2.286	40.652	5	4	silty clay to clay
36.253	8.29	0.1955	2.360	42.311	5	4	silty clay to clay
36.417	8.74	0.2149	2.458	36.443	6	4	silty clay to clay
36.581	10.16	0.2180	2.145	34.509	5	5	clayey silt to silty clay
36.745	11.66	0.2137	1.833	27.665	6	5	clayey silt to silty clay
36.909	10.90	0.2139	1.963	24.912	5	5	clayey silt to silty clay
37.073	10.25	0.2130	2.077	25.838	5	5	clayey silt to silty clay
37.238	9.82	0.2234	2.274	28.210	6	4	silty clay to clay
37.402	9.77	0.2391	2.446	28.875	6	4	silty clay to clay
37.566	13.17	0.2109	1.602	29.738	6	5	clayey silt to silty clay
37.730	14.30	0.2301	1.609	21.225	7	5	clayey silt to silty clay
37.894	11.17	0.2425	2.170	21.220	5	5	clayey silt to silty clay
38.058	9.51	0.2586	2.719	26.790	6	4	silty clay to clay
38.222	11.04	0.2726	2.469	30.171	5	5	clayey silt to silty clay
38.386	24.20	0.3445	1.423	12.951	9	6	sandy silt to clayey silt
38.550	20.70	0.4467	2.158	15.462	10	5	clayey silt to silty clay
38.714	14.49	0.4873	3.362	19.157	9	4	silty clay to clay
38.878	10.74	0.3826	3.564	31.032	10	3	clay
39.042	10.55	0.3936	3.732	39.791	10	3	clay
39.206	12.13	0.4132	3.405	48.266	8	4	silty clay to clay
39.370	27.27	0.6286	2.305	31.924	10	6	sandy silt to clayey silt
39.534	23.06	0.6882	2.985	20.680	11	5	clayey silt to silty clay
39.698	14.47	0.7196	4.973	24.530	14	3	clay

Depth ft	Tip (Qt) (tsf)	Sleeve (Fs) (tsf)	F.Ratio (%)	PP (U2) (psi)	SPT (blows/ft)	Zone	Soil Behavior Type UBC-1983
39.862	14.97	0.6551	4.376	36.890	14	3	clay
40.026	20.77	0.8090	3.895	57.145	13	4	silty clay to clay
40.190	21.45	0.8512	3.969	24.396	14	4	silty clay to clay
40.354	13.46	0.8155	6.061	26.022	13	3	clay
40.518	12.31	0.6068	4.930	39.451	12	3	clay
40.682	11.03	0.4305	3.903	48.875	11	3	clay
40.846	8.40	0.3236	3.852	47.775	8	3	clay
41.011	8.87	0.2466	2.781	50.944	6	4	silty clay to clay
41.175	9.80	0.2068	2.111	46.520	5	5	clayey silt to silty clay
41.339	10.51	0.3763	3.580	40.896	10	3	clay
41.503	16.44	0.4240	2.578	37.342	8	5	clayey silt to silty clay
41.667	15.50	0.3551	2.291	18.257	7	5	clayey silt to silty clay
41.831	15.25	0.3157	2.070	20.909	7	5	clayey silt to silty clay
41.995	16.59	0.4271	2.574	29.939	8	5	clayey silt to silty clay
42.159	19.43	0.4935	2.540	35.305	9	5	clayey silt to silty clay
42.323	16.27	0.4877	2.998	31.900	8	5	clayey silt to silty clay
42.487	18.56	0.5706	3.074	37.074	9	5	clayey silt to silty clay
42.651	16.34	0.5865	3.590	17.789	10	4	silty clay to clay
42.815	14.25	0.4715	3.309	15.065	9	4	silty clay to clay
42.979	12.71	0.3728	2.934	17.767	8	4	silty clay to clay
43.143	11.43	0.2796	2.446	19.719	5	5	clayey silt to silty clay
43.307	9.38	0.2661	2.837	22.359	6	4	silty clay to clay
43.471	10.39	0.2621	2.522	26.864	7	4	silty clay to clay
43.635	10.91	0.2706	2.479	26.531	7	4	silty clay to clay
43.799	10.84	0.2736	2.523	30.888	7	4	silty clay to clay
43.963	12.26	0.2921	2.382	31.003	6	5	clayey silt to silty clay
44.127	13.13	0.3711	2.827	34.401	8	4	silty clay to clay
44.291	16.17	0.5154	3.187	36.840	10	4	silty clay to clay
44.455	16.60	0.4128	2.487	28.762	8	5	clayey silt to silty clay
44.619	17.24	0.5897	3.421	25.888	11	4	silty clay to clay
44.783	18.58	0.7216	3.884	35.221	12	4	silty clay to clay
44.948	20.73	0.7218	3.482	35.448	10	5	clayey silt to silty clay
45.112	18.46	0.6278	3.400	36.381	9	5	clayey silt to silty clay
45.276	20.37	0.8646	4.245	48.761	13	4	silty clay to clay
45.440	29.64	1.1962	4.035	60.617	19	4	silty clay to clay
45.604	29.97	1.2286	4.100	24.185	19	4	silty clay to clay
45.768	26.43	1.0871	4.113	24.044	17	4	silty clay to clay
45.932	27.68	0.9267	3.348	31.204	13	5	clayey silt to silty clay
46.096	27.19	1.5399	5.664	36.379	26	3	clay
46.260	28.15	1.4310	5.083	46.568	27	3	clay
46.424	28.02	1.3179	4.704	24.178	27	3	clay
46.588	25.74	1.1255	4.373	31.661	16	4	silty clay to clay
46.752	27.18	0.9964	3.666	44.918	13	5	clayey silt to silty clay
46.916	24.27	0.9070	3.737	56.750	15	4	silty clay to clay
47.080	23.54	0.7931	3.370	66.420	11	5	clayey silt to silty clay
47.244	26.08	0.8604	3.299	66.760	12	5	clayey silt to silty clay
47.408	24.40	0.9109	3.733	59.431	16	4	silty clay to clay
47.572	24.89	0.9234	3.709	52.929	16	4	silty clay to clay
47.736	27.01	0.9914	3.671	52.154	13	5	clayey silt to silty clay
47.900	25.43	0.7274	2.861	38.170	12	5	clayey silt to silty clay
48.064	22.64	0.5388	2.380	31.395	11	5	clayey silt to silty clay
48.228	58.61	0.5019	0.856	6.935	19	7	silty sand to sandy silt
48.392	59.52	0.6080	1.021	7.035	19	7	silty sand to sandy silt
48.556	57.72	0.7188	1.245	16.682	18	7	silty sand to sandy silt
48.720	56.86	0.7250	1.275	16.808	18	7	silty sand to sandy silt
48.885	55.11	0.7365	1.336	16.873	18	7	silty sand to sandy silt
49.049	54.74	0.7555	1.380	16.973	17	7	silty sand to sandy silt

Depth ft	Tip (Qt) (tsf)	Sleeve (Fs) (tsf)	F.Ratio (%)	PP (U2) (psi)	SPT (blows/ft)	Zone	Soil Behavior Type UBC-1983
49.213	54.79	0.7832	1.429	17.141	17	7	silty sand to sandy silt
49.377	55.18	0.8190	1.484	17.253	18	7	silty sand to sandy silt
49.541	56.16	0.8269	1.472	17.421	18	7	silty sand to sandy silt
49.705	56.37	0.8587	1.523	17.531	18	7	silty sand to sandy silt
49.869	54.75	0.9012	1.646	17.614	17	7	silty sand to sandy silt
50.033	49.72	0.8779	1.766	17.746	16	7	silty sand to sandy silt
50.197	47.22	0.7423	1.572	17.746	15	7	silty sand to sandy silt
50.361	47.26	0.5979	1.265	12.829	15	7	silty sand to sandy silt
50.525	45.70	0.4723	1.033	12.286	15	7	silty sand to sandy silt
50.689	37.30	0.4901	1.314	11.815	12	7	silty sand to sandy silt
50.853	24.42	0.5393	2.208	12.631	9	6	sandy silt to clayey silt
51.017	17.17	1.1664	6.794	15.536	16	3	clay
51.181	26.67	1.5344	5.754	21.263	26	3	clay
51.345	95.27	1.6995	1.784	5.395	30	7	silty sand to sandy silt
51.509	152.46	2.1048	1.381	6.798	36	8	sand to silty sand
51.673	177.46	2.4628	1.388	6.203	42	8	sand to silty sand
51.837	315.62	2.6900	0.852	8.291	60	9	sand
52.001	401.26	2.7383	0.682	9.082	64	10	gravelly sand to sand
52.165	428.67	2.9308	0.684	13.901	68	10	gravelly sand to sand
52.329	425.67	2.9108	0.684	13.336	68	10	gravelly sand to sand

GeoEngineers / CPT-3b / 5528 NW Doane Ave Portland

OPERATOR: OGE DMM
CONE ID: DDG1654
TEST DATE: 2/8/2024 12:43:24 PM
TOTAL DEPTH: 76.608 ft



REMARKS

Water table measured using water level indicator after tooling was removed from the hole

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 (*)

*SBT/SPT CORRELATION: UBC-1983

GeoEngineers / CPT-3b / 5528 NW Doane Ave Portland

OPERATOR: OGE DMM
CONE ID: DDG1654
TEST DATE: 2/8/2024 12:43:24 PM
TOTAL DEPTH: 76.608 ft

Depth ft	Tip (Qt) (tsf)	Sleeve (Fs) (tsf)	F.Ratio (%)	PP (U2) (psi)	SPT (blows/ft)	Zone	Soil Behavior Type UBC-1983
0.164	142.54	0.4100	0.288	-0.328	27	9	sand
0.328	138.17	0.5683	0.411	-0.043	26	9	sand
0.492	172.30	1.1574	0.672	0.104	33	9	sand
0.656	209.99	1.6888	0.804	0.018	40	9	sand
0.820	194.33	2.1037	1.083	0.069	37	9	sand
0.984	178.05	2.2467	1.262	0.122	43	8	sand to silty sand
1.148	160.44	1.8231	1.136	0.178	38	8	sand to silty sand
1.312	143.04	1.5683	1.096	0.191	34	8	sand to silty sand
1.476	112.31	1.3900	1.238	0.508	27	8	sand to silty sand
1.640	103.35	1.2595	1.219	0.399	25	8	sand to silty sand
1.804	86.25	1.0491	1.216	0.320	21	8	sand to silty sand
1.969	72.12	0.8671	1.202	0.325	23	7	silty sand to sandy silt
2.133	62.57	0.8478	1.355	0.295	20	7	silty sand to sandy silt
2.297	56.09	0.8164	1.456	0.280	18	7	silty sand to sandy silt
2.461	55.44	0.6721	1.212	0.198	18	7	silty sand to sandy silt
2.625	62.34	0.5753	0.923	0.048	20	7	silty sand to sandy silt
2.789	77.52	0.6389	0.824	-0.272	19	8	sand to silty sand
2.953	74.74	0.6188	0.828	-0.358	18	8	sand to silty sand
3.117	72.94	0.6499	0.891	-0.343	17	8	sand to silty sand
3.281	64.50	0.6007	0.931	-0.282	15	8	sand to silty sand
3.445	57.36	0.5009	0.873	-0.254	18	7	silty sand to sandy silt
3.609	48.42	0.4410	0.911	-0.198	15	7	silty sand to sandy silt
3.773	52.53	0.4297	0.818	-0.203	17	7	silty sand to sandy silt
3.937	54.76	0.4420	0.807	-0.216	17	7	silty sand to sandy silt
4.101	56.12	0.4366	0.778	-0.226	13	8	sand to silty sand
4.265	59.31	0.4542	0.766	-0.226	14	8	sand to silty sand
4.429	60.67	0.4314	0.711	-0.208	15	8	sand to silty sand
4.593	60.58	0.4330	0.715	-0.196	15	8	sand to silty sand
4.757	55.49	0.4095	0.738	-0.033	13	8	sand to silty sand
4.921	49.68	0.3731	0.751	0.003	16	7	silty sand to sandy silt
5.085	53.86	0.3662	0.680	-0.018	13	8	sand to silty sand
5.249	56.83	0.4010	0.706	-0.046	14	8	sand to silty sand
5.413	56.70	0.4275	0.754	-0.046	14	8	sand to silty sand
5.577	56.23	0.4390	0.781	-0.051	13	8	sand to silty sand
5.741	56.28	0.4265	0.758	-0.053	13	8	sand to silty sand
5.906	57.75	0.4263	0.738	-0.066	14	8	sand to silty sand
6.070	61.66	0.4308	0.699	-0.094	15	8	sand to silty sand
6.234	61.83	0.4535	0.733	-0.097	15	8	sand to silty sand
6.398	57.09	0.4607	0.807	-0.071	18	7	silty sand to sandy silt
6.562	55.19	0.4351	0.788	-0.071	18	7	silty sand to sandy silt
6.726	55.60	0.4403	0.792	-0.061	18	7	silty sand to sandy silt
6.890	60.59	0.4637	0.765	-0.079	15	8	sand to silty sand
7.054	63.84	0.4990	0.782	-0.076	15	8	sand to silty sand
7.218	65.44	0.5183	0.792	-0.091	16	8	sand to silty sand
7.382	63.35	0.5262	0.831	-0.104	15	8	sand to silty sand
7.546	58.12	0.6867	1.182	-0.091	19	7	silty sand to sandy silt

Depth ft	Tip (Qt) (tsf)	Sleeve (Fs) (tsf)	F.Ratio (%)	PP (U2) (psi)	SPT (blows/ft)	Zone	Soil Behavior Type UBC-1983
7.710	50.87	0.6508	1.279	-0.041	16	7	silty sand to sandy silt
7.874	56.41	0.5172	0.917	0.224	18	7	silty sand to sandy silt
8.038	51.81	0.4360	0.842	0.219	17	7	silty sand to sandy silt
8.202	48.63	0.4331	0.891	0.196	16	7	silty sand to sandy silt
8.366	44.55	0.4108	0.922	0.191	14	7	silty sand to sandy silt
8.530	42.47	0.3734	0.879	0.188	14	7	silty sand to sandy silt
8.694	38.52	0.3883	1.008	0.170	12	7	silty sand to sandy silt
8.858	39.11	0.4187	1.071	0.155	12	7	silty sand to sandy silt
9.022	52.90	0.4245	0.803	0.147	17	7	silty sand to sandy silt
9.186	54.94	0.4552	0.829	0.102	18	7	silty sand to sandy silt
9.350	53.42	0.4309	0.807	0.086	17	7	silty sand to sandy silt
9.514	49.24	0.4319	0.877	0.076	16	7	silty sand to sandy silt
9.678	47.05	0.3880	0.825	0.066	15	7	silty sand to sandy silt
9.843	45.10	0.3605	0.799	0.048	14	7	silty sand to sandy silt
10.007	46.91	0.3430	0.731	0.013	15	7	silty sand to sandy silt
10.171	43.37	0.3413	0.787	0.020	14	7	silty sand to sandy silt
10.335	37.61	0.3194	0.849	0.018	12	7	silty sand to sandy silt
10.499	35.29	0.2930	0.830	0.018	11	7	silty sand to sandy silt
10.663	33.43	0.2856	0.854	0.000	11	7	silty sand to sandy silt
10.827	36.21	0.3063	0.846	-0.010	12	7	silty sand to sandy silt
10.991	39.54	0.2917	0.738	-0.030	13	7	silty sand to sandy silt
11.155	34.21	0.2667	0.780	-0.041	11	7	silty sand to sandy silt
11.319	30.33	0.2931	0.966	-0.043	10	7	silty sand to sandy silt
11.483	37.60	0.3133	0.833	-0.066	12	7	silty sand to sandy silt
11.647	37.59	0.3102	0.825	-0.084	12	7	silty sand to sandy silt
11.811	38.83	0.3296	0.849	-0.109	12	7	silty sand to sandy silt
11.975	43.17	0.3493	0.809	-0.130	14	7	silty sand to sandy silt
12.139	38.70	0.3167	0.818	-0.119	12	7	silty sand to sandy silt
12.303	35.04	0.2957	0.844	-0.137	11	7	silty sand to sandy silt
12.467	33.56	0.3455	1.030	-0.160	11	7	silty sand to sandy silt
12.631	44.65	0.4573	1.024	-0.224	14	7	silty sand to sandy silt
12.795	65.99	0.5730	0.868	-0.229	16	8	sand to silty sand
12.959	80.98	0.6122	0.756	-0.244	19	8	sand to silty sand
13.123	88.47	0.6446	0.729	-0.269	21	8	sand to silty sand
13.287	86.07	0.6602	0.767	-0.264	21	8	sand to silty sand
13.451	84.32	0.5795	0.687	-0.272	20	8	sand to silty sand
13.615	82.62	0.4935	0.597	-0.267	20	8	sand to silty sand
13.780	74.74	0.5493	0.735	-0.239	18	8	sand to silty sand
13.944	71.33	0.5268	0.739	-0.219	17	8	sand to silty sand
14.108	71.83	0.4868	0.678	-0.226	17	8	sand to silty sand
14.272	71.14	0.4409	0.620	-0.211	17	8	sand to silty sand
14.436	68.11	0.4295	0.631	-0.198	16	8	sand to silty sand
14.600	67.47	0.4198	0.622	-0.076	16	8	sand to silty sand
14.764	67.96	0.4359	0.641	-0.094	16	8	sand to silty sand
14.928	67.17	0.4289	0.638	-0.081	16	8	sand to silty sand
15.092	65.02	0.4380	0.674	-0.071	16	8	sand to silty sand
15.256	62.83	0.4411	0.702	-0.056	15	8	sand to silty sand
15.420	59.76	0.4227	0.707	-0.030	14	8	sand to silty sand
15.584	60.05	0.4121	0.686	-0.005	14	8	sand to silty sand
15.748	65.24	0.4180	0.641	0.028	16	8	sand to silty sand
15.912	64.35	0.4726	0.734	0.058	15	8	sand to silty sand
16.076	63.09	0.4910	0.778	0.183	15	8	sand to silty sand
16.240	63.24	0.4603	0.728	0.280	15	8	sand to silty sand
16.404	69.01	0.4516	0.654	0.305	17	8	sand to silty sand
16.568	71.36	0.4712	0.660	0.341	17	8	sand to silty sand
16.732	67.63	0.4876	0.721	0.473	16	8	sand to silty sand
16.896	65.38	0.4754	0.727	0.793	16	8	sand to silty sand

Depth ft	Tip (Qt) (tsf)	Sleeve (Fs) (tsf)	F.Ratio (%)	PP (U2) (psi)	SPT (blows/ft)	Zone	Soil Behavior Type UBC-1983
17.060	65.88	0.4719	0.716	1.072	16	8	sand to silty sand
17.224	67.60	0.4720	0.698	0.996	16	8	sand to silty sand
17.388	68.73	0.5163	0.751	0.953	16	8	sand to silty sand
17.552	66.17	0.4569	0.691	1.042	16	8	sand to silty sand
17.717	60.16	0.4678	0.778	1.408	14	8	sand to silty sand
17.881	68.64	0.4856	0.707	1.507	16	8	sand to silty sand
18.045	68.07	0.4835	0.710	1.497	16	8	sand to silty sand
18.209	68.19	0.4806	0.705	1.499	16	8	sand to silty sand
18.373	69.12	0.4994	0.722	1.492	17	8	sand to silty sand
18.537	71.96	0.5171	0.719	1.459	17	8	sand to silty sand
18.701	69.65	0.5214	0.749	1.476	17	8	sand to silty sand
18.865	66.56	0.4938	0.742	1.489	16	8	sand to silty sand
19.029	66.79	0.4678	0.700	1.515	16	8	sand to silty sand
19.193	65.51	0.4534	0.692	1.532	16	8	sand to silty sand
19.357	62.18	0.4532	0.729	1.558	15	8	sand to silty sand
19.521	63.57	0.4602	0.724	1.565	15	8	sand to silty sand
19.685	65.11	0.4578	0.703	1.603	16	8	sand to silty sand
19.849	63.11	0.4245	0.673	1.649	15	8	sand to silty sand
20.013	70.09	0.4000	0.571	1.677	17	8	sand to silty sand
20.177	83.42	0.4419	0.530	1.657	20	8	sand to silty sand
20.341	87.71	0.5622	0.641	1.545	21	8	sand to silty sand
20.505	86.57	0.7039	0.813	1.448	21	8	sand to silty sand
20.669	84.77	0.6588	0.777	1.400	20	8	sand to silty sand
20.833	82.86	0.3737	0.451	1.380	20	8	sand to silty sand
20.997	82.44	0.5082	0.616	1.352	20	8	sand to silty sand
21.161	81.20	0.5702	0.702	1.520	19	8	sand to silty sand
21.325	77.99	0.5106	0.655	1.502	19	8	sand to silty sand
21.490	74.48	0.4744	0.637	1.507	18	8	sand to silty sand
21.654	79.70	0.4386	0.550	1.527	19	8	sand to silty sand
21.818	87.31	0.4472	0.512	1.527	21	8	sand to silty sand
21.982	91.77	0.5658	0.617	1.489	22	8	sand to silty sand
22.146	84.67	0.6623	0.782	1.502	20	8	sand to silty sand
22.310	76.11	0.6077	0.798	1.515	18	8	sand to silty sand
22.474	79.82	0.5180	0.649	1.520	19	8	sand to silty sand
22.638	78.60	0.4910	0.625	1.563	19	8	sand to silty sand
22.802	78.52	0.4640	0.591	1.583	19	8	sand to silty sand
22.966	72.63	0.4317	0.594	1.596	17	8	sand to silty sand
23.130	62.01	0.3800	0.613	1.626	15	8	sand to silty sand
23.294	49.07	0.3233	0.659	1.631	16	7	silty sand to sandy silt
23.458	41.08	0.2993	0.728	1.680	13	7	silty sand to sandy silt
23.622	36.69	0.2719	0.741	1.748	12	7	silty sand to sandy silt
23.786	35.42	0.2482	0.701	1.840	11	7	silty sand to sandy silt
23.950	35.25	0.2436	0.691	1.913	11	7	silty sand to sandy silt
24.114	37.95	0.2212	0.583	2.007	12	7	silty sand to sandy silt
24.278	37.99	0.2234	0.588	2.051	12	7	silty sand to sandy silt
24.442	40.15	0.4074	1.015	2.198	13	7	silty sand to sandy silt
24.606	34.66	0.8276	2.388	2.124	13	6	sandy silt to clayey silt
24.770	22.16	0.7708	3.478	2.145	11	5	clayey silt to silty clay
24.934	14.64	0.6591	4.501	3.471	14	3	clay
25.098	14.40	0.4759	3.305	4.815	9	4	silty clay to clay
25.262	15.73	0.5429	3.452	5.809	10	4	silty clay to clay
25.427	14.60	0.5495	3.764	6.421	9	4	silty clay to clay
25.591	13.03	0.5006	3.841	7.250	12	3	clay
25.755	12.93	0.4557	3.525	7.827	8	4	silty clay to clay
25.919	13.55	0.4654	3.434	8.246	9	4	silty clay to clay
26.083	14.66	0.4949	3.376	8.810	9	4	silty clay to clay
26.247	14.34	0.4918	3.430	9.395	9	4	silty clay to clay

Depth ft	Tip (Qt) (tsf)	Sleeve (Fs) (tsf)	F.Ratio (%)	PP (U2) (psi)	SPT (blows/ft)	Zone	Soil Behavior Type UBC-1983
26.411	14.53	0.4732	3.256	10.129	9	4	silty clay to clay
26.575	13.86	0.4784	3.452	10.995	9	4	silty clay to clay
26.739	13.82	0.4688	3.392	11.684	9	4	silty clay to clay
26.903	13.39	0.4573	3.415	12.246	9	4	silty clay to clay
27.067	13.37	0.4055	3.032	12.995	9	4	silty clay to clay
27.231	12.68	0.3696	2.914	13.793	8	4	silty clay to clay
27.395	12.64	0.3325	2.631	15.155	6	5	clayey silt to silty clay
27.559	12.76	0.4260	3.338	16.591	8	4	silty clay to clay
27.723	13.13	0.4947	3.768	22.835	8	4	silty clay to clay
27.887	13.19	0.3984	3.021	25.203	8	4	silty clay to clay
28.051	12.15	0.2148	1.767	26.008	6	5	clayey silt to silty clay
28.215	9.48	0.1369	1.445	25.881	5	5	clayey silt to silty clay
28.379	10.19	0.1435	1.408	28.499	5	5	clayey silt to silty clay
28.543	11.74	0.1969	1.677	30.844	6	5	clayey silt to silty clay
28.707	13.01	0.2500	1.921	30.725	6	5	clayey silt to silty clay
28.871	12.30	0.2566	2.086	30.542	6	5	clayey silt to silty clay
29.035	14.04	0.2595	1.848	29.330	7	5	clayey silt to silty clay
29.199	14.31	0.3274	2.287	27.269	7	5	clayey silt to silty clay
29.364	16.80	0.4732	2.817	28.367	8	5	clayey silt to silty clay
29.528	20.09	0.7263	3.616	29.899	13	4	silty clay to clay
29.692	21.58	0.8689	4.027	30.697	14	4	silty clay to clay
29.856	20.32	0.9365	4.609	31.152	19	3	clay
30.020	18.12	0.8922	4.922	28.974	17	3	clay
30.184	16.64	0.8525	5.123	27.876	16	3	clay
30.348	17.32	0.7644	4.413	26.641	17	3	clay
30.512	17.34	0.6992	4.033	25.548	11	4	silty clay to clay
30.676	19.44	0.6363	3.273	27.068	9	5	clayey silt to silty clay
30.840	22.44	0.6862	3.058	31.868	11	5	clayey silt to silty clay
31.004	23.30	0.6313	2.710	52.207	11	5	clayey silt to silty clay
31.168	21.32	0.5868	2.752	59.958	10	5	clayey silt to silty clay
31.332	22.58	0.7183	3.182	68.420	11	5	clayey silt to silty clay
31.496	27.67	1.0648	3.848	74.292	18	4	silty clay to clay
31.660	28.24	1.1346	4.017	71.566	18	4	silty clay to clay
31.824	25.80	0.9368	3.631	71.561	12	5	clayey silt to silty clay
31.988	23.16	0.5885	2.541	75.830	11	5	clayey silt to silty clay
32.152	21.00	0.4407	2.098	85.511	8	6	sandy silt to clayey silt
32.316	22.30	0.4195	1.881	98.207	9	6	sandy silt to clayey silt
32.480	24.34	0.5617	2.307	110.023	9	6	sandy silt to clayey silt
32.644	25.14	0.6816	2.712	115.136	12	5	clayey silt to silty clay
32.808	22.45	0.6762	3.012	104.801	11	5	clayey silt to silty clay
32.972	21.45	0.6928	3.230	97.020	10	5	clayey silt to silty clay
33.136	18.69	0.6898	3.690	85.402	12	4	silty clay to clay
33.301	16.63	0.6302	3.790	75.545	11	4	silty clay to clay
33.465	18.66	0.6190	3.317	63.607	9	5	clayey silt to silty clay
33.629	20.50	0.6950	3.390	41.370	10	5	clayey silt to silty clay
33.793	18.19	0.6928	3.808	31.246	12	4	silty clay to clay
33.957	17.52	0.4487	2.562	30.740	8	5	clayey silt to silty clay
34.121	20.10	0.6812	3.390	31.754	10	5	clayey silt to silty clay
34.285	22.24	0.8892	3.998	26.207	14	4	silty clay to clay
34.449	22.47	0.7805	3.473	37.494	11	5	clayey silt to silty clay
34.613	18.65	0.6708	3.596	40.160	12	4	silty clay to clay
34.777	16.11	0.4128	2.562	44.625	8	5	clayey silt to silty clay
34.941	15.58	0.4195	2.692	49.400	7	5	clayey silt to silty clay
35.105	14.92	0.5130	3.438	55.338	10	4	silty clay to clay
35.269	15.55	0.5771	3.711	58.301	10	4	silty clay to clay
35.433	20.90	0.5504	2.634	49.832	10	5	clayey silt to silty clay
35.597	25.98	0.6546	2.519	24.184	12	5	clayey silt to silty clay

Depth ft	Tip (Qt) (tsf)	Sleeve (Fs) (tsf)	F.Ratio (%)	PP (U2) (psi)	SPT (blows/ft)	Zone	Soil Behavior Type UBC-1983
35.761	25.49	0.7128	2.796	20.799	12	5	clayey silt to silty clay
35.925	20.48	0.7963	3.888	21.650	13	4	silty clay to clay
36.089	16.73	0.6443	3.852	25.467	11	4	silty clay to clay
36.253	15.25	0.3985	2.613	32.748	7	5	clayey silt to silty clay
36.417	14.26	0.3339	2.341	38.211	7	5	clayey silt to silty clay
36.581	14.99	0.3309	2.207	45.664	7	5	clayey silt to silty clay
36.745	16.85	0.3224	1.914	51.913	8	5	clayey silt to silty clay
36.909	15.59	0.3412	2.189	56.891	7	5	clayey silt to silty clay
37.073	15.34	0.3438	2.241	65.947	7	5	clayey silt to silty clay
37.238	16.88	0.4923	2.917	75.202	8	5	clayey silt to silty clay
37.402	16.99	0.7749	4.562	78.635	16	3	clay
37.566	20.17	0.9762	4.839	49.039	19	3	clay
37.730	22.02	0.8851	4.020	26.545	14	4	silty clay to clay
37.894	17.59	0.8228	4.676	24.959	17	3	clay
38.058	20.50	0.7807	3.809	28.735	13	4	silty clay to clay
38.222	16.02	0.5121	3.196	35.553	10	4	silty clay to clay
38.386	14.31	0.3024	2.114	39.548	7	5	clayey silt to silty clay
38.550	12.80	0.2104	1.644	43.827	6	5	clayey silt to silty clay
38.714	13.28	0.2425	1.826	50.121	6	5	clayey silt to silty clay
38.878	13.91	0.3836	2.758	56.665	7	5	clayey silt to silty clay
39.042	17.51	0.7257	4.144	60.639	17	3	clay
39.206	22.36	0.9856	4.409	48.978	21	3	clay
39.370	32.18	0.8625	2.680	26.537	12	6	sandy silt to clayey silt
39.534	38.68	0.6275	1.622	15.094	15	6	sandy silt to clayey silt
39.698	41.82	0.5531	1.323	11.666	13	7	silty sand to sandy silt
39.862	42.93	0.6223	1.450	10.246	14	7	silty sand to sandy silt
40.026	43.16	0.6862	1.590	9.618	14	7	silty sand to sandy silt
40.190	42.92	0.7517	1.751	9.308	14	7	silty sand to sandy silt
40.354	43.35	0.7887	1.820	9.181	17	6	sandy silt to clayey silt
40.518	43.28	1.0050	2.322	8.914	17	6	sandy silt to clayey silt
40.682	39.48	1.4210	3.599	8.716	19	5	clayey silt to silty clay
40.846	27.78	1.3141	4.730	9.801	27	3	clay
41.011	21.18	1.0452	4.934	13.082	20	3	clay
41.175	33.33	0.7799	2.340	14.248	13	6	sandy silt to clayey silt
41.339	40.42	0.7071	1.749	11.173	15	6	sandy silt to clayey silt
41.503	41.52	0.8593	2.069	10.137	16	6	sandy silt to clayey silt
41.667	38.38	1.1017	2.871	10.058	15	6	sandy silt to clayey silt
41.831	33.29	1.2371	3.716	10.264	16	5	clayey silt to silty clay
41.995	22.36	1.0144	4.538	10.502	21	3	clay
42.159	15.63	0.6339	4.054	12.568	15	3	clay
42.323	15.41	0.6141	3.986	15.475	10	4	silty clay to clay
42.487	25.25	0.6582	2.607	18.436	12	5	clayey silt to silty clay
42.651	43.84	0.7223	1.648	15.976	14	7	silty sand to sandy silt
42.815	51.06	0.6680	1.308	11.438	16	7	silty sand to sandy silt
42.979	57.96	0.8185	1.412	10.048	18	7	silty sand to sandy silt
43.143	75.14	0.9963	1.326	9.341	24	7	silty sand to sandy silt
43.307	89.28	0.9896	1.108	8.731	21	8	sand to silty sand
43.471	72.58	0.8790	1.211	7.890	23	7	silty sand to sandy silt
43.635	56.73	0.5343	0.942	7.443	18	7	silty sand to sandy silt
43.799	48.49	0.9325	1.923	6.942	19	6	sandy silt to clayey silt
43.963	38.29	1.0451	2.729	6.345	15	6	sandy silt to clayey silt
44.127	27.33	0.9247	3.383	9.133	13	5	clayey silt to silty clay
44.291	34.63	0.6971	2.013	7.974	13	6	sandy silt to clayey silt
44.455	38.24	0.5393	1.410	6.932	12	7	silty sand to sandy silt
44.619	43.41	0.5894	1.358	6.505	14	7	silty sand to sandy silt
44.783	48.34	0.7063	1.461	6.409	15	7	silty sand to sandy silt
44.948	50.23	0.8235	1.640	6.386	16	7	silty sand to sandy silt

Depth ft	Tip (Qt) (tsf)	Sleeve (Fs) (tsf)	F.Ratio (%)	PP (U2) (psi)	SPT (blows/ft)	Zone	Soil Behavior Type UBC-1983
45.112	51.65	0.9539	1.847	6.515	16	7	silty sand to sandy silt
45.276	51.85	1.0099	1.948	6.640	17	7	silty sand to sandy silt
45.440	49.71	1.0075	2.027	6.790	19	6	sandy silt to clayey silt
45.604	48.06	0.9825	2.044	6.945	18	6	sandy silt to clayey silt
45.768	48.44	1.1676	2.410	7.057	19	6	sandy silt to clayey silt
45.932	37.97	1.2404	3.267	7.209	18	5	clayey silt to silty clay
46.096	23.35	1.1766	5.038	8.193	22	3	clay
46.260	19.85	0.9113	4.592	10.424	19	3	clay
46.424	23.96	1.0086	4.209	11.824	15	4	silty clay to clay
46.588	29.74	1.2651	4.254	11.834	19	4	silty clay to clay
46.752	31.83	1.3481	4.236	11.018	20	4	silty clay to clay
46.916	31.68	1.1144	3.518	10.868	15	5	clayey silt to silty clay
47.080	24.18	0.8226	3.402	10.782	12	5	clayey silt to silty clay
47.244	17.76	0.5785	3.257	12.647	9	5	clayey silt to silty clay
47.408	21.18	0.6963	3.288	26.484	10	5	clayey silt to silty clay
47.572	23.01	0.7631	3.317	31.365	11	5	clayey silt to silty clay
47.736	24.32	1.1489	4.725	34.963	23	3	clay
47.900	26.34	1.3806	5.241	39.253	25	3	clay
48.064	22.41	1.4243	6.354	40.655	21	3	clay
48.228	19.52	1.1262	5.770	41.403	19	3	clay
48.392	19.76	0.8165	4.133	43.959	13	4	silty clay to clay
48.556	18.86	0.8993	4.769	48.175	18	3	clay
48.720	23.77	0.9464	3.981	56.403	15	4	silty clay to clay
48.885	21.75	0.6895	3.170	57.554	10	5	clayey silt to silty clay
49.049	22.10	1.0464	4.735	60.804	21	3	clay
49.213	33.81	1.7034	5.039	71.589	32	3	clay
49.377	39.73	2.2120	5.568	72.811	38	3	clay
49.541	35.26	2.2535	6.391	86.947	34	3	clay
49.705	35.94	1.8498	5.147	100.014	34	3	clay
49.869	35.18	2.0203	5.742	98.875	34	3	clay
50.033	48.71	2.5896	5.316	124.142	47	3	clay
50.197	61.14	4.1465	6.782	121.202	59	11	very stiff fine grained (*)
50.361	75.92	5.6234	7.407	76.793	73	11	very stiff fine grained (*)
50.525	70.94	5.4623	7.700	58.131	68	11	very stiff fine grained (*)
50.689	87.91	4.5849	5.215	34.900	84	11	very stiff fine grained (*)
50.853	97.97	3.7630	3.841	22.087	47	5	clayey silt to silty clay
51.017	112.86	2.8235	2.502	13.366	36	7	silty sand to sandy silt
51.181	113.58	3.3888	2.984	9.306	44	6	sandy silt to clayey silt
51.345	107.53	2.8960	2.693	7.440	41	6	sandy silt to clayey silt
51.509	97.87	3.5454	3.623	6.147	37	6	sandy silt to clayey silt
51.673	59.65	3.4728	5.822	5.964	57	11	very stiff fine grained (*)
51.837	46.53	2.8881	6.207	6.487	45	3	clay
52.001	36.04	2.0385	5.656	8.233	35	3	clay
52.165	43.73	2.1858	4.998	9.387	42	3	clay
52.329	51.61	1.8757	3.634	10.614	25	5	clayey silt to silty clay
52.493	31.44	1.6459	5.236	7.738	30	3	clay
52.657	29.53	1.1859	4.015	7.865	19	4	silty clay to clay
52.822	51.67	1.1064	2.141	7.773	20	6	sandy silt to clayey silt
52.986	56.98	1.0719	1.881	7.115	18	7	silty sand to sandy silt
53.150	56.71	1.2265	2.163	7.372	22	6	sandy silt to clayey silt
53.314	55.13	1.3306	2.414	8.111	21	6	sandy silt to clayey silt
53.478	51.91	1.3460	2.593	8.675	20	6	sandy silt to clayey silt
53.642	41.63	1.1929	2.865	9.349	16	6	sandy silt to clayey silt
53.806	26.56	0.7659	2.884	9.882	13	5	clayey silt to silty clay
53.970	30.57	0.7001	2.290	12.779	12	6	sandy silt to clayey silt
54.134	33.30	0.7062	2.121	12.574	13	6	sandy silt to clayey silt
54.298	33.99	0.6936	2.041	13.191	13	6	sandy silt to clayey silt

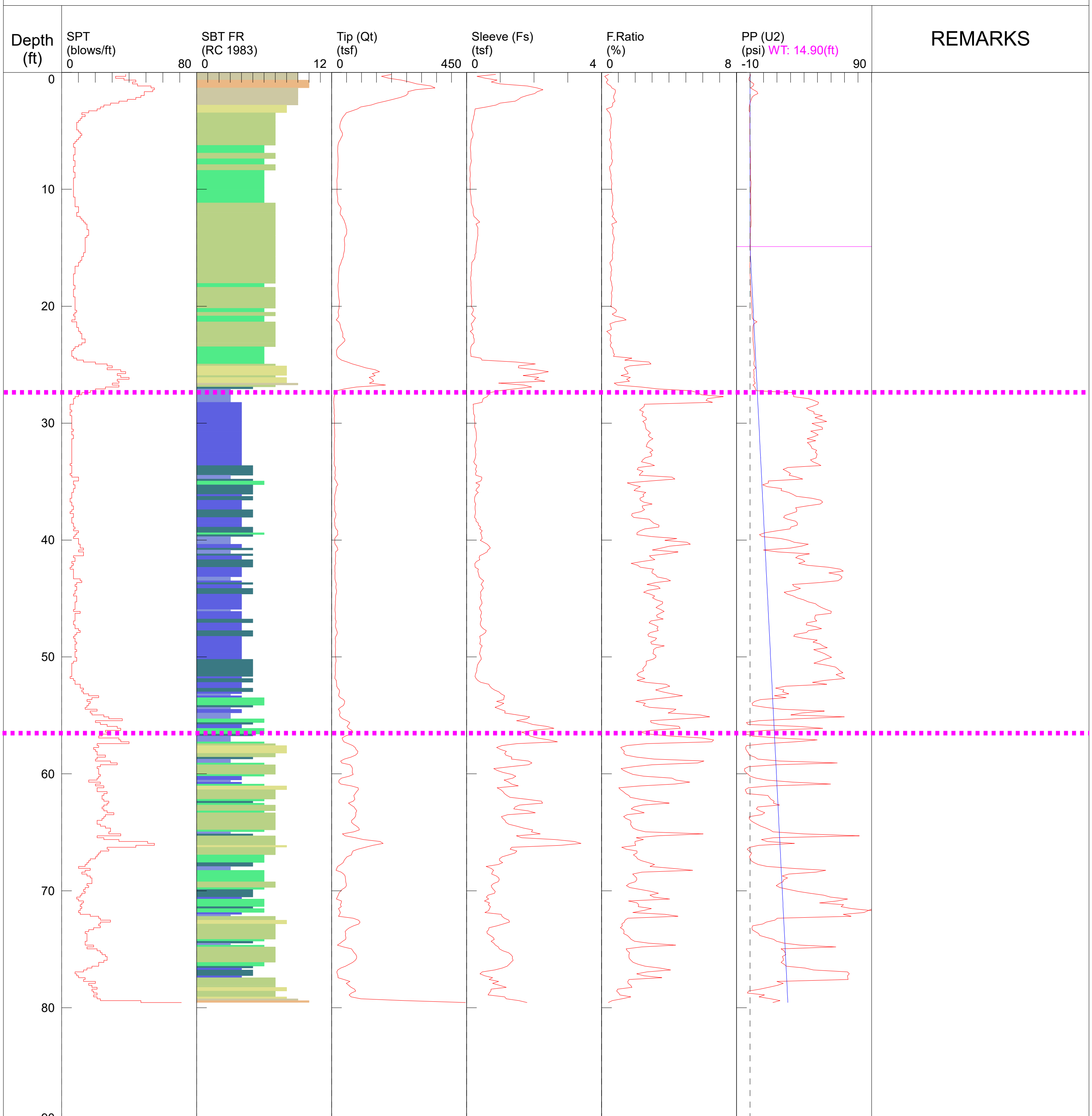
Depth ft	Tip (Qt) (tsf)	Sleeve (Fs) (tsf)	F.Ratio (%)	PP (U2) (psi)	SPT (blows/ft)	Zone	Soil Behavior Type UBC-1983
54.462	33.00	0.7635	2.314	13.920	13	6	sandy silt to clayey silt
54.626	27.44	0.9400	3.425	14.985	13	5	clayey silt to silty clay
54.790	26.46	0.8241	3.114	16.207	13	5	clayey silt to silty clay
54.954	37.73	0.6936	1.838	16.771	14	6	sandy silt to clayey silt
55.118	42.85	0.7355	1.716	15.407	14	7	silty sand to sandy silt
55.282	43.07	0.8583	1.993	15.811	16	6	sandy silt to clayey silt
55.446	34.87	1.0896	3.125	16.555	17	5	clayey silt to silty clay
55.610	24.84	0.8973	3.612	17.498	12	5	clayey silt to silty clay
55.774	18.58	0.6481	3.488	19.097	12	4	silty clay to clay
55.938	14.01	0.4034	2.879	21.391	7	5	clayey silt to silty clay
56.102	13.32	0.2155	1.618	23.678	6	5	clayey silt to silty clay
56.266	13.23	0.1798	1.359	27.068	6	5	clayey silt to silty clay
56.430	12.31	0.2104	1.709	31.594	6	5	clayey silt to silty clay
56.594	12.20	0.2473	2.027	36.671	6	5	clayey silt to silty clay
56.759	20.17	0.3576	1.773	40.000	8	6	sandy silt to clayey silt
56.923	18.62	0.4532	2.435	38.031	9	5	clayey silt to silty clay
57.087	23.65	0.4941	2.089	37.253	9	6	sandy silt to clayey silt
57.251	24.52	0.4080	1.664	32.448	9	6	sandy silt to clayey silt
57.415	24.19	0.5384	2.226	33.594	9	6	sandy silt to clayey silt
57.579	18.85	0.5425	2.878	36.971	9	5	clayey silt to silty clay
57.743	29.18	0.6316	2.165	42.714	11	6	sandy silt to clayey silt
57.907	29.85	0.9004	3.017	32.161	14	5	clayey silt to silty clay
58.071	34.27	0.9229	2.693	38.056	13	6	sandy silt to clayey silt
58.235	60.33	1.0619	1.760	27.249	19	7	silty sand to sandy silt
58.399	44.00	1.1919	2.709	23.879	17	6	sandy silt to clayey silt
58.563	34.76	0.9000	2.589	24.565	13	6	sandy silt to clayey silt
58.727	28.71	0.9450	3.292	24.369	14	5	clayey silt to silty clay
58.891	19.02	0.6789	3.569	23.714	12	4	silty clay to clay
59.055	15.94	0.5558	3.486	26.944	10	4	silty clay to clay
59.219	16.15	0.3737	2.314	31.487	8	5	clayey silt to silty clay
59.383	14.17	0.3016	2.128	36.112	7	5	clayey silt to silty clay
59.547	16.29	0.2374	1.458	43.242	6	6	sandy silt to clayey silt
59.711	19.29	0.2666	1.382	47.234	7	6	sandy silt to clayey silt
59.875	23.58	0.4684	1.987	48.777	9	6	sandy silt to clayey silt
60.039	23.75	0.6349	2.673	50.553	11	5	clayey silt to silty clay
60.203	27.97	0.5844	2.090	42.785	11	6	sandy silt to clayey silt
60.367	23.93	0.6017	2.514	41.207	11	5	clayey silt to silty clay
60.532	23.98	0.6433	2.682	32.956	11	5	clayey silt to silty clay
60.696	44.53	0.7913	1.777	39.179	14	7	silty sand to sandy silt
60.860	68.63	1.0690	1.558	26.402	22	7	silty sand to sandy silt
61.024	88.62	1.6295	1.839	23.038	28	7	silty sand to sandy silt
61.188	84.85	2.2245	2.622	20.642	33	6	sandy silt to clayey silt
61.352	71.68	2.4151	3.369	20.799	27	6	sandy silt to clayey silt
61.516	75.11	1.3914	1.853	17.440	24	7	silty sand to sandy silt
61.680	65.94	1.2355	1.874	12.045	21	7	silty sand to sandy silt
61.844	45.37	1.1666	2.571	11.217	17	6	sandy silt to clayey silt
62.008	36.30	1.2301	3.388	12.020	17	5	clayey silt to silty clay
62.172	37.03	1.3213	3.568	13.587	18	5	clayey silt to silty clay
62.336	39.40	1.2519	3.178	15.999	19	5	clayey silt to silty clay
62.500	51.12	1.1374	2.225	16.850	20	6	sandy silt to clayey silt
62.664	55.52	1.0544	1.899	15.384	18	7	silty sand to sandy silt
62.828	58.10	0.9594	1.651	14.355	19	7	silty sand to sandy silt
62.992	57.06	1.0988	1.925	13.966	18	7	silty sand to sandy silt
63.156	52.87	1.2952	2.450	14.368	20	6	sandy silt to clayey silt
63.320	52.35	1.2539	2.395	14.949	20	6	sandy silt to clayey silt
63.484	58.94	1.0811	1.834	15.300	19	7	silty sand to sandy silt
63.648	53.39	1.0169	1.905	14.073	17	7	silty sand to sandy silt

Depth ft	Tip (Qt) (tsf)	Sleeve (Fs) (tsf)	F.Ratio (%)	PP (U2) (psi)	SPT (blows/ft)	Zone	Soil Behavior Type UBC-1983
63.812	29.10	1.0433	3.585	20.029	14	5	clayey silt to silty clay
63.976	24.95	1.1419	4.577	23.261	24	3	clay
64.140	44.91	1.2177	2.712	27.790	17	6	sandy silt to clayey silt
64.304	65.43	1.3141	2.008	21.010	21	7	silty sand to sandy silt
64.469	74.10	1.1595	1.565	16.182	24	7	silty sand to sandy silt
64.633	75.94	1.2300	1.620	14.619	24	7	silty sand to sandy silt
64.797	71.04	1.3246	1.865	13.333	23	7	silty sand to sandy silt
64.961	69.41	1.4653	2.111	14.182	22	7	silty sand to sandy silt
65.125	72.02	1.5402	2.138	14.604	23	7	silty sand to sandy silt
65.289	76.70	1.6965	2.212	15.046	24	7	silty sand to sandy silt
65.453	83.74	2.0112	2.402	15.801	32	6	sandy silt to clayey silt
65.617	85.64	2.3340	2.725	16.492	33	6	sandy silt to clayey silt
65.781	72.80	2.2969	3.155	16.411	28	6	sandy silt to clayey silt
65.945	72.77	1.6702	2.295	15.572	28	6	sandy silt to clayey silt
66.109	85.21	1.5381	1.805	10.271	27	7	silty sand to sandy silt
66.273	80.76	1.3272	1.643	9.156	26	7	silty sand to sandy silt
66.437	74.37	1.3829	1.859	9.473	24	7	silty sand to sandy silt
66.601	69.70	1.4433	2.071	10.353	22	7	silty sand to sandy silt
66.765	66.39	1.3595	2.048	11.247	21	7	silty sand to sandy silt
66.929	64.92	1.3310	2.050	12.108	21	7	silty sand to sandy silt
67.093	62.72	1.2930	2.061	16.855	20	7	silty sand to sandy silt
67.257	60.12	1.2705	2.113	18.060	23	6	sandy silt to clayey silt
67.421	59.52	1.3008	2.185	19.056	23	6	sandy silt to clayey silt
67.585	58.93	1.3414	2.276	20.845	23	6	sandy silt to clayey silt
67.749	58.67	1.3677	2.331	21.356	22	6	sandy silt to clayey silt
67.913	57.58	1.4003	2.432	21.930	22	6	sandy silt to clayey silt
68.077	55.70	1.4366	2.579	22.568	21	6	sandy silt to clayey silt
68.241	52.82	1.4080	2.666	23.221	20	6	sandy silt to clayey silt
68.406	53.62	1.3313	2.483	23.495	21	6	sandy silt to clayey silt
68.570	53.73	1.3104	2.439	23.269	21	6	sandy silt to clayey silt
68.734	52.29	1.5115	2.891	23.803	20	6	sandy silt to clayey silt
68.898	55.45	1.4626	2.638	24.583	21	6	sandy silt to clayey silt
69.062	67.99	1.5035	2.211	22.380	26	6	sandy silt to clayey silt
69.226	52.98	1.5635	2.951	20.034	20	6	sandy silt to clayey silt
69.390	38.41	1.3726	3.574	21.640	18	5	clayey silt to silty clay
69.554	36.53	1.3121	3.592	23.993	17	5	clayey silt to silty clay
69.718	37.68	1.2463	3.307	25.002	18	5	clayey silt to silty clay
69.882	39.84	1.2075	3.031	25.716	19	5	clayey silt to silty clay
70.046	42.67	1.4674	3.439	26.669	20	5	clayey silt to silty clay
70.210	45.34	1.5066	3.323	27.942	22	5	clayey silt to silty clay
70.374	86.70	1.7639	2.034	22.934	28	7	silty sand to sandy silt
70.538	99.02	2.1002	2.121	18.197	32	7	silty sand to sandy silt
70.702	101.00	2.7297	2.703	17.236	39	6	sandy silt to clayey silt
70.866	94.85	3.1745	3.347	16.967	36	6	sandy silt to clayey silt
71.030	96.46	2.5081	2.600	15.420	37	6	sandy silt to clayey silt
71.194	99.54	2.0472	2.057	11.374	32	7	silty sand to sandy silt
71.358	89.74	2.1880	2.438	9.367	29	7	silty sand to sandy silt
71.522	56.05	2.1474	3.831	8.437	27	5	clayey silt to silty clay
71.686	43.05	1.7581	4.084	9.326	21	5	clayey silt to silty clay
71.850	35.38	1.7173	4.854	11.158	34	3	clay
72.014	58.33	1.4018	2.403	13.282	22	6	sandy silt to clayey silt
72.178	97.78	1.4529	1.486	11.473	31	7	silty sand to sandy silt
72.343	100.00	1.7493	1.749	9.547	32	7	silty sand to sandy silt
72.507	76.73	2.1301	2.776	9.087	29	6	sandy silt to clayey silt
72.671	64.01	2.6181	4.090	10.238	31	5	clayey silt to silty clay
72.835	56.98	2.2212	3.898	11.636	27	5	clayey silt to silty clay
72.999	75.65	1.3225	1.748	11.509	24	7	silty sand to sandy silt

Depth ft	Tip (Qt) (tsf)	Sleeve (Fs) (tsf)	F.Ratio (%)	PP (U2) (psi)	SPT (blows/ft)	Zone	Soil Behavior Type UBC-1983
73.163	80.36	1.0927	1.360	9.184	26	7	silty sand to sandy silt
73.327	59.35	1.2765	2.151	8.111	23	6	sandy silt to clayey silt
73.491	44.67	1.4256	3.191	9.041	21	5	clayey silt to silty clay
73.655	35.54	1.4213	4.000	15.785	23	4	silty clay to clay
73.819	37.77	1.2808	3.391	21.137	18	5	clayey silt to silty clay
73.983	25.84	1.2601	4.877	20.756	25	3	clay
74.147	39.11	1.3567	3.469	23.119	19	5	clayey silt to silty clay
74.311	54.54	1.4420	2.644	22.400	21	6	sandy silt to clayey silt
74.475	50.72	1.3965	2.754	21.211	19	6	sandy silt to clayey silt
74.639	35.06	1.2218	3.485	19.341	17	5	clayey silt to silty clay
74.803	25.49	0.9589	3.763	22.047	16	4	silty clay to clay
74.967	30.63	0.8013	2.616	27.193	12	6	sandy silt to clayey silt
75.131	39.55	0.7239	1.830	28.694	15	6	sandy silt to clayey silt
75.295	27.93	0.5956	2.133	28.407	11	6	sandy silt to clayey silt
75.459	20.91	0.4625	2.211	31.632	10	5	clayey silt to silty clay
75.623	16.98	0.3733	2.198	36.724	8	5	clayey silt to silty clay
75.787	16.04	0.4162	2.595	40.689	8	5	clayey silt to silty clay
75.951	17.49	0.6171	3.529	45.209	11	4	silty clay to clay
76.115	33.06	0.7830	2.369	46.653	13	6	sandy silt to clayey silt
76.280	301.30	0.8491	0.282	45.174	48	10	gravelly sand to sand
76.444	500.81	0.9202	0.184	34.920	80	10	gravelly sand to sand
76.608	581.31	0.9902	0.170	30.903	93	10	gravelly sand to sand

GeoEngineers / CPT-4a2 / 5528 Doane Ave Portland

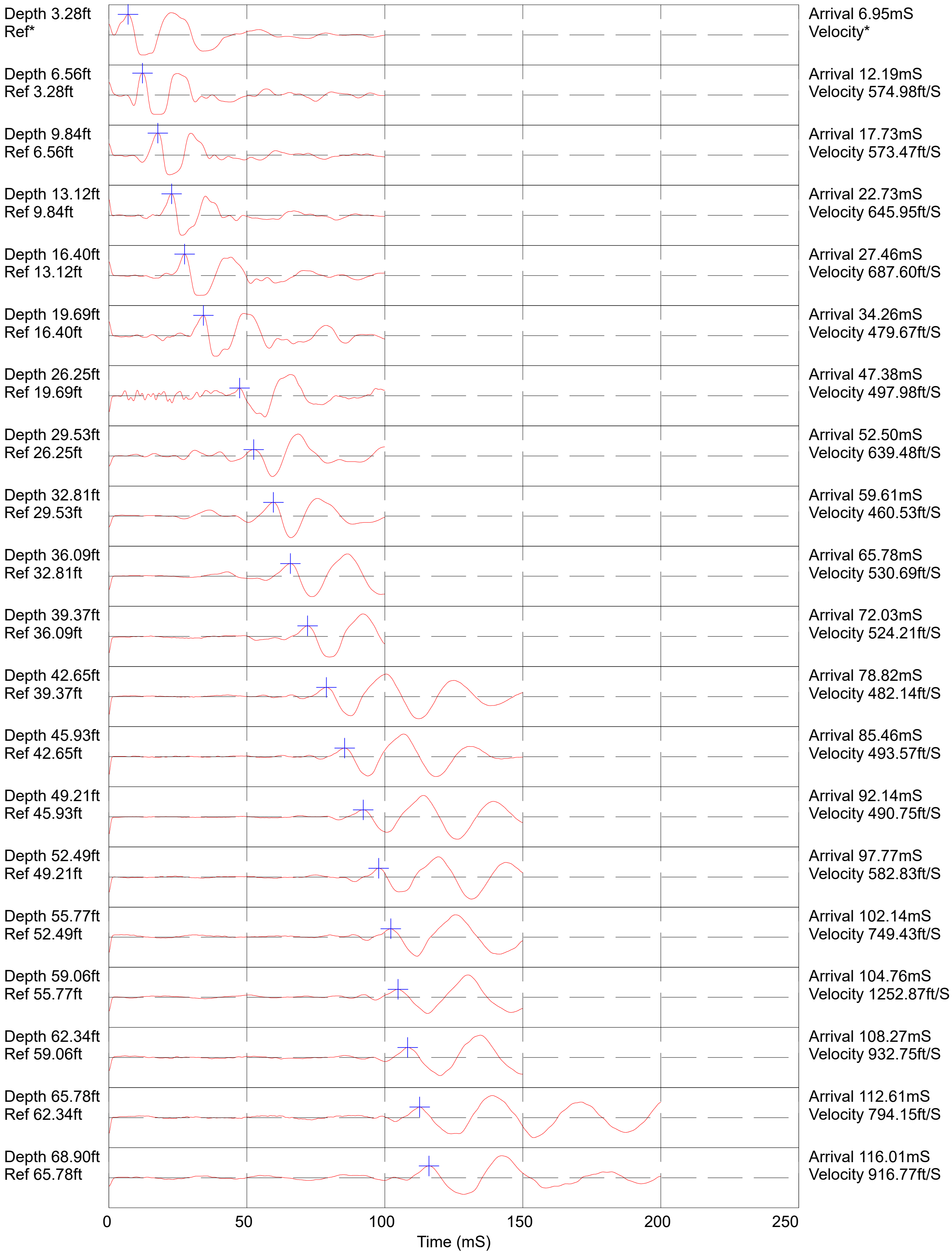
OPERATOR: OGE BAK
CONE ID: DDG1661
TEST DATE: 2/7/2024 11:01:47 AM
TOTAL DEPTH: 79.560 ft



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 (*)

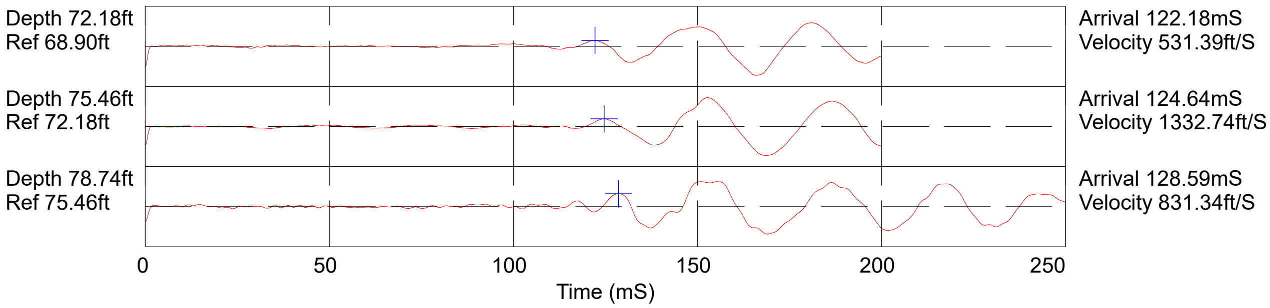
*SBT/SPT CORRELATION: UBC-1983

COMMENT: GeoEngineers / CPT-4a2 / 5528 Doane Ave Portland



Hammer to Rod String Distance (ft): 2.03
* = Not Determined

COMMENT: GeoEngineers / CPT-4a2 / 5528 Doane Ave Portland



Hammer to Rod String Distance (ft): 2.03
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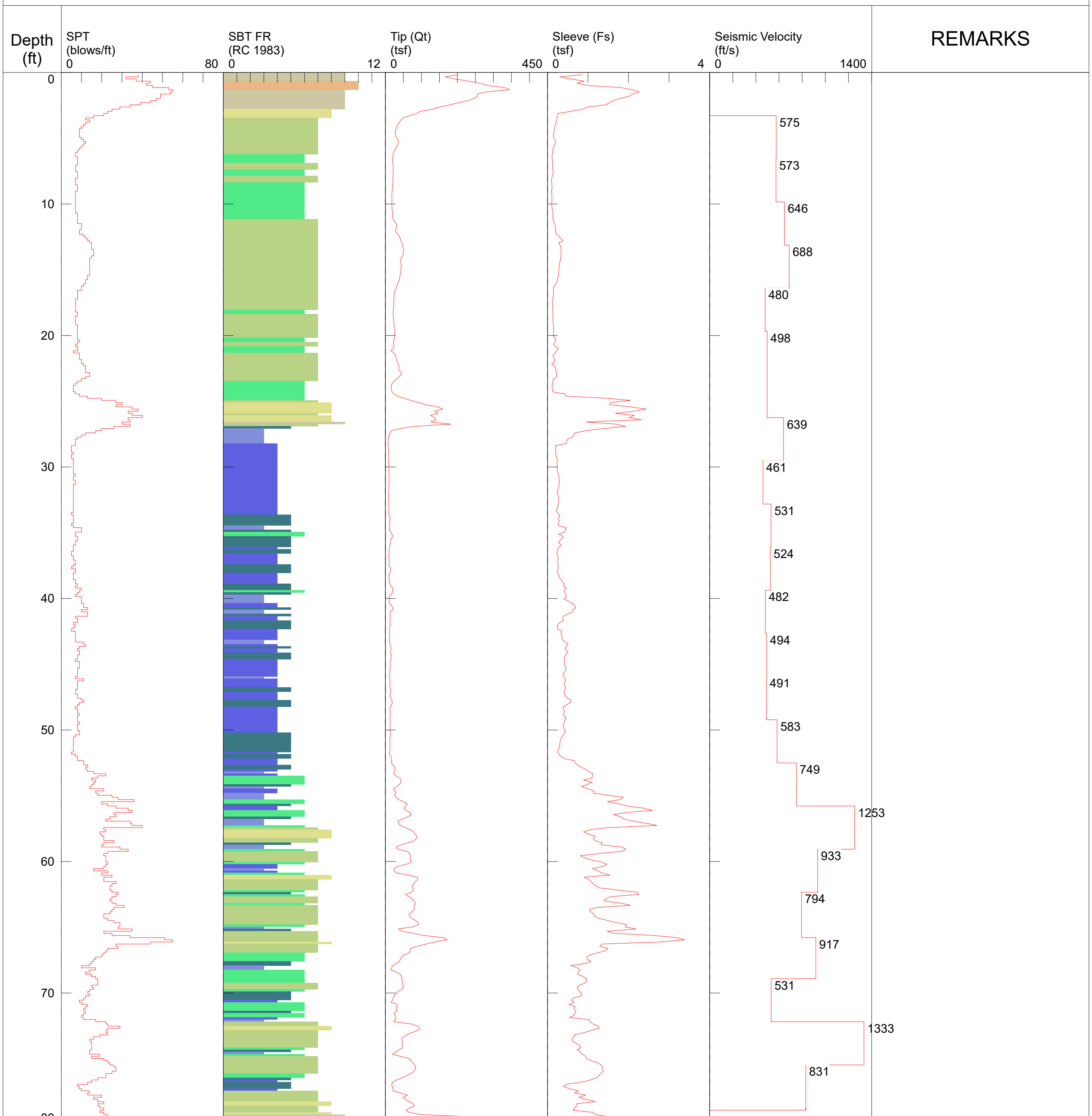

1 sensitive fine grained
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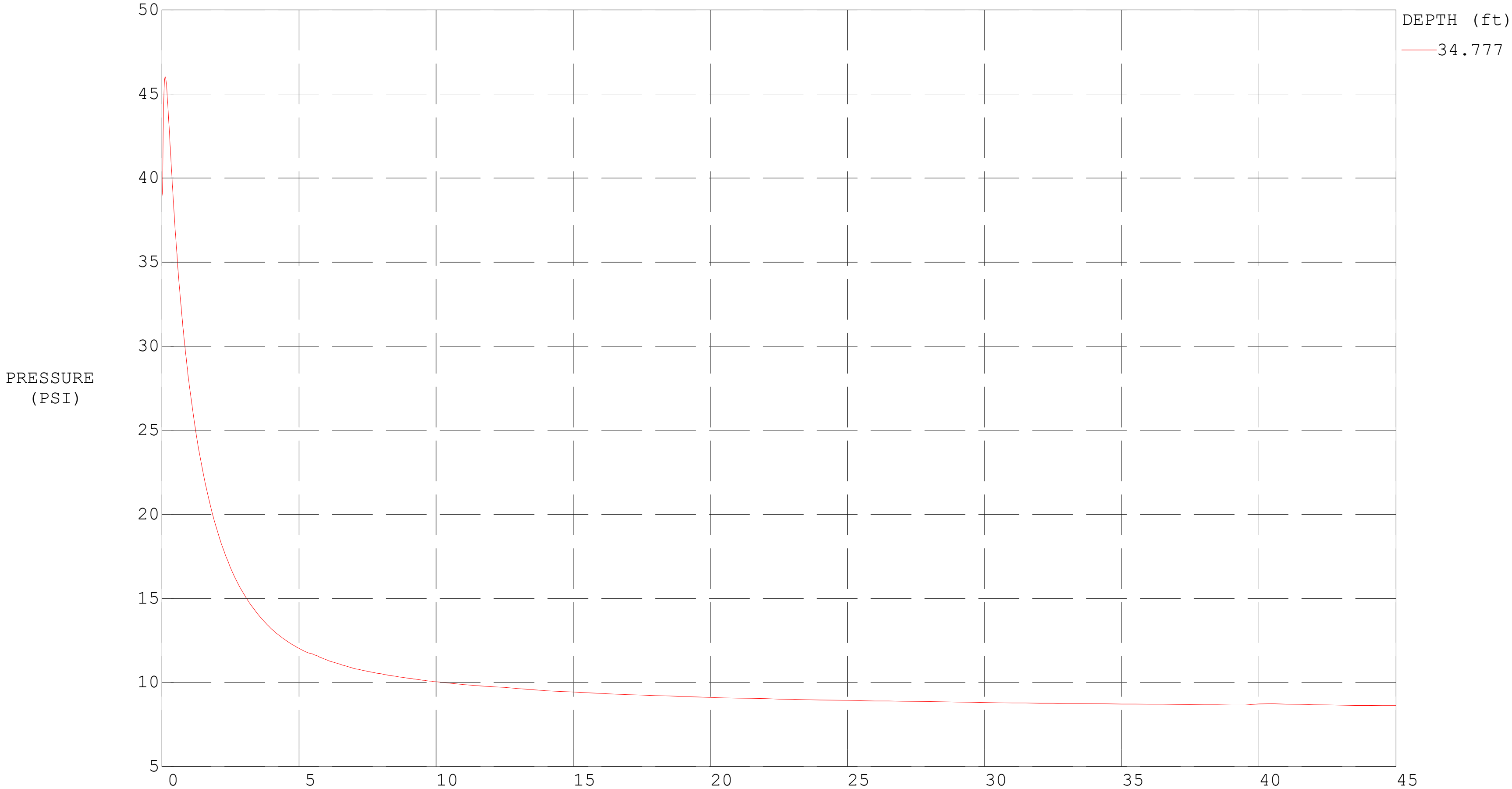

MAXIMUM PRESSURE = 46.028 (PSI)
HYDROSTATIC PRESSURE = 8.617 (PSI), WATER TABLE: 14.90 ft

GeoEngineers / CPT-4a2 / 5528 Doane Ave Portland

OPERATOR: OGE BAK
CONE ID: DDG1661
TEST DATE: 2/7/2024 11:01:47 AM
TOTAL DEPTH: 79.560 ft

Depth ft	Tip (Qt) (tsf)	Sleeve (Fs) (tsf)	F.Ratio (%)	PP (U2) (psi)	SPT (blows/ft)	Zone	Soil Behavior Type UBC-1983
0.164	199.73	0.8613	0.431	0.371	38	9	sand
0.328	166.36	0.3059	0.184	0.086	32	9	sand
0.492	190.86	0.5612	0.294	-0.873	37	9	sand
0.656	231.70	0.9024	0.389	-0.894	44	9	sand
0.820	260.74	0.7320	0.281	0.885	42	10	gravelly sand to sand
0.984	284.11	1.0038	0.353	2.817	45	10	gravelly sand to sand
1.148	331.16	1.7978	0.543	2.255	53	10	gravelly sand to sand
1.312	344.55	2.0889	0.606	0.435	55	10	gravelly sand to sand
1.476	280.09	2.2598	0.807	2.506	54	9	sand
1.640	254.58	2.0971	0.824	5.038	49	9	sand
1.804	255.02	2.0156	0.790	5.780	49	9	sand
1.969	247.33	1.7909	0.724	2.838	47	9	sand
2.133	229.09	1.6139	0.704	1.401	44	9	sand
2.297	204.28	1.5364	0.752	0.964	39	9	sand
2.461	179.47	1.4373	0.801	0.433	34	9	sand
2.625	151.29	0.9944	0.657	-0.019	29	9	sand
2.789	131.98	0.8714	0.660	-0.390	25	9	sand
2.953	96.77	0.5861	0.606	-0.686	23	8	sand to silty sand
3.117	86.51	0.2528	0.292	-0.679	21	8	sand to silty sand
3.281	68.50	0.2445	0.357	-0.751	16	8	sand to silty sand
3.445	49.22	0.2337	0.475	-0.055	12	8	sand to silty sand
3.609	44.46	0.2286	0.514	-0.110	14	7	silty sand to sandy silt
3.773	38.80	0.2092	0.539	-0.167	12	7	silty sand to sandy silt
3.937	34.89	0.2023	0.580	-0.213	11	7	silty sand to sandy silt
4.101	32.17	0.1848	0.574	-0.230	10	7	silty sand to sandy silt
4.265	29.68	0.1718	0.579	-0.215	9	7	silty sand to sandy silt
4.429	28.04	0.1582	0.564	-0.213	9	7	silty sand to sandy silt
4.593	27.34	0.1491	0.546	-0.213	9	7	silty sand to sandy silt
4.757	27.98	0.1509	0.539	-0.213	9	7	silty sand to sandy silt
4.921	30.05	0.1475	0.491	-0.210	10	7	silty sand to sandy silt
5.085	34.21	0.1767	0.517	-0.194	11	7	silty sand to sandy silt
5.249	36.61	0.1941	0.530	-0.177	12	7	silty sand to sandy silt
5.413	35.86	0.1895	0.528	-0.196	11	7	silty sand to sandy silt
5.577	31.98	0.1690	0.529	-0.220	10	7	silty sand to sandy silt
5.741	28.74	0.1342	0.467	-0.232	9	7	silty sand to sandy silt
5.906	25.30	0.1312	0.519	-0.210	8	7	silty sand to sandy silt
6.070	22.54	0.1265	0.561	-0.019	7	7	silty sand to sandy silt
6.234	21.36	0.1206	0.565	-0.038	7	7	silty sand to sandy silt
6.398	20.32	0.1103	0.543	-0.045	8	6	sandy silt to clayey silt
6.562	20.29	0.1146	0.565	-0.041	8	6	sandy silt to clayey silt
6.726	19.96	0.1217	0.610	0.275	8	6	sandy silt to clayey silt
6.890	20.50	0.1246	0.608	0.275	8	6	sandy silt to clayey silt
7.054	21.23	0.1261	0.594	0.289	7	7	silty sand to sandy silt
7.382	21.79	0.1290	0.592	0.316	7	7	silty sand to sandy silt
7.546	21.73	0.1425	0.656	0.323	8	6	sandy silt to clayey silt
7.710	21.30	0.1304	0.612	0.323	8	6	sandy silt to clayey silt

Depth ft	Tip (Qt) (tsf)	Sleeve (Fs) (tsf)	F.Ratio (%)	PP (U2) (psi)	SPT (blows/ft)	Zone	Soil Behavior Type UBC-1983
7.874	20.69	0.1206	0.583	0.325	8	6	sandy silt to clayey silt
8.038	21.42	0.1039	0.485	0.366	7	7	silty sand to sandy silt
8.202	21.00	0.0996	0.474	0.344	7	7	silty sand to sandy silt
8.366	20.52	0.1022	0.498	0.366	7	7	silty sand to sandy silt
8.530	20.22	0.1067	0.528	0.363	8	6	sandy silt to clayey silt
8.694	19.99	0.1111	0.556	0.371	8	6	sandy silt to clayey silt
8.858	19.66	0.1100	0.560	0.375	8	6	sandy silt to clayey silt
9.022	18.41	0.0996	0.541	0.380	7	6	sandy silt to clayey silt
9.186	17.88	0.1025	0.573	0.397	7	6	sandy silt to clayey silt
9.350	17.87	0.1057	0.591	0.665	7	6	sandy silt to clayey silt
9.514	18.07	0.1108	0.613	0.624	7	6	sandy silt to clayey silt
9.678	18.11	0.1072	0.592	0.605	7	6	sandy silt to clayey silt
9.843	18.22	0.1076	0.590	0.581	7	6	sandy silt to clayey silt
10.007	18.08	0.1066	0.590	0.473	7	6	sandy silt to clayey silt
10.171	17.83	0.1077	0.604	0.462	7	6	sandy silt to clayey silt
10.335	18.22	0.1134	0.622	0.471	7	6	sandy silt to clayey silt
10.499	19.07	0.1259	0.660	0.478	7	6	sandy silt to clayey silt
10.663	20.20	0.1320	0.653	0.490	8	6	sandy silt to clayey silt
10.827	20.31	0.1344	0.662	0.493	8	6	sandy silt to clayey silt
10.991	20.15	0.1344	0.667	0.488	8	6	sandy silt to clayey silt
11.155	21.48	0.1371	0.638	0.495	8	6	sandy silt to clayey silt
11.319	24.26	0.1545	0.637	0.514	8	7	silty sand to sandy silt
11.483	29.81	0.1743	0.585	0.552	10	7	silty sand to sandy silt
11.647	31.77	0.1925	0.606	0.560	10	7	silty sand to sandy silt
11.811	30.28	0.1967	0.649	0.567	10	7	silty sand to sandy silt
11.975	28.47	0.1939	0.681	0.562	9	7	silty sand to sandy silt
12.139	29.12	0.2026	0.696	0.550	9	7	silty sand to sandy silt
12.303	35.05	0.2180	0.622	0.572	11	7	silty sand to sandy silt
12.467	37.68	0.2813	0.747	0.583	12	7	silty sand to sandy silt
12.631	41.61	0.3326	0.799	0.617	13	7	silty sand to sandy silt
12.795	43.41	0.3906	0.900	0.811	14	7	silty sand to sandy silt
12.959	45.86	0.2773	0.605	0.748	15	7	silty sand to sandy silt
13.123	47.96	0.3066	0.639	0.689	15	7	silty sand to sandy silt
13.287	48.21	0.3354	0.696	0.239	15	7	silty sand to sandy silt
13.451	50.44	0.3350	0.664	0.210	16	7	silty sand to sandy silt
13.615	50.38	0.3334	0.662	0.222	16	7	silty sand to sandy silt
13.780	50.38	0.3295	0.654	0.201	16	7	silty sand to sandy silt
13.944	47.53	0.3375	0.710	0.175	15	7	silty sand to sandy silt
14.108	43.58	0.3215	0.738	0.158	14	7	silty sand to sandy silt
14.272	42.38	0.3178	0.750	0.215	14	7	silty sand to sandy silt
14.436	42.52	0.3049	0.717	0.196	14	7	silty sand to sandy silt
14.600	43.45	0.2992	0.689	0.189	14	7	silty sand to sandy silt
14.764	44.03	0.2864	0.650	0.213	14	7	silty sand to sandy silt
14.928	42.87	0.2827	0.659	0.198	14	7	silty sand to sandy silt
15.092	42.92	0.2795	0.651	0.187	14	7	silty sand to sandy silt
15.256	42.46	0.2742	0.646	0.165	14	7	silty sand to sandy silt
15.420	40.81	0.2773	0.680	0.132	13	7	silty sand to sandy silt
15.584	39.49	0.2482	0.628	0.112	13	7	silty sand to sandy silt
15.748	38.71	0.2489	0.643	0.091	12	7	silty sand to sandy silt
15.912	36.68	0.2395	0.653	0.014	12	7	silty sand to sandy silt
16.076	33.57	0.2033	0.606	-0.022	11	7	silty sand to sandy silt
16.240	31.74	0.1583	0.499	-0.053	10	7	silty sand to sandy silt
16.404	29.98	0.1520	0.507	-0.086	10	7	silty sand to sandy silt
16.568	26.16	0.1467	0.561	-0.146	8	7	silty sand to sandy silt
16.732	24.80	0.1436	0.579	-0.127	8	7	silty sand to sandy silt
16.896	24.30	0.1478	0.608	-0.105	8	7	silty sand to sandy silt
17.060	23.86	0.1327	0.556	-0.077	8	7	silty sand to sandy silt

Depth ft	Tip (Qt) (tsf)	Sleeve (Fs) (tsf)	F.Ratio (%)	PP (U2) (psi)	SPT (blows/ft)	Zone	Soil Behavior Type UBC-1983
17.224	23.46	0.1331	0.567	-0.002	7	7	silty sand to sandy silt
17.388	22.99	0.1306	0.568	0.045	7	7	silty sand to sandy silt
17.552	23.10	0.1203	0.521	0.100	7	7	silty sand to sandy silt
17.717	23.25	0.1235	0.531	0.153	7	7	silty sand to sandy silt
17.881	22.62	0.1295	0.573	0.213	7	7	silty sand to sandy silt
18.045	21.71	0.1307	0.602	0.273	7	7	silty sand to sandy silt
18.209	21.20	0.1287	0.607	0.349	8	6	sandy silt to clayey silt
18.373	21.34	0.1371	0.642	0.402	8	6	sandy silt to clayey silt
18.537	21.72	0.1367	0.629	0.442	7	7	silty sand to sandy silt
18.701	22.06	0.1363	0.618	0.531	7	7	silty sand to sandy silt
18.865	23.01	0.1322	0.575	0.619	7	7	silty sand to sandy silt
19.029	23.10	0.1397	0.605	0.713	7	7	silty sand to sandy silt
19.193	23.69	0.1439	0.608	0.854	8	7	silty sand to sandy silt
19.357	24.62	0.1449	0.589	0.887	8	7	silty sand to sandy silt
19.521	25.83	0.1518	0.588	0.940	8	7	silty sand to sandy silt
19.685	26.26	0.1517	0.578	1.016	8	7	silty sand to sandy silt
19.849	25.33	0.1482	0.585	1.112	8	7	silty sand to sandy silt
20.013	25.05	0.1349	0.538	1.212	8	7	silty sand to sandy silt
20.177	24.99	0.1890	0.757	1.308	8	7	silty sand to sandy silt
20.341	22.75	0.2011	0.884	1.449	9	6	sandy silt to clayey silt
20.505	21.67	0.1830	0.845	1.578	8	6	sandy silt to clayey silt
20.669	23.25	0.1456	0.626	1.387	7	7	silty sand to sandy silt
20.833	24.08	0.1916	0.796	1.497	8	7	silty sand to sandy silt
20.997	21.04	0.2665	1.267	1.638	8	6	sandy silt to clayey silt
21.161	15.45	0.2256	1.460	2.678	6	6	sandy silt to clayey silt
21.325	23.94	0.1946	0.813	5.172	9	6	sandy silt to clayey silt
21.490	27.96	0.1382	0.494	2.786	9	7	silty sand to sandy silt
21.654	29.19	0.1529	0.524	2.300	9	7	silty sand to sandy silt
21.818	30.04	0.1706	0.568	2.150	10	7	silty sand to sandy silt
21.982	32.10	0.1878	0.585	2.119	10	7	silty sand to sandy silt
22.146	35.22	0.1071	0.304	2.128	11	7	silty sand to sandy silt
22.310	37.67	0.1653	0.439	2.169	12	7	silty sand to sandy silt
22.474	37.47	0.2083	0.556	2.190	12	7	silty sand to sandy silt
22.638	37.18	0.2003	0.539	2.205	12	7	silty sand to sandy silt
22.802	43.80	0.2231	0.509	2.119	14	7	silty sand to sandy silt
22.966	44.36	0.2270	0.512	2.238	14	7	silty sand to sandy silt
23.130	37.81	0.2206	0.583	2.363	12	7	silty sand to sandy silt
23.294	31.09	0.1591	0.512	2.387	10	7	silty sand to sandy silt
23.458	24.14	0.1292	0.535	2.623	8	7	silty sand to sandy silt
23.622	18.97	0.1229	0.648	2.752	7	6	sandy silt to clayey silt
23.786	16.53	0.1135	0.687	2.932	6	6	sandy silt to clayey silt
23.950	16.32	0.1128	0.691	3.039	6	6	sandy silt to clayey silt
24.114	16.79	0.1277	0.760	3.082	6	6	sandy silt to clayey silt
24.278	18.36	0.1292	0.704	3.142	7	6	sandy silt to clayey silt
24.442	23.76	0.4268	1.796	3.207	9	6	sandy silt to clayey silt
24.606	32.88	0.4533	1.379	3.243	13	6	sandy silt to clayey silt
24.770	52.11	1.4625	2.807	2.846	20	6	sandy silt to clayey silt
24.934	69.55	2.0384	2.931	2.738	27	6	sandy silt to clayey silt
25.098	92.43	1.5433	1.670	3.123	30	7	silty sand to sandy silt
25.262	113.42	1.5348	1.353	4.297	27	8	sand to silty sand
25.427	145.22	2.0449	1.408	2.587	35	8	sand to silty sand
25.591	159.42	2.4233	1.520	2.989	38	8	sand to silty sand
25.755	135.95	2.1430	1.576	2.805	33	8	sand to silty sand
25.919	145.04	1.6776	1.157	3.058	35	8	sand to silty sand
26.083	125.14	2.1368	1.707	2.764	40	7	silty sand to sandy silt
26.247	136.00	2.0160	1.482	3.195	33	8	sand to silty sand
26.411	140.44	2.3170	1.650	2.850	34	8	sand to silty sand

Depth ft	Tip (Qt) (tsf)	Sleeve (Fs) (tsf)	F.Ratio (%)	PP (U2) (psi)	SPT (blows/ft)	Zone	Soil Behavior Type UBC-1983
26.575	126.46	0.9498	0.751	3.893	30	8	sand to silty sand
26.739	180.04	1.7005	0.945	2.126	34	9	sand
26.903	82.96	1.9310	2.328	3.530	26	7	silty sand to sandy silt
27.067	42.25	1.4113	3.340	3.948	20	5	clayey silt to silty clay
27.231	18.27	0.9683	5.299	4.818	17	3	clay
27.395	12.01	0.6807	5.667	32.201	12	3	clay
27.559	10.95	0.6486	5.926	32.232	10	3	clay
27.723	8.44	0.6091	7.216	32.191	8	3	clay
27.887	7.78	0.4896	6.292	42.995	7	3	clay
28.051	7.66	0.4743	6.188	47.106	7	3	clay
28.215	6.94	0.4557	6.565	50.757	7	3	clay
28.379	7.59	0.1926	2.539	50.148	5	4	silty clay to clay
28.543	8.00	0.2023	2.529	46.422	5	4	silty clay to clay
28.707	8.13	0.1952	2.401	41.778	5	4	silty clay to clay
28.871	9.00	0.2023	2.249	43.694	6	4	silty clay to clay
29.035	8.43	0.2090	2.480	47.357	5	4	silty clay to clay
29.199	8.51	0.2056	2.415	47.364	5	4	silty clay to clay
29.364	9.94	0.2394	2.408	53.572	6	4	silty clay to clay
29.528	9.04	0.2375	2.626	48.039	6	4	silty clay to clay
29.692	8.64	0.2296	2.657	52.587	6	4	silty clay to clay
29.856	9.72	0.2374	2.443	56.683	6	4	silty clay to clay
30.020	9.72	0.2498	2.571	45.946	6	4	silty clay to clay
30.184	9.25	0.2390	2.584	49.325	6	4	silty clay to clay
30.348	9.31	0.2437	2.617	53.457	6	4	silty clay to clay
30.512	10.19	0.2689	2.640	53.665	7	4	silty clay to clay
30.676	10.08	0.2808	2.786	44.966	6	4	silty clay to clay
30.840	9.90	0.2871	2.900	48.115	6	4	silty clay to clay
31.004	10.35	0.2863	2.766	50.466	7	4	silty clay to clay
31.168	10.32	0.2925	2.835	46.919	7	4	silty clay to clay
31.332	9.44	0.2875	3.045	42.223	6	4	silty clay to clay
31.496	9.93	0.2819	2.839	48.634	6	4	silty clay to clay
31.660	9.51	0.2601	2.736	42.409	6	4	silty clay to clay
31.824	9.19	0.2469	2.686	44.160	6	4	silty clay to clay
31.988	9.11	0.2411	2.647	45.241	6	4	silty clay to clay
32.152	9.01	0.2612	2.898	45.855	6	4	silty clay to clay
32.316	8.91	0.2712	3.044	49.318	6	4	silty clay to clay
32.480	9.26	0.2648	2.859	48.309	6	4	silty clay to clay
32.644	9.18	0.2729	2.974	49.662	6	4	silty clay to clay
32.808	9.07	0.2688	2.965	49.265	6	4	silty clay to clay
32.972	9.91	0.2558	2.582	48.947	6	4	silty clay to clay
33.136	9.71	0.2371	2.441	44.707	6	4	silty clay to clay
33.301	9.00	0.2127	2.363	46.549	6	4	silty clay to clay
33.465	8.29	0.2376	2.867	50.019	5	4	silty clay to clay
33.629	9.60	0.3009	3.136	52.482	6	4	silty clay to clay
33.793	13.10	0.2867	2.189	28.031	6	5	clayey silt to silty clay
33.957	12.44	0.2626	2.111	24.439	6	5	clayey silt to silty clay
34.121	11.89	0.2925	2.461	27.273	6	5	clayey silt to silty clay
34.285	11.38	0.2642	2.321	29.936	5	5	clayey silt to silty clay
34.449	12.24	0.2730	2.230	28.966	6	5	clayey silt to silty clay
34.613	10.78	0.4498	4.174	34.602	10	3	clay
34.777	10.37	0.4503	4.341	39.023	10	3	clay
34.941	14.66	0.4219	2.877	13.446	7	5	clayey silt to silty clay
35.105	17.92	0.2738	1.528	13.673	7	6	sandy silt to clayey silt
35.269	20.86	0.3854	1.848	9.328	8	6	sandy silt to clayey silt
35.433	15.78	0.3625	2.297	12.774	8	5	clayey silt to silty clay
35.597	14.69	0.3025	2.059	23.372	7	5	clayey silt to silty clay
35.761	14.35	0.2752	1.918	22.916	7	5	clayey silt to silty clay

Depth ft	Tip (Qt) (tsf)	Sleeve (Fs) (tsf)	F.Ratio (%)	PP (U2) (psi)	SPT (blows/ft)	Zone	Soil Behavior Type UBC-1983
35.925	13.14	0.3442	2.619	28.571	6	5	clayey silt to silty clay
36.089	11.54	0.2834	2.455	31.720	6	5	clayey silt to silty clay
36.253	10.04	0.2517	2.507	34.322	6	4	silty clay to clay
36.417	10.31	0.2339	2.270	45.642	5	5	clayey silt to silty clay
36.581	10.28	0.2356	2.293	52.250	5	5	clayey silt to silty clay
36.745	9.74	0.2426	2.491	53.718	6	4	silty clay to clay
36.909	9.07	0.2717	2.996	51.922	6	4	silty clay to clay
37.073	10.55	0.2537	2.404	42.048	7	4	silty clay to clay
37.238	10.27	0.2496	2.431	33.753	7	4	silty clay to clay
37.402	9.96	0.2536	2.547	34.633	6	4	silty clay to clay
37.566	11.30	0.2639	2.334	34.451	5	5	clayey silt to silty clay
37.730	14.47	0.2623	1.813	30.255	7	5	clayey silt to silty clay
37.894	14.00	0.2485	1.775	25.864	7	5	clayey silt to silty clay
38.058	12.36	0.2356	1.906	24.841	6	5	clayey silt to silty clay
38.222	10.01	0.2825	2.822	27.837	6	4	silty clay to clay
38.386	9.47	0.2793	2.948	33.636	6	4	silty clay to clay
38.550	11.16	0.3444	3.087	35.090	7	4	silty clay to clay
38.714	10.34	0.3515	3.400	34.898	7	4	silty clay to clay
38.878	12.31	0.4183	3.398	30.245	8	4	silty clay to clay
39.042	15.08	0.3862	2.561	29.621	7	5	clayey silt to silty clay
39.206	20.42	0.4651	2.277	19.805	10	5	clayey silt to silty clay
39.370	19.78	0.4148	2.097	11.174	9	5	clayey silt to silty clay
39.534	20.77	0.4343	2.091	6.975	8	6	sandy silt to clayey silt
39.698	15.06	0.4498	2.987	8.977	7	5	clayey silt to silty clay
39.862	10.45	0.4661	4.460	22.674	10	3	clay
40.026	10.87	0.4081	3.753	29.033	10	3	clay
40.190	10.05	0.5082	5.055	32.636	10	3	clay
40.354	11.61	0.6102	5.257	43.144	11	3	clay
40.518	16.48	0.6661	4.043	38.139	11	4	silty clay to clay
40.682	19.86	0.7020	3.535	29.893	13	4	silty clay to clay
40.846	20.98	0.6317	3.011	9.953	10	5	clayey silt to silty clay
41.011	13.43	0.6088	4.533	20.462	13	3	clay
41.175	13.12	0.5060	3.857	43.923	13	3	clay
41.339	13.79	0.3910	2.836	35.948	7	5	clayey silt to silty clay
41.503	12.61	0.3734	2.961	33.763	8	4	silty clay to clay
41.667	12.48	0.3942	3.159	38.669	8	4	silty clay to clay
41.831	12.27	0.2970	2.421	40.628	6	5	clayey silt to silty clay
41.995	14.20	0.2484	1.750	35.912	7	5	clayey silt to silty clay
42.159	11.51	0.2468	2.144	42.670	6	5	clayey silt to silty clay
42.323	11.03	0.2535	2.299	57.958	5	5	clayey silt to silty clay
42.487	11.34	0.3492	3.079	66.796	7	4	silty clay to clay
42.651	10.99	0.3296	2.998	69.036	7	4	silty clay to clay
42.815	11.61	0.3466	2.986	58.390	7	4	silty clay to clay
42.979	11.32	0.3632	3.209	66.236	7	4	silty clay to clay
43.143	11.42	0.3762	3.294	68.085	7	4	silty clay to clay
43.307	11.23	0.4317	3.845	67.374	11	3	clay
43.471	12.37	0.5054	4.086	65.005	12	3	clay
43.635	14.44	0.4634	3.209	51.520	9	4	silty clay to clay
43.799	15.91	0.4287	2.695	31.412	8	5	clayey silt to silty clay
43.963	14.21	0.4734	3.332	32.765	9	4	silty clay to clay
44.127	14.32	0.4974	3.474	37.385	9	4	silty clay to clay
44.291	16.31	0.4507	2.762	33.222	8	5	clayey silt to silty clay
44.455	16.54	0.4120	2.491	25.300	8	5	clayey silt to silty clay
44.619	14.62	0.4185	2.863	27.418	7	5	clayey silt to silty clay
44.783	13.51	0.4246	3.143	31.885	9	4	silty clay to clay
44.948	13.50	0.4073	3.018	34.915	9	4	silty clay to clay
45.112	13.32	0.4329	3.249	39.698	9	4	silty clay to clay

Depth ft	Tip (Qt) (tsf)	Sleeve (Fs) (tsf)	F.Ratio (%)	PP (U2) (psi)	SPT (blows/ft)	Zone	Soil Behavior Type UBC-1983
45.276	12.47	0.4518	3.622	43.153	8	4	silty clay to clay
45.440	12.67	0.4575	3.611	48.378	8	4	silty clay to clay
45.604	12.23	0.4154	3.397	50.246	8	4	silty clay to clay
45.768	11.86	0.3825	3.225	53.005	8	4	silty clay to clay
45.932	11.35	0.3965	3.492	56.339	7	4	silty clay to clay
46.096	11.69	0.4338	3.711	60.248	11	3	clay
46.260	12.77	0.4432	3.469	59.854	8	4	silty clay to clay
46.424	12.84	0.4183	3.257	48.498	8	4	silty clay to clay
46.588	12.47	0.4114	3.299	47.962	8	4	silty clay to clay
46.752	12.16	0.4439	3.649	47.945	8	4	silty clay to clay
46.916	14.64	0.4284	2.927	49.370	7	5	clayey silt to silty clay
47.080	14.49	0.4085	2.819	42.070	7	5	clayey silt to silty clay
47.244	13.11	0.4087	3.117	41.525	8	4	silty clay to clay
47.408	13.20	0.4422	3.350	48.615	8	4	silty clay to clay
47.572	14.97	0.4912	3.283	52.888	10	4	silty clay to clay
47.736	17.28	0.5804	3.358	43.349	11	4	silty clay to clay
47.900	19.37	0.5608	2.895	41.745	9	5	clayey silt to silty clay
48.064	17.73	0.4861	2.743	33.306	8	5	clayey silt to silty clay
48.228	14.05	0.3908	2.781	32.318	7	5	clayey silt to silty clay
48.392	12.20	0.3978	3.261	39.126	8	4	silty clay to clay
48.556	13.05	0.4140	3.172	49.686	8	4	silty clay to clay
48.720	13.81	0.4018	2.911	46.570	9	4	silty clay to clay
48.885	13.18	0.3899	2.958	51.831	8	4	silty clay to clay
49.049	12.47	0.4634	3.715	53.931	8	4	silty clay to clay
49.213	13.19	0.4268	3.235	57.197	8	4	silty clay to clay
49.377	13.36	0.4110	3.077	47.178	9	4	silty clay to clay
49.541	12.98	0.3999	3.081	51.126	8	4	silty clay to clay
49.705	12.56	0.3820	3.041	55.062	8	4	silty clay to clay
49.869	12.68	0.4087	3.224	57.673	8	4	silty clay to clay
50.033	13.32	0.4286	3.218	60.387	9	4	silty clay to clay
50.197	14.34	0.4356	3.037	55.157	9	4	silty clay to clay
50.361	14.80	0.3910	2.643	51.611	7	5	clayey silt to silty clay
50.525	13.37	0.3549	2.653	47.324	6	5	clayey silt to silty clay
50.689	12.13	0.3204	2.642	56.286	6	5	clayey silt to silty clay
50.853	12.44	0.3133	2.518	63.154	6	5	clayey silt to silty clay
51.017	12.39	0.3016	2.434	64.964	6	5	clayey silt to silty clay
51.181	12.21	0.3046	2.494	66.478	6	5	clayey silt to silty clay
51.345	12.82	0.2840	2.216	68.809	6	5	clayey silt to silty clay
51.509	12.44	0.2598	2.088	63.713	6	5	clayey silt to silty clay
51.673	11.41	0.2512	2.201	67.040	5	5	clayey silt to silty clay
51.837	10.94	0.2816	2.574	70.165	7	4	silty clay to clay
52.001	15.73	0.3302	2.099	55.153	8	5	clayey silt to silty clay
52.165	15.94	0.4199	2.635	46.219	8	5	clayey silt to silty clay
52.329	17.23	0.6583	3.820	56.817	11	4	silty clay to clay
52.493	17.05	0.6888	4.039	26.412	11	4	silty clay to clay
52.657	20.52	0.7405	3.608	18.870	13	4	silty clay to clay
52.822	25.71	0.8254	3.210	25.068	12	5	clayey silt to silty clay
52.986	26.40	0.9552	3.618	24.197	13	5	clayey silt to silty clay
53.150	24.50	1.0206	4.166	28.645	16	4	silty clay to clay
53.314	23.44	1.1246	4.798	18.004	22	3	clay
53.478	28.76	1.1067	3.849	26.692	18	4	silty clay to clay
53.642	39.83	1.0988	2.758	7.889	15	6	sandy silt to clayey silt
53.806	43.89	0.8900	2.028	3.446	17	6	sandy silt to clayey silt
53.970	42.63	1.1051	2.592	1.454	16	6	sandy silt to clayey silt
54.134	40.00	0.9881	2.470	1.884	15	6	sandy silt to clayey silt
54.298	29.72	0.8440	2.839	11.263	14	5	clayey silt to silty clay
54.462	22.42	0.9879	4.407	23.626	21	3	clay

Depth ft	Tip (Qt) (tsf)	Sleeve (Fs) (tsf)	F.Ratio (%)	PP (U2) (psi)	SPT (blows/ft)	Zone	Soil Behavior Type UBC-1983
54.626	27.02	1.1434	4.232	54.976	17	4	silty clay to clay
54.790	28.62	1.1281	3.941	38.163	18	4	silty clay to clay
54.954	25.71	1.5002	5.835	30.123	25	3	clay
55.118	29.16	1.8692	6.410	69.811	28	3	clay
55.282	37.13	1.7842	4.805	34.750	36	3	clay
55.446	50.99	1.4791	2.901	6.440	20	6	sandy silt to clayey silt
55.610	60.59	1.7804	2.938	-2.666	23	6	sandy silt to clayey silt
55.774	57.24	1.9605	3.425	-1.408	27	5	clayey silt to silty clay
55.938	51.70	2.3688	4.582	35.523	33	4	silty clay to clay
56.102	55.54	2.5888	4.661	53.754	35	4	silty clay to clay
56.266	67.68	1.9618	2.898	11.940	26	6	sandy silt to clayey silt
56.430	71.51	1.6365	2.289	-3.821	27	6	sandy silt to clayey silt
56.594	63.00	1.7986	2.855	-2.716	24	6	sandy silt to clayey silt
56.759	46.31	1.8836	4.067	1.815	22	5	clayey silt to silty clay
56.923	35.22	2.0936	5.944	20.292	34	3	clay
57.087	36.76	2.4421	6.643	49.710	35	3	clay
57.251	41.25	2.6943	6.531	33.284	40	3	clay
57.415	55.68	1.4953	2.685	11.428	21	6	sandy silt to clayey silt
57.579	70.41	1.0488	1.490	-2.523	22	7	silty sand to sandy silt
57.743	78.30	0.8906	1.137	-3.281	19	8	sand to silty sand
57.907	81.73	1.0056	1.230	-3.066	20	8	sand to silty sand
58.071	87.72	1.1789	1.344	-2.186	21	8	sand to silty sand
58.235	86.23	1.1279	1.308	-1.145	21	8	sand to silty sand
58.399	80.03	1.1605	1.450	-0.641	26	7	silty sand to sandy silt
58.563	64.38	1.3417	2.084	3.565	21	7	silty sand to sandy silt
58.727	42.62	1.3275	3.115	5.691	20	5	clayey silt to silty clay
58.891	30.18	1.8294	6.062	21.718	29	3	clay
59.055	33.99	1.9364	5.697	64.593	33	3	clay
59.219	61.06	1.8331	3.002	19.601	23	6	sandy silt to clayey silt
59.383	67.06	1.2946	1.930	-1.000	21	7	silty sand to sandy silt
59.547	69.55	0.8044	1.157	-3.958	22	7	silty sand to sandy silt
59.711	69.20	0.9158	1.323	-4.261	22	7	silty sand to sandy silt
59.875	68.59	1.0824	1.578	-3.082	22	7	silty sand to sandy silt
60.039	71.94	1.3363	1.858	-1.358	23	7	silty sand to sandy silt
60.203	58.23	1.4568	2.502	0.160	22	6	sandy silt to clayey silt
60.367	32.80	1.3688	4.173	4.426	21	4	silty clay to clay
60.532	25.22	1.1103	4.403	12.318	16	4	silty clay to clay
60.696	23.85	1.2477	5.231	31.355	23	3	clay
60.860	30.60	1.3307	4.348	59.665	20	4	silty clay to clay
61.024	65.33	1.5332	2.347	15.534	25	6	sandy silt to clayey silt
61.188	88.55	0.9024	1.019	-1.289	21	8	sand to silty sand
61.352	89.62	1.0104	1.127	-3.570	21	8	sand to silty sand
61.516	83.22	1.0772	1.294	-3.035	27	7	silty sand to sandy silt
61.680	79.02	1.1240	1.422	-2.267	25	7	silty sand to sandy silt
61.844	74.06	1.1980	1.618	13.123	24	7	silty sand to sandy silt
62.008	75.68	1.2855	1.699	14.290	24	7	silty sand to sandy silt
62.172	77.89	1.7799	2.285	15.467	25	7	silty sand to sandy silt
62.336	72.04	2.2199	3.081	17.825	28	6	sandy silt to clayey silt
62.500	56.18	2.2531	4.011	17.291	27	5	clayey silt to silty clay
62.664	64.56	1.5690	2.430	21.761	25	6	sandy silt to clayey silt
62.828	74.81	1.4366	1.920	5.560	24	7	silty sand to sandy silt
62.992	79.40	1.3970	1.760	4.713	25	7	silty sand to sandy silt
63.156	84.19	1.8130	2.153	7.334	27	7	silty sand to sandy silt
63.320	80.47	2.0367	2.531	10.818	31	6	sandy silt to clayey silt
63.484	80.73	1.2192	1.510	8.530	26	7	silty sand to sandy silt
63.648	79.37	1.0333	1.302	-0.524	25	7	silty sand to sandy silt
63.812	74.46	1.0721	1.440	-0.638	24	7	silty sand to sandy silt

Depth ft	Tip (Qt) (tsf)	Sleeve (Fs) (tsf)	F.Ratio (%)	PP (U2) (psi)	SPT (blows/ft)	Zone	Soil Behavior Type UBC-1983
63.976	66.62	1.1339	1.702	0.359	21	7	silty sand to sandy silt
64.140	66.50	1.1636	1.750	2.418	21	7	silty sand to sandy silt
64.304	72.94	1.2505	1.714	5.079	23	7	silty sand to sandy silt
64.469	83.02	1.4818	1.785	7.949	26	7	silty sand to sandy silt
64.633	90.76	1.6898	1.862	11.729	29	7	silty sand to sandy silt
64.797	92.41	1.9563	2.117	14.785	29	7	silty sand to sandy silt
64.961	74.13	1.9236	2.595	17.062	28	6	sandy silt to clayey silt
65.125	36.27	2.1823	6.017	39.681	35	3	clay
65.289	44.79	1.4793	3.302	80.931	21	5	clayey silt to silty clay
65.453	79.20	1.6315	2.060	17.157	25	7	silty sand to sandy silt
65.617	105.82	2.6214	2.477	8.463	34	7	silty sand to sandy silt
65.781	161.09	3.1900	1.980	12.561	51	7	silty sand to sandy silt
65.945	171.45	3.3866	1.975	32.916	55	7	silty sand to sandy silt
66.109	137.84	2.7866	2.022	6.509	44	7	silty sand to sandy silt
66.273	112.70	1.3406	1.189	-0.151	27	8	sand to silty sand
66.437	87.92	1.2888	1.466	-2.188	28	7	silty sand to sandy silt
66.601	70.88	1.4900	2.102	-0.598	23	7	silty sand to sandy silt
66.765	68.16	1.4346	2.105	1.167	22	7	silty sand to sandy silt
66.929	66.38	1.2514	1.885	0.349	21	7	silty sand to sandy silt
67.093	51.47	1.0542	2.048	-0.445	20	6	sandy silt to clayey silt
67.257	45.54	0.9704	2.131	0.277	17	6	sandy silt to clayey silt
67.421	41.63	0.9811	2.357	0.641	16	6	sandy silt to clayey silt
67.585	38.69	1.0684	2.762	2.929	15	6	sandy silt to clayey silt
67.749	30.01	0.9415	3.137	6.057	14	5	clayey silt to silty clay
67.913	20.09	0.5769	2.871	12.528	10	5	clayey silt to silty clay
68.077	17.29	0.7288	4.214	31.677	17	3	clay
68.241	14.86	0.8017	5.397	56.107	14	3	clay
68.406	31.67	0.7582	2.394	48.787	12	6	sandy silt to clayey silt
68.570	40.36	0.7425	1.839	28.660	15	6	sandy silt to clayey silt
68.734	44.74	0.8804	1.968	23.820	17	6	sandy silt to clayey silt
68.898	45.83	0.9540	2.082	27.715	18	6	sandy silt to clayey silt
69.062	46.44	0.9694	2.088	24.786	18	6	sandy silt to clayey silt
69.226	46.80	0.9320	1.991	24.336	18	6	sandy silt to clayey silt
69.390	48.49	0.7957	1.641	21.653	15	7	silty sand to sandy silt
69.554	49.66	0.7298	1.469	19.341	16	7	silty sand to sandy silt
69.718	44.79	0.7492	1.672	22.196	14	7	silty sand to sandy silt
69.882	34.60	0.7994	2.310	28.815	13	6	sandy silt to clayey silt
70.046	28.82	0.8474	2.940	35.613	14	5	clayey silt to silty clay
70.210	24.62	0.8294	3.369	44.284	12	5	clayey silt to silty clay
70.374	22.13	0.6341	2.866	54.471	11	5	clayey silt to silty clay
70.538	18.96	0.5853	3.087	59.569	9	5	clayey silt to silty clay
70.702	16.17	0.6532	4.039	72.315	10	4	silty clay to clay
70.866	32.92	0.5281	1.604	64.603	13	6	sandy silt to clayey silt
71.030	32.23	0.6784	2.105	51.484	12	6	sandy silt to clayey silt
71.194	29.27	0.6441	2.200	69.189	11	6	sandy silt to clayey silt
71.358	32.56	0.7054	2.167	67.439	12	6	sandy silt to clayey silt
71.522	23.01	0.6788	2.950	75.137	11	5	clayey silt to silty clay
71.686	26.49	0.6517	2.460	92.684	10	6	sandy silt to clayey silt
71.850	29.44	0.5435	1.846	84.173	11	6	sandy silt to clayey silt
72.014	27.21	1.0252	3.768	68.967	17	4	silty clay to clay
72.178	23.21	1.0511	4.529	74.814	22	3	clay
72.343	71.94	1.1226	1.560	20.233	23	7	silty sand to sandy silt
72.507	89.89	1.2521	1.393	18.384	29	7	silty sand to sandy silt
72.671	94.69	1.2723	1.344	12.136	23	8	sand to silty sand
72.835	90.36	1.0798	1.195	9.362	22	8	sand to silty sand
72.999	73.41	0.9051	1.233	1.906	23	7	silty sand to sandy silt
73.163	59.60	0.6638	1.114	1.423	19	7	silty sand to sandy silt

Depth ft	Tip (Qt) (tsf)	Sleeve (Fs) (tsf)	F.Ratio (%)	PP (U2) (psi)	SPT (blows/ft)	Zone	Soil Behavior Type UBC-1983
73.327	50.53	0.6554	1.297	1.559	16	7	silty sand to sandy silt
73.491	44.97	0.6003	1.335	3.855	14	7	silty sand to sandy silt
73.655	46.88	0.6107	1.303	6.330	15	7	silty sand to sandy silt
73.819	45.86	0.7006	1.528	7.093	15	7	silty sand to sandy silt
73.983	47.75	0.7874	1.649	12.064	15	7	silty sand to sandy silt
74.147	46.89	0.8223	1.754	17.418	15	7	silty sand to sandy silt
74.311	37.12	0.7268	1.958	19.620	14	6	sandy silt to clayey silt
74.475	28.83	0.8058	2.795	25.266	14	5	clayey silt to silty clay
74.639	19.48	0.8540	4.385	37.376	19	3	clay
74.803	39.52	0.9426	2.385	63.493	15	6	sandy silt to clayey silt
74.967	66.37	0.9703	1.462	22.899	21	7	silty sand to sandy silt
75.131	70.69	1.0749	1.521	25.140	23	7	silty sand to sandy silt
75.295	74.90	1.2452	1.662	25.998	24	7	silty sand to sandy silt
75.459	81.23	1.3157	1.620	26.385	26	7	silty sand to sandy silt
75.623	83.90	1.3597	1.621	23.954	27	7	silty sand to sandy silt
75.787	83.02	1.3560	1.633	24.566	27	7	silty sand to sandy silt
75.951	78.90	1.3794	1.748	24.709	25	7	silty sand to sandy silt
76.115	70.33	1.3136	1.868	22.932	22	7	silty sand to sandy silt
76.280	58.18	1.2640	2.172	25.319	22	6	sandy silt to clayey silt
76.444	46.40	1.2008	2.588	26.931	18	6	sandy silt to clayey silt
76.608	30.58	1.0570	3.457	40.719	15	5	clayey silt to silty clay
76.772	20.36	0.8328	4.090	46.943	13	4	silty clay to clay
76.936	17.32	0.4651	2.685	72.135	8	5	clayey silt to silty clay
77.100	18.96	0.3947	2.082	73.518	9	5	clayey silt to silty clay
77.264	21.34	0.5358	2.510	72.112	10	5	clayey silt to silty clay
77.428	21.53	0.7691	3.572	72.360	14	4	silty clay to clay
77.592	40.94	0.6686	1.633	72.595	13	7	silty sand to sandy silt
77.756	61.17	0.9460	1.547	19.912	20	7	silty sand to sandy silt
77.920	49.16	0.7719	1.570	24.542	16	7	silty sand to sandy silt
78.084	56.62	0.9731	1.719	20.460	18	7	silty sand to sandy silt
78.248	65.35	1.1766	1.800	13.573	21	7	silty sand to sandy silt
78.412	76.45	0.7274	0.952	7.246	18	8	sand to silty sand
78.576	80.28	0.7177	0.894	-1.232	19	8	sand to silty sand
78.740	66.30	0.6712	1.012	-1.927	21	7	silty sand to sandy silt
78.904	58.31	0.6338	1.087	10.648	19	7	silty sand to sandy silt
79.068	65.34	1.1297	1.729	6.650	21	7	silty sand to sandy silt
79.232	97.95	1.1776	1.202	15.426	23	8	sand to silty sand
79.396	243.68	1.5504	0.636	22.093	47	9	sand
79.560	446.26	1.7905	0.401	16.892	71	10	gravelly sand to sand

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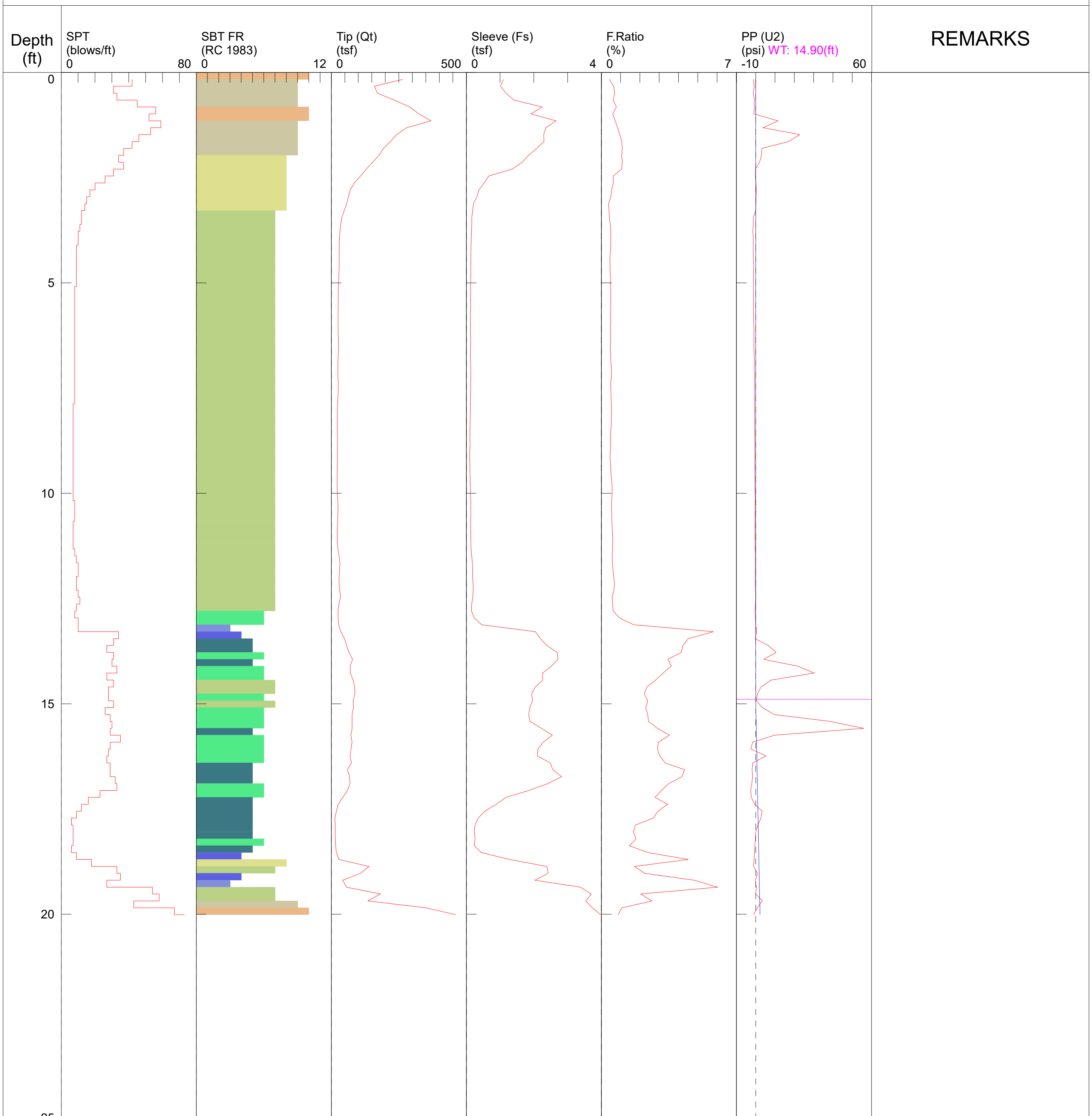
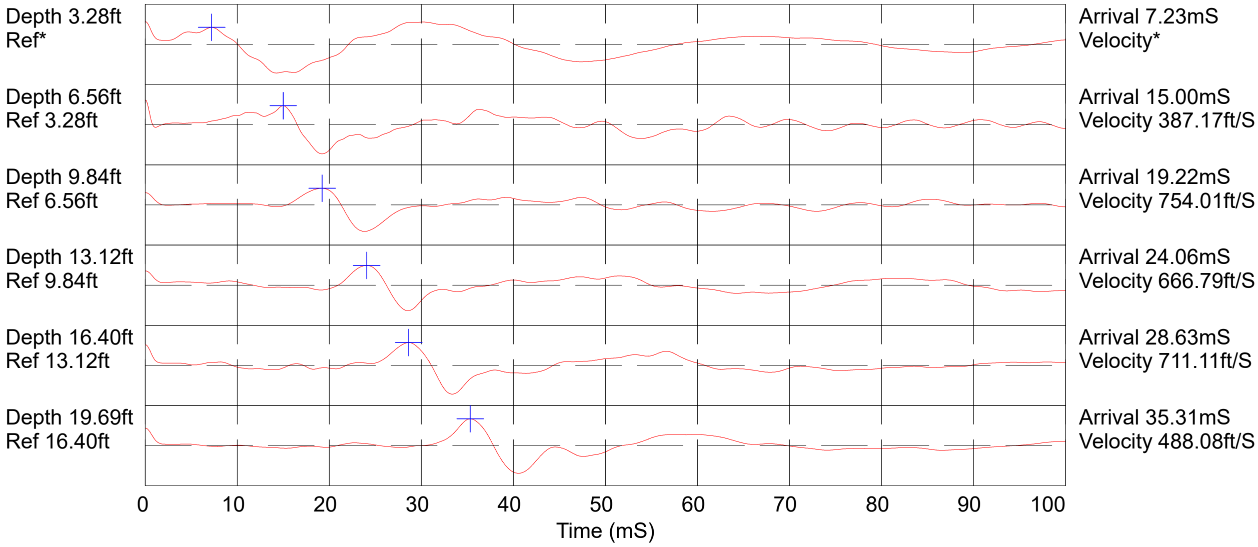
OPERATOR: OGE BAK
CONE ID: DDG1661
TEST DATE: 2/7/2024 9:45:33 AM
TOTAL DEPTH: 20.013 ft



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 (*)

*SBT/SPT CORRELATION: UBC-1983

COMMENT: GeoEngineers / CPT-4a / 5528 Doane Ave Portland



Hammer to Rod String Distance (ft): 2.03
* = Not Determined

GeoEngineers / CPT-4a / 5528 Doane Ave Portland

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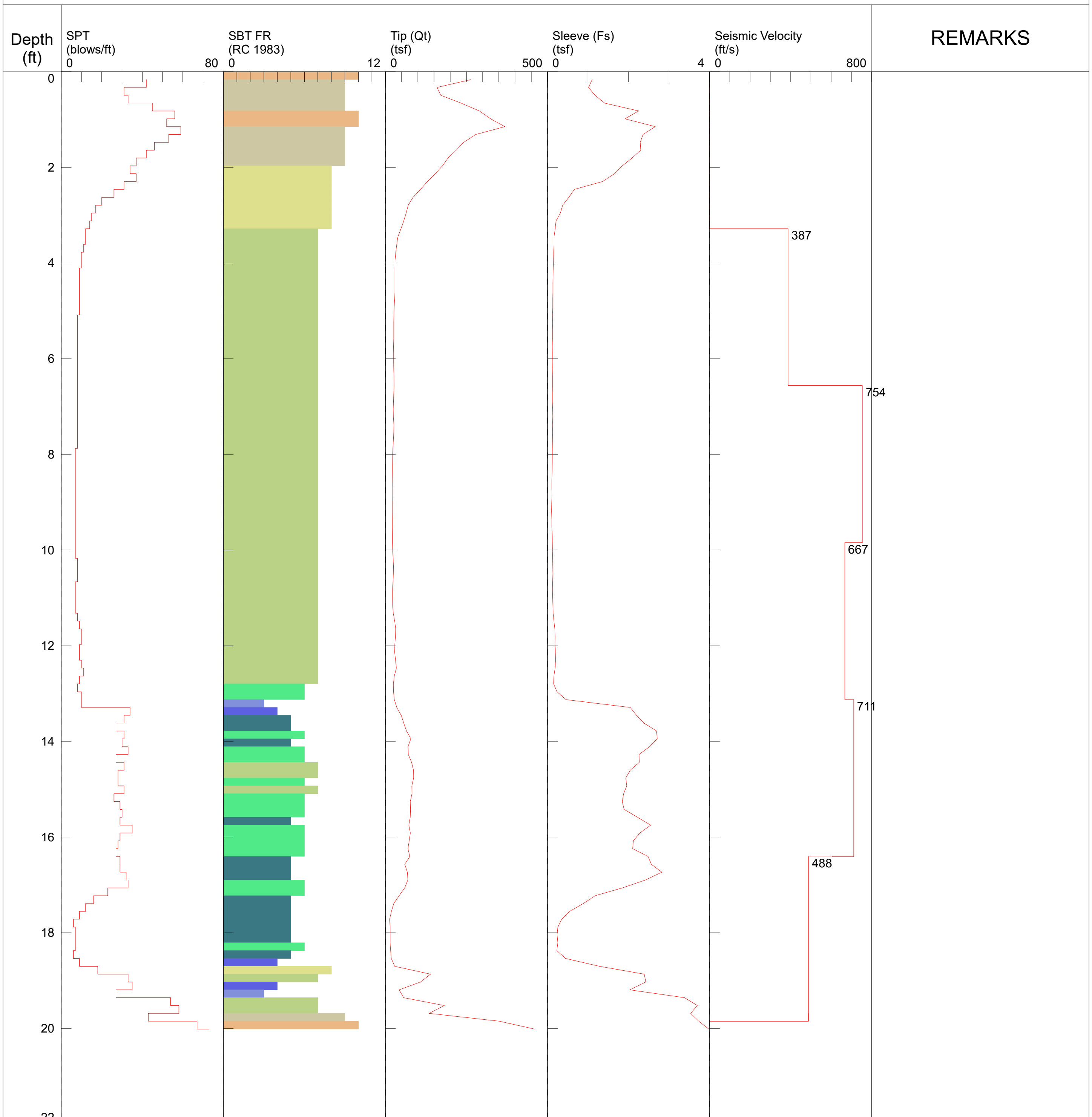
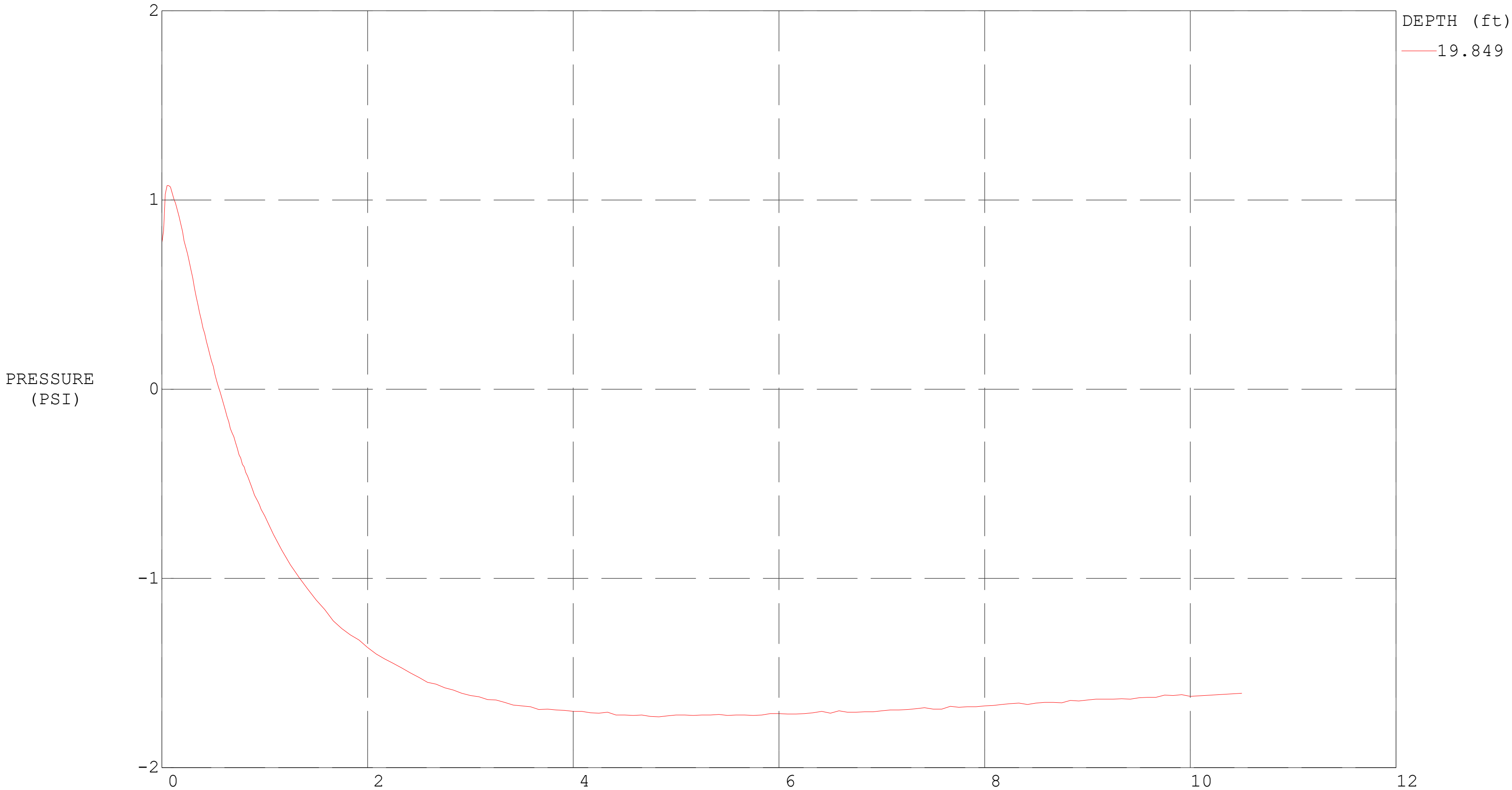


1 sensitive fine grained	4 silty clay to clay	7 silty sand to sandy silt	10 gravelly sand to sand
2 organic material	5 clayey silt to silty clay	8 sand to silty sand	11 very stiff fine grained (*)
3 clay	6 sandy silt to clayey silt	9 sand	12 sand to clayey sand (*)

*SBT/SPT CORRELATION: UBC-1983

COMMENT: GeoEngineers / CPT-4a / 5528 Doane Ave Portland

CONE ID: DDG1661
TEST DATE: 2/7/2024 9:45:33 AM



MAXIMUM PRESSURE = 1.076 (PSI)
HYDROSTATIC PRESSURE = 2.147 (PSI), WATER TABLE: 14.90 ft

GeoEngineers / CPT-4a / 5528 Doane Ave Portland

OPERATOR: OGE BAK
CONE ID: DDG1661
TEST DATE: 2/7/2024 9:45:33 AM
TOTAL DEPTH: 20.013 ft

Depth ft	Tip (Qt) (tsf)	Sleeve (Fs) (tsf)	F.Ratio (%)	PP (U2) (psi)	SPT (blows/ft)	Zone	Soil Behavior Type UBC-1983
0.164	263.61	1.1005	0.417	-1.342	42	10	gravelly sand to sand
0.328	159.25	1.0106	0.635	-0.753	31	9	sand
0.492	170.58	1.1724	0.687	-1.229	33	9	sand
0.656	233.37	1.4104	0.604	-0.622	45	9	sand
0.820	290.02	2.2499	0.776	-0.722	56	9	sand
0.984	324.29	1.9117	0.590	-0.916	52	10	gravelly sand to sand
1.148	368.52	2.6594	0.722	11.550	59	10	gravelly sand to sand
1.312	278.78	2.3561	0.845	3.812	53	9	sand
1.476	241.31	2.2891	0.949	22.727	46	9	sand
1.640	218.48	2.2993	1.052	17.040	42	9	sand
1.804	193.34	2.0881	1.080	3.195	37	9	sand
1.969	175.99	1.8438	1.048	2.917	34	9	sand
2.133	153.39	1.6537	1.078	1.973	37	8	sand to silty sand
2.297	129.40	1.3568	1.049	-0.191	31	8	sand to silty sand
2.461	108.20	0.6635	0.613	-0.225	26	8	sand to silty sand
2.625	84.98	0.5264	0.619	0.115	20	8	sand to silty sand
2.789	70.13	0.3734	0.533	0.270	17	8	sand to silty sand
2.953	63.76	0.3167	0.497	0.158	15	8	sand to silty sand
3.117	56.60	0.2097	0.370	0.105	14	8	sand to silty sand
3.281	48.15	0.1879	0.390	-0.022	12	8	sand to silty sand
3.445	38.86	0.1604	0.413	-1.287	12	7	silty sand to sandy silt
3.609	34.91	0.1604	0.459	-1.258	11	7	silty sand to sandy silt
3.773	31.86	0.1537	0.483	-1.518	10	7	silty sand to sandy silt
3.937	29.90	0.1467	0.491	-1.287	10	7	silty sand to sandy silt
4.101	29.27	0.1406	0.480	-1.232	9	7	silty sand to sandy silt
4.265	29.41	0.1355	0.461	-1.229	9	7	silty sand to sandy silt
4.429	29.29	0.1305	0.446	-1.193	9	7	silty sand to sandy silt
4.593	29.13	0.1308	0.449	-1.043	9	7	silty sand to sandy silt
4.757	28.44	0.1283	0.451	-1.150	9	7	silty sand to sandy silt
4.921	27.14	0.1287	0.474	-1.133	9	7	silty sand to sandy silt
5.085	26.16	0.1229	0.470	-1.165	8	7	silty sand to sandy silt
5.249	25.84	0.1255	0.486	-1.162	8	7	silty sand to sandy silt
5.413	25.80	0.1233	0.478	-1.126	8	7	silty sand to sandy silt
5.577	25.61	0.1236	0.483	-1.143	8	7	silty sand to sandy silt
5.741	24.76	0.1171	0.473	-1.117	8	7	silty sand to sandy silt
5.906	25.05	0.1200	0.479	-1.102	8	7	silty sand to sandy silt
6.070	25.09	0.1156	0.461	-1.000	8	7	silty sand to sandy silt
6.234	25.75	0.1219	0.473	-0.983	8	7	silty sand to sandy silt
6.398	26.46	0.1242	0.469	-1.007	8	7	silty sand to sandy silt
6.562	26.61	0.1231	0.463	-1.000	8	7	silty sand to sandy silt
6.726	25.35	0.1203	0.474	-0.775	8	7	silty sand to sandy silt
6.890	24.71	0.1226	0.496	-0.751	8	7	silty sand to sandy silt
7.054	24.30	0.1275	0.525	-0.751	8	7	silty sand to sandy silt
7.218	24.65	0.1293	0.525	-0.737	8	7	silty sand to sandy silt
7.382	25.95	0.1247	0.481	-0.708	8	7	silty sand to sandy silt
7.546	25.56	0.1261	0.493	-0.682	8	7	silty sand to sandy silt

Depth ft	Tip (Qt) (tsf)	Sleeve (Fs) (tsf)	F.Ratio (%)	PP (U2) (psi)	SPT (blows/ft)	Zone	Soil Behavior Type UBC-1983
7.710	24.50	0.1232	0.503	-0.725	8	7	silty sand to sandy silt
7.874	23.01	0.1186	0.515	-0.722	7	7	silty sand to sandy silt
8.038	22.20	0.1158	0.522	-0.693	7	7	silty sand to sandy silt
8.202	22.09	0.1138	0.515	-0.641	7	7	silty sand to sandy silt
8.366	21.85	0.1115	0.510	-0.605	7	7	silty sand to sandy silt
8.530	22.39	0.1068	0.477	-0.560	7	7	silty sand to sandy silt
8.694	22.57	0.1042	0.462	-0.528	7	7	silty sand to sandy silt
8.858	22.58	0.1089	0.482	-0.483	7	7	silty sand to sandy silt
9.022	22.48	0.1015	0.452	-0.473	7	7	silty sand to sandy silt
9.186	22.23	0.0997	0.449	-0.481	7	7	silty sand to sandy silt
9.350	21.99	0.1048	0.477	-0.488	7	7	silty sand to sandy silt
9.514	21.66	0.1075	0.496	-0.481	7	7	silty sand to sandy silt
9.678	21.44	0.1139	0.531	-0.457	7	7	silty sand to sandy silt
9.843	21.51	0.1186	0.552	-0.450	7	7	silty sand to sandy silt
10.007	22.34	0.1235	0.553	-0.352	7	7	silty sand to sandy silt
10.171	23.90	0.1271	0.532	-0.383	8	7	silty sand to sandy silt
10.335	24.85	0.1290	0.519	-0.330	8	7	silty sand to sandy silt
10.499	24.36	0.1307	0.536	-0.397	8	7	silty sand to sandy silt
10.663	23.27	0.1264	0.543	-0.414	7	7	silty sand to sandy silt
10.827	22.18	0.1251	0.564	-0.430	7	7	silty sand to sandy silt
10.991	22.13	0.1275	0.576	-0.402	7	7	silty sand to sandy silt
11.155	22.46	0.1309	0.583	-0.373	7	7	silty sand to sandy silt
11.319	24.21	0.1400	0.578	-0.282	8	7	silty sand to sandy silt
11.483	28.50	0.1581	0.555	-0.177	9	7	silty sand to sandy silt
11.647	31.64	0.1790	0.566	-0.184	10	7	silty sand to sandy silt
11.811	30.86	0.1856	0.602	-0.230	10	7	silty sand to sandy silt
11.975	28.80	0.1835	0.637	-0.232	9	7	silty sand to sandy silt
12.139	28.66	0.1955	0.682	-0.234	9	7	silty sand to sandy silt
12.303	31.12	0.2013	0.647	-0.155	10	7	silty sand to sandy silt
12.467	33.70	0.1867	0.554	-0.041	11	7	silty sand to sandy silt
12.631	27.38	0.1586	0.579	-0.249	9	7	silty sand to sandy silt
12.795	24.54	0.1499	0.611	-0.304	8	7	silty sand to sandy silt
12.959	24.88	0.2333	0.937	-0.220	10	6	sandy silt to clayey silt
13.123	27.21	0.4565	1.678	0.029	10	6	sandy silt to clayey silt
13.287	35.17	2.0422	5.806	0.517	34	3	clay
13.451	48.78	2.1961	4.502	-0.347	31	4	silty clay to clay
13.615	56.33	2.3809	4.227	6.394	27	5	clayey silt to silty clay
13.780	65.04	2.6900	4.136	10.538	31	5	clayey silt to silty clay
13.944	78.41	2.7060	3.451	4.132	30	6	sandy silt to clayey silt
14.108	69.56	2.5188	3.621	21.792	33	5	clayey silt to silty clay
14.272	70.59	2.2570	3.197	30.183	27	6	sandy silt to clayey silt
14.436	80.58	2.2632	2.809	7.958	31	6	sandy silt to clayey silt
14.600	86.42	2.0425	2.363	2.705	28	7	silty sand to sandy silt
14.764	87.08	1.9317	2.218	0.827	28	7	silty sand to sandy silt
14.928	81.25	1.9559	2.407	0.368	31	6	sandy silt to clayey silt
15.092	81.85	1.8788	2.295	3.489	26	7	silty sand to sandy silt
15.256	76.66	1.8435	2.405	9.582	29	6	sandy silt to clayey silt
15.420	77.15	1.8875	2.447	38.658	30	6	sandy silt to clayey silt
15.584	76.66	2.2239	2.901	56.018	29	6	sandy silt to clayey silt
15.748	72.26	2.5470	3.525	9.828	35	5	clayey silt to silty clay
15.912	76.99	2.2770	2.958	-1.664	29	6	sandy silt to clayey silt
16.076	73.08	2.1196	2.900	-2.497	28	6	sandy silt to clayey silt
16.240	69.39	2.1028	3.030	5.294	27	6	sandy silt to clayey silt
16.404	74.99	2.4826	3.311	-1.573	29	6	sandy silt to clayey silt
16.568	59.54	2.5628	4.304	-1.858	29	5	clayey silt to silty clay
16.732	67.57	2.8204	4.174	-1.662	32	5	clayey silt to silty clay
16.896	69.17	2.4160	3.493	-2.021	33	5	clayey silt to silty clay

Depth ft	Tip (Qt) (tsf)	Sleeve (Fs) (tsf)	F.Ratio (%)	PP (U2) (psi)	SPT (blows/ft)	Zone	Soil Behavior Type UBC-1983
17.060	59.44	1.8502	3.113	-2.647	23	6	sandy silt to clayey silt
17.224	42.35	1.1765	2.778	-2.286	16	6	sandy silt to clayey silt
17.388	25.74	0.8834	3.431	-0.576	12	5	clayey silt to silty clay
17.552	18.59	0.5442	2.928	3.278	9	5	clayey silt to silty clay
17.717	13.09	0.3507	2.679	2.681	6	5	clayey silt to silty clay
17.881	14.42	0.2540	1.762	1.126	7	5	clayey silt to silty clay
18.045	14.17	0.2348	1.657	0.077	7	5	clayey silt to silty clay
18.209	14.21	0.2549	1.793	-0.201	7	5	clayey silt to silty clay
18.373	15.90	0.2310	1.452	-0.478	6	6	sandy silt to clayey silt
18.537	18.40	0.4436	2.410	-0.679	9	5	clayey silt to silty clay
18.701	28.43	1.2799	4.501	-0.899	18	4	silty clay to clay
18.865	139.31	2.3868	1.713	-1.165	33	8	sand to silty sand
19.029	108.34	2.4241	2.237	0.880	35	7	silty sand to sandy silt
19.193	42.14	2.0306	4.819	-0.218	27	4	silty clay to clay
19.357	56.21	3.3803	6.013	0.270	54	3	clay
19.521	181.57	3.6983	2.037	-0.024	58	7	silty sand to sandy silt
19.685	134.84	3.5339	2.621	3.503	43	7	silty sand to sandy silt
19.849	351.54	3.7310	1.061	0.806	67	9	sand
20.013	459.61	3.9810	0.866	-1.133	73	10	gravelly sand to sand

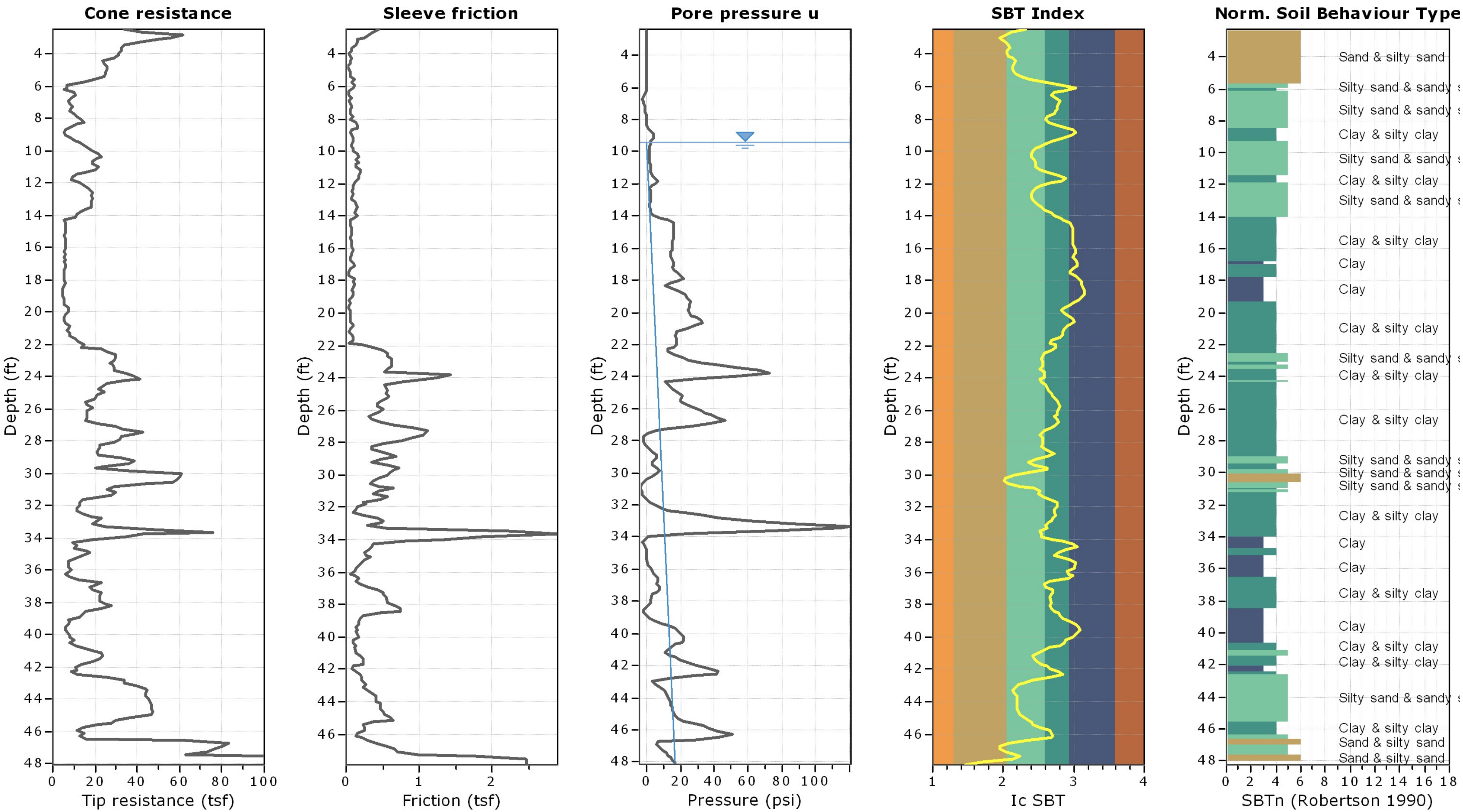
Appendix A.2

Previous Subsurface Exploration Logs

Professional Service Industries, Inc. (PSI)
6032 North Cutter Circle, Suite 480
Portland, Oregon 97217
(503) 289-1778

CPT: CPT-01
Total depth: 48.06 ft, Date: 10/27/2016
Surface Elevation: 184.00 ft
Coords: X:0.00, Y:0.00
Cone Type: Uknown
Cone Operator: Uknown

Project: P66 - Proposed Butane Tank
Location: Portland, Oregon

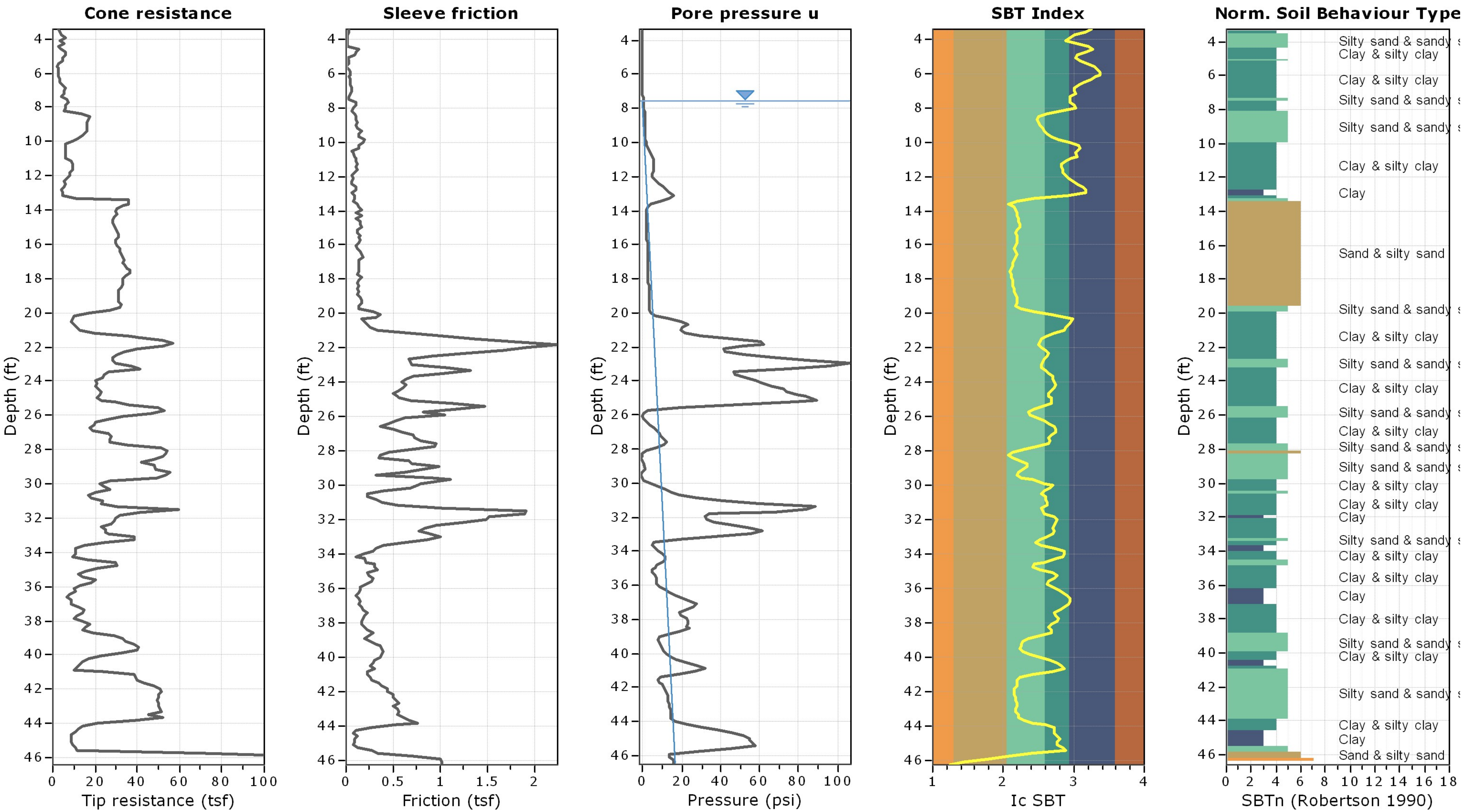


CPeT-IT v.1.7.6.42 - CPTU data presentation & interpretation software - Report created on: 10/27/2016, 9:55:42 AM
Project file: P:\704 Geotech & Environmental\0704900 - 0704999\0704999 - GEO P66 Butane Tank Portland, OR\Analysis\CPT\P66 project.cpt

Professional Service Industries, Inc. (PSI)
6032 North Cutter Circle, Suite 480
Portland, Oregon 97217
(503) 289-1778

CPT: CPT-02
Total depth: 46.42 ft, Date: 10/27/2016
Surface Elevation: 184.00 ft
Coords: X:0.00, Y:0.00
Cone Type: Uknown
Cone Operator: Uknown

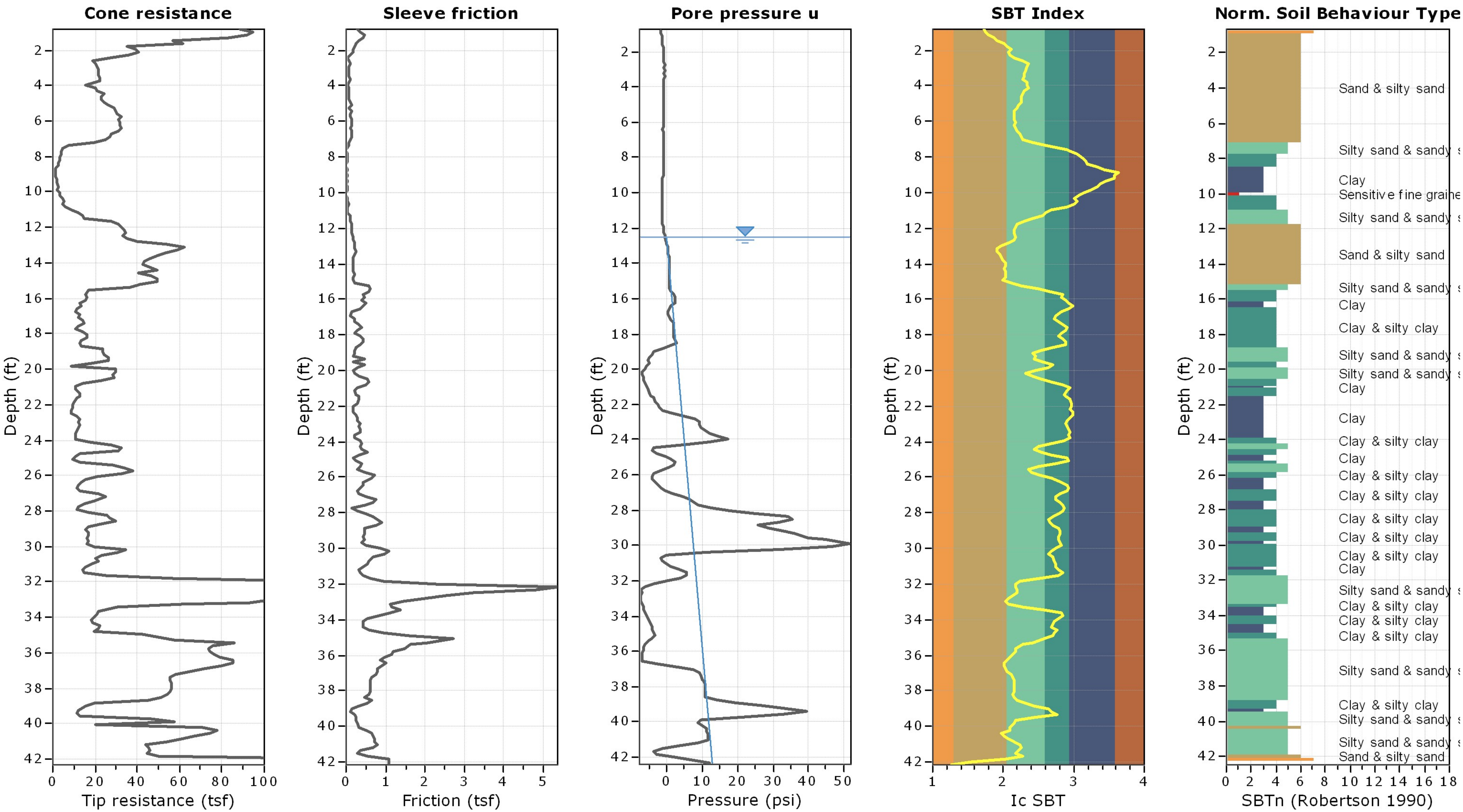
Project: **P66 - Proposed Butane Tank**
Location: **Portland, Oregon**



Professional Service Industries, Inc. (PSI)
6032 North Cutter Circle, Suite 480
Portland, Oregon 97217
(503) 289-1778

CPT: CPT-03
Total depth: 42.32 ft, Date: 10/27/2016
Surface Elevation: 184.00 ft
Coords: X:0.00, Y:0.00
Cone Type: Uknown
Cone Operator: Uknown

Project: **P66 - Proposed Butane Tank**
Location: **Portland, Oregon**



Appendix A.3
Historical Geotechnical Reports

Information To Build On
Engineering • Consulting • Testing

GEOTECHNICAL ENGINEERING REPORT

Proposed 90,000-Gallon Butane Tank
Philips 66
Portland Terminal
5528 NW Doane Avenue
Portland, Oregon

Prepared for:

Sunoco Logistics Partners, LP
525 Fritztown Road
Sinking Spring, Pennsylvania

Prepared by:

Professional Service Industries, Inc.
6032 North Cutter Circle, Suite 480
Portland, Oregon 97217

October 31, 2016

PSI PROJECT NO. 0704999

October 31, 2016

Sunoco Logistics Partners, LP
525 Fritztown Road
Sinking Spring, Pennsylvania 19608

Attention: Mr. Richard Voytek
BD Engineering Specialist
(281) 637-6356
REVOYTEK@sunocologistics.com

Subject: Geotechnical Engineering Report
Proposed 90,000-Gallon Butane Storage Tank
Philips 66 Portland Terminal
5528 NW Doane Avenue
Portland, Oregon
PSI Project No. 0704999

Dear Mr. Voytek:

Professional Service Industries, Inc. (PSI) is pleased to submit this geotechnical engineering report for the proposed 90,000-gallon butane storage tank to be located at the existing Philips 66 facility in Portland, Oregon. This report summarizes the work accomplished and provides PSI's recommendations for design and construction of the proposed project. PSI performed the requested geotechnical engineering services in general accordance with PSI proposal No.: 0704-191429R1.

We thank you for choosing us as your consultant for this project. Please contact the undersigned at (503) 289-1778, if you have any questions or we if may be of further service.

Respectfully Submitted,

PROFESSIONAL SERVICE INDUSTRIES, INC.

REGISTERED PROFESSIONAL ENGINEER
88923
OREGON
Nov. 26, 2014
EDWARD SEAN RAHE
10/31/16
EXPIRES: 12/31/16

Jonathan D. Bunch, EI
Staff Engineer

E. Sean Rahe, PE
Department Manager

Reviewed by: Michael S. Place, PE – Principal Consultant

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FIGURES

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1 PROJECT INFORMATION

1.1 PROJECT AUTHORIZATION

This report presents the results of our geotechnical investigation performed for the proposed 90,000-Gallon Butane Tank to be located at the existing Philips 66 (P66) facility located within the Portland Terminal, situated at 5528 NW Doane Avenue in Portland, Oregon (see Figure 1, *Site Vicinity Map*). This exploration was performed for Sunoco Logistics Partners, LP (Sunoco Logistics), in general accordance with PSI Proposal No. 0704-191429R1, dated October 19, 2016. PSI's services were authorized by Mr. Rick Voytek with Sunoco Logistics Service on October 19, 2016.

1.2 PROJECT DESCRIPTION

Project information was provided by Mr. Rick Voytek of Sunoco Logistics Partners, LP (Sunoco Logistics) on September 28 and 29, 2016, via email and telephone. PSI has also reviewed an undated drawing titled, "*P66 Portland Butane Overhead Final*", which contains an aerial view of the project site indicating the approximate locations of the proposed butane tank and equipment placement.

PSI understands that Sunoco Logistics is planning on constructing a new butane blending facility at the existing Philips 66 Portland terminal facility, which will include the construction of a new 90,000-gallon above-ground butane tank (measuring approximately 12 feet in diameter and 140 feet in length), offload station, related equipment, pumps, and piping. The new tank will be located near the southeast section of the existing facility. PSI anticipates that the new butane tank will supported by two concrete "saddle" spread footings near each end of the proposed 90,000-gallon butane storage tank. PSI anticipates that the 90,000-gallon butane tank will weigh approximately 160 kips empty. Structural loads were not provided to us; however, based on an approximate specific gravity of 0.6, PSI anticipates the new storage tank will weigh approximately 620 kips when full of liquid butane.

Should any of the above information or assumptions made by PSI be inconsistent with the planned construction, it is requested that you contact us immediately to allow us to make any necessary modifications to this report.

1.3 PURPOSE AND SCOPE-OF-SERVICES

The purpose of this exploration was to evaluate the subsurface conditions at the site and to develop geotechnical design criteria for support of foundations for the planned project. PSI has also completed a Site-Specific Hazard Study at the project site. The scope of the exploration and analysis included a reconnaissance of the project site, completion of Cone Penetration Test (CPT) soundings, completion of geophysical testing, an engineering analysis and evaluation of the subsurface materials encountered, analysis of the seismic hazards, and the preparation of this report.

As directed by the client, PSI did not provide any service to investigate or detect the presence of moisture, mold or other biological contaminates in or around any structure, or any service that was designed or intended to prevent or lower the risk of the occurrence of the amplification of the same. Client acknowledges that mold is ubiquitous to the environment with mold amplification occurring when building materials are impacted by moisture. Client further acknowledges that site conditions are outside of PSI's control, and that mold amplification will likely occur, or continue to occur, in the presence of moisture. As such, PSI cannot and shall not be held responsible for the occurrence or recurrence of mold amplification.

1.3.1 FIELD EXPLORATION PROGRAM

PSI investigated the subsurface materials and conditions on October 20, 2016 and October 21, 2016. The field activities consisted of 3 Cone Penetration Tests (CPTs) and geophysical testing using refraction microtremor (ReMi) methods (see Figure 2, *Site Exploration Map* for approximate locations). PSI subcontracted Terra Hydr to vacuum excavate around each CPT location in general accordance with Sunoco Logistics and Philips 66 requirements. The CPTs were performed by Oregon Geotechnical Explorations, Inc. (OGE) and are designated as CPT-01 and CPT-02 for the soundings near the proposed butane tank location, and CPT-03 for the sounding near the planned equipment placement. Vacuum excavation was performed at the locations of CPT-01 and CPT-02 to depths of approximately 6 feet below existing ground surface (bgs), and to depths of approximately 11 feet bgs at the location of CPT-03. The planned depths of the CPTs were to 60 feet bgs, or to refusal. Tip refusal was encountered at the locations of CPT-01 and CPT-02 at a depth of approximately 48 and 46½ feet bgs, respectively. Tip refusal was encountered at the location of CPT-03 at a depth of approximately 42½ feet bgs.

The soil profile shown on the CPT logs represent the conditions only at actual exploration location. Variations may occur and should be expected. The stratifications represent the approximate boundary between subsurface materials; the actual transition may be gradual.

Cone Penetration Test with Pore-Pressure Readings (CPTu)

CPTu is an in-situ testing method used to determine the geotechnical engineering properties of soils and to delineate soil lithology. CPTu data is commonly used in the analysis and design of foundations. CPTu probing is a fast and cost-effective method for identifying subsurface soil types and evaluating the engineering properties of soils.

During a CPTu, the electric cone (tip angle 60°, section area 10 cm²) and the sounding rods are pushed continuously into the ground. Intermittent measurements of the cone resistance (q_c), sleeve friction (f_s), and pore pressure (u) are measured and recorded by the electric cone while it is being pushed into the ground. The measurements from a CPTu can be used to correlate a multitude of geotechnical parameters, including:

- Effective friction angle (φ', degree)
- Coefficient of consolidation (C_v, cm^2/sec)
- Overconsolidation Ratio (OCR)
- Undrained shear strength (s_u)

The results of the measured and correlated data are used in various geotechnical analyses, including: soil behavior type, soil bearing capacity, estimated settlement, liquefaction settlement, lateral spread, foundation-design criteria, slope stability, and seismic site class.

Refraction Microtremor (ReMi)

One Refraction Microtremor (ReMi) array was performed at the project site. The ReMi method uses standard P-wave recording equipment and ambient noise to determine shear-wave velocities. The equipment used for our ReMi evaluation included a Seismic Source DAQLink III 24-Bit ADC acquisition system and STC-85 - SM-4 10-hertz geophones developed by Seismic Source Technology. Field acquisition of the data incorporated 24 geophone locations with equal spacing of 13 feet. Data was recorded at various sample intervals and various sampling rates per channel at gains of 1 and 16. SeisOpt ReMi Version 4.0 (Vspect and Disper modules) software developed by Optim LLC was used to process the collected data, and to create shear wave velocity profiles. To provide a robust data profile, both individual recordings and multiple summed (stacked) recordings were evaluated.

Each individual record of the traces is pre-processed to reduce or eliminate anomalies in the raw data. The data is then processed to produce a velocity spectrum. This process involves computing a surface wave, phase velocity dispersion spectral ratio image by p-tau and Fourier transforms across the array. This process is described in the document titled, "*Faster, Better: Shear-wave Velocity to 100 Meters Depth from Refraction Microtremor Arrays*", Bulletin of the Seismological Society of America by Louie, J, N. (2001). The resulting spectrum is in the slowness-frequency (p-f) domain. The p-f transformation helps segregate the Rayleigh Wave arrivals from other surface waves, body waves, sound waves, etc. The p-f image is generated for each record, and a final p-f image for each test is generated by combining some, or all, of the individual images.

The fundamental mode dispersion curve on the final p-f image can be seen as a distinct trend from the aliasing and wave-field transformation truncation artifact trends in the spectra. Once the fundamental mode dispersion curve is visually interpreted, data points along this curve are picked. Using the picked data points, an interactive forward-modeling process is used to model a shear wave velocity profile, with a resulting dispersion curve that approximately matches the picked data points. The process and resulting velocity profiles are able to identify the various velocity layers in the subsurface, including velocity inversions within the profile.

The results of the ReMi testing indicates that the weighted-average shear wave velocity in the upper 100 feet of the project site (VS_{100}) is approximately 1,022 feet per second (i.e., the weighted-average shear wave velocity in the upper 30 meters of the project site [VS_{30}] is approximately 312 meters per second). These results indicate that the project site is classified as a Site Class "D", in accordance with ASCE 7-10.

1.3.2 SITE-SPECIFIC SEISMIC HAZARD STUDY

The site-specific seismic hazard study (SHA) has been completed to satisfy the requirements of the 2014 Oregon Structural Specialty Code (OSSC). The 2014 OSSC is predominantly based on the 2012 International Building Code (IBC). PSI performed this ground motion hazard analysis according to the updated provisions provided in FEMA 750 (2009) and ASCE 7 (2010), which are incorporated into the 2012 International Building Code (IBC) and 2014 OSSC.

The purpose of the SHA study was to evaluate the potential seismic hazards associated with regional and local seismicity and to estimate the effect those hazards might have on the site. PSI's work was based on the potential for regional and local seismic activity, as described in the existing scientific literature, and on subsurface conditions at the site, interpreted from geotechnical explorations and geophysical measurements made in the vicinity of the site. Specifically, PSI's Scope-of-Services for this site-specific SHA study included the following tasks:

1) A review of the literature, including published papers, maps, open-file reports, seismic histories and catalogs, works' in progress, and other sources of information regarding the tectonic setting, regional and local geology, and historical seismic activity that might have a significant effect on the site.
2) Compilation, examination, and evaluation of existing subsurface data gathered at and in the vicinity of the site, including classification and laboratory analyses of soil samples, rock quality data and shear wave velocity measurements. This information was used to prepare a generalized subsurface profile for the site.
3) Identification of the potential seismic events appropriate for the site and characterization of those events in terms of a series of generalized design events.
4) Office studies, based on the generalized subsurface profile and the generalized design earthquakes, resulting in conclusions and recommendations concerning:
    a. Specific seismic events that might have a significant effect on the site;
    b. Potential for seismic energy amplification at the site; and,
    c. Recommended site-specific acceleration response spectrum for the site.
5) The U.S. Geological Survey (USGS) database was examined for recorded earthquakes within 1000 km of the site and at least a moment magnitude (M_W) of 4, or that caused ground shaking at the site more intense than the Modified Mercalli III intensity.

6) The 2008 USGS probabilistic seismic hazard deaggregation was performed for the project site location for a 2,475-year return period (2% probability of exceedance in 50 years). USGS 2008 provides result for the B/C interface (V_{s30} = 760 m/sec).

7) Probabilistic seismic hazard analysis (PSHA) was performed using EZ-Frisk™ Version 7.65 (Build 004) by Fugro Consultants, Inc. The PSHA was based on identified seismic sources, appropriate attenuation relationships for the site using a measured shear wave velocity (V_{s30}), and the maximum rotational component of motion (MRC). PSI measured the V_{s30} based on our geophysical testing and our subsurface exploration at this project site. The PSHA was used to develop site specific bedrock response spectra for 2,475-year recurrence interval earthquakes.

8) Recommended response spectra are provided based on our site-specific analysis in accordance with ASCE 07-10 using the 2008 USGS national seismic hazard maps.

9) Other seismic hazards, including earthquake-induced landslides, regional subsidence, and fault displacement were evaluated.

2 SITE AND SUBSURFACE CONDITIONS

2.1 SITE DESCRIPTION

The proposed 90,000-gallon butane tank and equipment is to be located at the existing Phillips 66 Portland terminal facility, situated at 5528 NW Doane Avenue in Portland, Oregon. The project site is located approximately 1/5-mile to the south of the intersection of NW Front Avenue and NW Doane Avenue. The project site is bordered by oil and gas facilities to the north, east and west, and Highway 30 and the Portland Hills to the south. The Willamette River is located approximately 2/5-mile to the northeast of the project site. The surface of the site is predominantly covered with asphaltic concrete or gravel fill.

The current location of the proposed 90,000-gallon butane storage tank is occupied by an open-air storage facility measuring approximately 1,100 square feet in plan area. PSI understands that this existing structure will be demolished prior to the construction of the planned butane tank and related equipment.

2.1.1 TOPOGRAPHY

Based on available topographic information at the project site, PSI anticipates that the site grades in the area of the proposed storage tank will be relatively level and near an elevation of about 40 feet above the Mean Sea Level (MSL). Elevations near the site range from about 40 feet above MSL near the proposed butane, to about 37 feet near the planned equipment placement. PSI anticipates that the final site grades will be within about 2 feet of the existing site grades.

2.2 GEOLOGIC SETTING

On a regional scale, the site lies at the northern end of the Willamette Valley; a broad, gently deformed, north-south-trending topographic feature separating the Coast Range to the west from the Cascade Mountains to the east. The valley lies approximately 150 km inland from the surface expression of the Cascadia Subduction Zone, an active plate boundary along which remnants of the Farallon Plate (the Gorda, Juan de Fuca, and Explorer plates) are being subducted beneath the western edge of the North American continent. The configuration of these plates and the location, extent, and geometry of the surface expression of the subduction zone are shown schematically on Figure 4, "*Tectonic Map of the Pacific Northwest*". The subduction zone is a broad, eastward-dipping zone of contact between the upper portion of the subducting slabs of the Gorda, Juan de Fuca, and Explorer plates and the over-riding North America Plate. Although seismic activity is clearly associated with converging plate margins in other parts of the world, there is little direct evidence of significant seismic activity attributable to the Cascadia Subduction Zone.

On a local scale, the site lies in the west-central portion of the Portland Basin, a large, well-defined, northwest-trending structural basin bounded by high-angle, northwest-trending, right-lateral strike-slip faults generally considered to be seismogenic. The distribution of these faults relative to the site is depicted on Figure 3, "*Geologic Map of Portland*"; Figure 4 Tectonic Map of the Pacific Northwest; and Figure 5, "*USGS Vancouver – Portland Fault Map*". Within the basin, information regarding the location and extent of discrete faults is lacking, although a limited number of intra-basinal faults have been mapped on the basis of stratigraphic and geophysical evidence. The relationship between specific earthquakes and individual faults in the Portland area is not well understood, since few of the faults in the area are expressed clearly at the ground surface, and the foci of local earthquakes have not been located with precision.

Because of the proximity of the site to the Cascadia Subduction Zone and its location within the fault bounded Portland Basin, three distinctly different sources of seismic activity contribute to the potential for the occurrence of damaging earthquakes. Each of these sources is generally considered to be capable of producing damaging earthquakes. Two of these sources are associated with deep-seated tectonic activity related to the subduction zone; the third is associated with movement on the local, relatively shallow structures within and adjacent to the Portland Basin.

Precise, quantitative information regarding historic seismic activity in the Pacific Northwest and in the Portland area is sparse. Events that may have occurred in the region prior to settlement of the Oregon Territory in the mid-nineteenth century are speculative and have not been clearly identified in terms of location, magnitude, or frequency. From the mid-nineteenth century to the time of the installation of the first dependable seismometers in the area (about 1940), reliable information regarding location and magnitude is not available, although rough estimates of these parameters have been based on records of eyewitness accounts. Since about 1940, seismographic records of increasing sophistication and accuracy are available for local events larger than about 3.5 (M_L). For this project, we examined a catalog (Open File Report 0-94-04) obtained from the Oregon Department of Geology and Mineral Industries (DOGAMI) containing a list of those earthquakes known to have occurred in Oregon during the period 1883-1993. We searched this catalog for all known earthquakes within the area bounded by 46°00' N latitude on the north, 45°00' N latitude on the south, 122°00' W longitude on the east, and 123°20' W longitude on the west. This area includes a 50-km radius of the project site. In addition we searched the Advanced National Seismic System (ANSS) catalog for all known earthquakes within the area bounded by 46°30' N latitude on the north, 42°00' N latitude on the south, 117°00' W longitude on the east, and 125°00' W longitude on the west. This area essentially covers Oregon and southern Washington states. Recent events that may have generated measurable accelerations in the vicinity of the project site are the 1962 Vancouver Earthquake and the 1993 Scotts Mills Earthquake. The larger of these events, the M_L 5.0 Vancouver Earthquake of 1962, produced peak horizontal accelerations of approximately 0.14 g at Portland State University, approximately 12 km southeast of the site (Dehlinger, et al., 1963).

2.2.1 LOCAL GEOLOGY

The site is located between the east flank of the Tualatin Mountains and the west bank of the Willamette River, and in the area mapped as Quaternary alluvium and artificial fill. The alluvial soils are derived from depositional sequences of the Willamette River.

Based on nearby well logs, available geotechnical explorations by others, and published geologic information, PSI anticipates that the alluvial and fill materials are underlain by the Troutdale Formation. The Troutdale Formation generally consists of friable to moderately strong conglomerates (i.e., gravel and cobbles to siltstone, sandstone, and claystone). The Troutdale Formation was generally deposited approximately 13,000 to 15,000 years ago by a series of catastrophic floods from the failure of the ice dams at glacial Lake Missoula. These catastrophic floods deposited large amounts of sediments along the Columbia River and Willamette River Basins. The Troutdale Formation is underlain by Columbia River Basalt.

The Tualatin Mountains are located to the east of the project site and locally referred to as the “Portland Hills” or the “West Hills”. The Portland Hills are characterized by Miocene basalt flows of the Columbia Group which are described as subaireal basalt and minor andesite lava and breccia flows. The Portland Hills fault is located near the east side of the range, and is interpreted to be part of a larger northwest-trending, dextral wrench fault zone.

According to Oregon Department of Geology and Mineral Industries (DOGAMI) IMS-1 the project is listed on the boundary of Zone A and Zone B –“moderate” to “high” relative earthquake hazard, which is based on susceptibility to liquefaction, landslides and amplification during seismic events. Several faults are mapped near the project site. Each fault and its distance to the project site is summarized in Table 1.

2.3 SEISMIC AND TECTONIC SETTING

Due to the limited history of earthquakes in Oregon, the geologic and seismologic information available for identifying the nature of the seismicity at the site is incomplete, and large uncertainties are associated with any estimates of the probable magnitude, location, and frequency of occurrence of earthquakes that might affect the site. For this reason, several methods were used to model the seismic sources during evaluation of seismic hazard at this site. This study has relied on existing information, primarily from published articles and the USGS Quaternary fault database, to develop the input parameters for the PSHA. The PSHA input parameters generally consist of: maximum earthquake magnitude, slip rate (rate of strain accumulation), and recurrence interval (Personius, 2002).

The information that is available indicates that the seismic hazards at the site can be grouped into three independent categories: subduction zone events related to sudden slip between the upper surface of the Juan de Fuca plate and the lower surface of the North American plate, subcrustal events related to deformation and volume changes within the subducted mass of the Juan de Fuca plate, and local crustal events associated with movement on shallow, local faults within and adjacent to the Portland Basin. The tectonic setting is depicted on Figure 4, "*USGS Vancouer – Portland Fault Map*". Based on our review of currently available information, we have developed generalized design earthquakes for each of these categories. The design earthquakes are characterized by three important properties: size, location relative to the subject site, and the peak horizontal bedrock accelerations produced by the event. In this study, size is expressed in Richter (local) magnitude (M_L), surface wave magnitude (M_S), Japanese Meteorological Association magnitude (M_{JMA}), or moment magnitude (M_w); location is expressed as epicentral or focal distance, measured radially from the subject site in kilometers; and peak horizontal bedrock accelerations are expressed in gravities (1 g = 980.6 cm/sec/sec).

2.3.1 CASCADIA SUBDUCTION ZONE (CSZ)

The CSZ is a megathrust structure that forms the convergent plate boundary between the subducting Explorer, Juan de Fuca, and Gorda Plates and the overriding North America Plate, and extends from offshore northern California to southern British Columbia. Subduction is driven by eastward movement of the Explorer, Juan de Fuca, and Gorda Plates due to sea-floor spreading at the Gorda-Juan de Fuca-Explorer Mid-Ocean Ridge System. The subduction plates are the remnants of the Farallon Plate, which once underlay most of the eastern Pacific and has been converging with the North America Plate since at least the Jurassic period (Atwater, 1970; Duncan and Kulm, 1989). Tectonic elements associated with the subduction zone include: 1) an accretionary wedge of sediments deformed by a broad fold and thrust belt and east-striking strike-slip faults; 2) a forearc basin of sedimentary and igneous rocks that accumulated during plate collision, broken in places by minor Quaternary faults and folds; and 3) a volcanic arc (Cascade Range) consisting of Eocene through Quaternary volcanic rocks, active andesitic volcanoes, and numerous, mostly extensional, Quaternary faults. The historic seismicity on the CSZ is limited. There are numerous records of intraplate events on the Gorda block and in the Puget Sound area; however, there are few or no records of these in Central CSZ. Geological studies show that great megathrust earthquakes have occurred repeatedly in the past 7,000 years (e.g., Atwater and others, 1995; Clague, 1997; Goldfinger, 2003; and Kelsey, 2005), and geodetic studies (e.g., Hyndman and Wang, 1995; Savage, et al., 2000) indicate rate of strain accumulation consistent with the assumption that the CSZ is locked beneath offshore northern California, Oregon, Washington, and southern British Columbia (Fluck and others, 1997; Wang, et al., 2001). Numerous geological and geophysical studies suggest the CSZ may be segmented (Hughes and Carr, 1980; Weaver and Michaelson, 1985; Guffanti and Weaver, 1988; Goldfinger, 1994; Kelsey, et al., 1994; Mitchell, et al., 1994; Personius, 1995; Nelson and Personius, 1996; Witter, 1999), but the most recent studies suggest that for the last great earthquake in 1700, most of the subduction zone ruptured in a single Mw 9 earthquake (Satake, et al., 1996; Atwater and Hemphill-Haley; Clague, et al., 2000).

The surface trace of the subduction zone megathrust is located offshore in deep water, so paleoseismic studies have focused on "off fault" evidence of subduction zone earthquakes, such as coseismic uplift and subsidence, earthquake-induced turbidite and tsunami records, and liquefaction features caused by seismic shaking. However, it is difficult to discern whether some of these paleoseismic features are related to displacements on crustal faults, which may or may not deform concurrent with subduction zone earthquakes (McNeill, et al., 1998; Yeats, et al., 2001; Kelsey, et al., 2002; Witter, et al., 2003).

Studies indicate coastal subsidence, tsunamis, liquefaction, and turbidite triggering consistent with a massive earthquake on the CSZ about 300 years ago. Tree rings in cedars rooted in the youngest buried soil beneath wetlands in southwestern Washington date tree death from submergence to between August AD 1699 and May AD 1700 (Atwater, et al., 1991; Atwater and Yamaguchi, 1991; Yamaguchi, et al., 1997; Jacoby, et al., 1997; Benson, et al., 2001). Historical documents from Japanese harbors inundated by a tsunami and trans-Pacific tsunami modeling show that the tsunami from a Cascadia megathrust earthquake was generated by a M_w =9 earthquake on the subduction zone on January 26, 1700 (Satake, et al., 1996; 2003).

Numerous detailed studies of coastal subsidence, tsunamis, and turbidites yield a wide range of recurrence intervals, but the most complete records (>4,000 years) indicate average intervals of 350 to 600 years between great earthquakes on the CSZ (Adams, 1990; Atwater and Hemphill-Haley, 1997; Witter, 1999; Clague, et al., 2000; Goldfinger, et al., 2002; Kelsey, et al., 2002; Kelsey, et al., 2005; Witter, et al., 2003). Magnetic anomaly studies on the Juan de Fuca plate and geodetic studies indicate a rate of oblique convergence of about 35 to 45 mm/yr in a northeast direction across the subduction zone. The total structure length is approximately 754 km. Fault rupture is expected to produce estimated M_w of 8.3 to 9.0 earthquakes.

2.3.2 SUBCRUSTAL EVENT

Estimates of the probable size, location, and frequency of subcrustal events in the Pacific Northwest are generally based on comparisons of the Cascadia Subduction Zone with active convergent plate margins in other parts of the world and on the historical seismic record for the region surrounding Puget Sound, where significant events known to have occurred within the subducting Juan de Fuca plate have been recorded. Published estimates of the probable maximum size of these events range from moment magnitude M_w of 7.0 to 7.5. Published information regarding the location and geometry of the subduction zone indicates that minimum focal distances of 40 to 60 km (measured from Portland) are probable (Weaver and Shedlock, 1989). Estimates of recurrence intervals applicable to the Portland area are not available.

2.3.3 LOCAL CRUSTAL EVENT

The history of local seismic activity is commonly used as a basis for determining the size and frequency to be expected of local crustal events. Although the historical record of local earthquakes is relatively short (the earliest reported seismic event in the area occurred in 1841), it can serve as a guide for estimating the potential for seismic activity in the area. A significant earthquake could occur on a local fault near the site within the design life of the proposed structure. Such an event would cause ground shaking at the site that could be more intense than the CSZ event, though the duration would be shorter. The precise relationship between specific earthquakes and individual faults is not well understood, since few of the faults in the area are expressed at the ground surface, and the foci of the observed earthquakes have not been located with precision.

The longest mapped fault in the vicinity of the subject site is the Portland Hills Fault, which extends for about 50 km. Rupture of a fixed maximum fraction (half of the length is generally used) of this fault length would produce a characteristic earthquake with a magnitude of approximately M_L= 6.4. It should be noted that the rupture length-magnitude relationship, though useful in estimating the size of possible events, does not indicate the frequency with which those events might occur. A table of the mapped faults closest to the site is provided in Table 1.

Table 1: Mapped Crustal Faults (within 25 Miles)

Fault Name	**Approximate Distance and Direction from Site (miles)**
Portland Hills Fault	0.1, Southwest
East Bank Fault	1.3, Northeast
Oatfield Fault	2.3, Southwest
Beaverton Fault Zone	6.4, Southwest
Helvetia Fault	8.6, East
Canby-Mollala Fault	9.2, Southwest
Grant Butte Fault	10.7, Southeast
Damascus-Tickle Creek Fault Zone	10.8, Southeast
Lacamas Lake Fault	13.8, Northeast
Gales Creek Fault Zone	19.7, Southwest
Newberg Fault	20.7, Southwest

The closest faults to the site are the Portland Hills Fault, the East Bank Fault, and the Oatfield Fault. The mapped faults are depicted on Figure 4, "*USGS Vancouver – Portland Fault Map*".

A summary of published USGS deaggregation data for the proposed improvements is provided below in Table 2 with respect to the seismic source, distance from site, and percent contribution to the seismic hazard based on the USGS probabilistic model and seismic hazard curve:

Table 2: USGS 2008 Deaggregation

Summary statistics for above PSHA PGA deaggregation, R=distance, e=epsilon:
Contribution from this GMPE(%): 100.0
Mean src-site R= 58.5 km; M= 7.55; eps0= 1.11. Mean calculated for all sources.
Modal src-site R= 84.9 km; M= 9.00; eps0= 1.04 from peak (R,M) bin
MODE R*= 84.9km; M*= 9.00; EPS.INTERVAL: 1 to 2 sigma % CONTRIB.= 8.633
Modal-source dmetric: distance to rupture surface (Rrup or Rcd)

Principal sources (faults, subduction, random seismicity having > 3% contribution)

Source Category:	% contr.	R(km)	M	epsilon0 (mean values).
Cascadia M8.3-M8.7 Floating	11.10	94.9	8.54	1.57
Cascadia Megathrust	27.96	94.5	9.02	1.16
Wash-Oreg Cascades-West faults	17.32	2.4	6.78	0.02
WUS Compr crustal gridded	14.13	9.6	6.05	1.19
50-km Deep Intraplate	28.03	66.0	6.90	1.46

Individual fault hazard details if its contribution to mean hazard > 2%:

Fault ID	% contr.	Rcd(km)	M	epsilon0	Site-to-src azimuth(d)
Portland Hills fault	4.91	0.2	7.00	-0.26	-131.5
Portland Hills fault, GR	10.91	1.8	6.74	-0.05	-133.0

2.4 HISTORICAL SEISMICITY

There is a limited database of historic earthquakes for Oregon due to a relatively short period of written records (approximately 170 years) and a regional rate of seismicity that is lower than that in the neighboring states of California and Washington. Table 4 lists the largest historical earthquakes felt in Oregon. Figure 6 depicts historical seismicity in Western Oregon on the central and southern CSZ (Burns, 2008). As shown on the figure, the Portland area is located in a zone of higher historic seismicity. Over 500 km to the south, the subducting Gorda Plate has been subject to considerably more historic earthquakes, primarily offshore of northern California and associated with the subduction trench axis. The historic record of moderate-sized earthquakes (M 5.0 to 7.0) in both the Puget Sound and Gorda Plate areas is generally associated with intraslab earthquakes. In the Puget Sound area, these moderate to large earthquakes are deep (40 to 60 km) and over 200 km from the deformation front of the subduction zone. At the Gorda Block, the earthquakes are shallower (up to 40 km) and located along the deformation front. Wong (2005) hypothesizes that due to subduction zone geometry, geophysical conditions and local geology, Oregon may not be subject to intra-slab earthquakes.

Table 3: Largest historical Earthquakes Felt in Oregon

Date	Latitude	Longitude	Magnitude	Modified Mercalli Intensity	Location
11/23/1873			6.8		Near Brookings, OR
10/12/1877	45.5	122.5	5.3	VII	Portland, OR
7/151936			6.4		Milton-Freewater, OR
4/13/1949	47.1	122.7	7	VIII	Olympia, WA
11/5/1962	45.6	122.6	5.3		Portland, OR
4/29/1965	47.4	122.4	6.8	VIII	Puget Sound, WA
1968	42.3	119.8	5.1		Adel, OR
4/12/1976			4.8		Maupin, OR
4/25/1992			7		Cape Mendocino, CA
3/25/1993	45.04	122.6	5.6		Scotts Mills, OR
9/21/1993	42.4	122.09	6		Klamath Falls, OR
2/28/2001	47.2	122.7	6.8		Nisqually, WA
6/14/2005	41.33	125.86	7	IV	near Crescent City, CA

Notes: 1) Data from Advanced National Seismic System (ANSS), US Geological Survey (USGS), and Johnson A. and Madin, I, 1994, Earthquake Database for Oregon, 1983 through October 25, 1993: Oregon Department of Geology and Mineral Industries Open File Report 0-94-4.
2) Magnitudes are M_s, M_L, mb or based on felt area of Modified Mercalli Intensity. Maximum reported magnitudes are listed on the table.

2.5 SUBSURFACE CONDITIONS

The area of the proposed butane tank is generally covered at the surface with asphaltic concrete and an existing, gazebo-type structure with a concrete floor, currently in use as a storage facility. The surficial material at the location of CPT-01 consisted of approximately 4 inches of compacted gravel fill, underlain by approximately 2 inches of asphaltic concrete, underlain by approximately 18 to 24 inches of compacted gravel with sand and cobbles. The surficial material at the location of CPT-02 consisted of approximately 2 inches of asphalt, underlain by approximately 18 to 24 inches of compacted gravel with sand and cobbles. The surficial material encountered at the location CPT-03 generally consisted of approximately 2 inches of gravel fill.

Based on the CPT exploration and mapped geologic information in the area, PSI expects that “dredged fill” extends from beneath the surficial materials to a depth of about 20 to 25 feet bgs. The old fill material generally consists of compacted gravel with sand and cobbles underlain by very loose to medium dense fine to medium silty sand and silty clay. The dredged fill is underlain by medium stiff to stiff sandy silt/clay (i.e., alluvium). A distinct chemical odor was noticed during the vacuum excavation, and an oil-type sheen was observed floating on the water in the excavation and coating the side walls of the excavated areas.

Since the sounding locations were vacuum excavated to a depth of approximately 6 to 11 feet bgs, it is PSI’s opinion that the CPT data does not accurately represent the in-situ conditions of the soil between the surface and this depth.

2.5.1 GROUNDWATER INFORMATION

Based on our investigation, PSI estimates groundwater at depths between approximately 7½ to 12½ feet based on the results of the pore-pressure dissipation tests (included in Appendix A) during the CPT exploration. Table 4 below summarizes the depth of groundwater and the elevation of groundwater encountered at each CPT location.

Table 4: Depth and Elevation of Groundwater Encountered at CPT Locations

CPT Location	Approximate Elevation* (Feet above MSL)	Approximate Depth to Groundwater (Feet BGS)	Approximate Groundwater Elevation* (Feet above MSL)
CPT-01	40	9½	30½
CPT-02	39	7½	31½
CPT-03	37	12½	24½

*Elevations were estimated using Google Earth Pro
MSL – mean sea level
BGS – Below Existing Ground Surface

Groundwater levels on this site are likely to vary based on seasonal conditions and precipitation. Fluctuations in groundwater levels should be anticipated.

3 GROUND MOTION HAZARD ANALYSIS

PSI has conducted a Probabilistic Seismic Hazard Analysis (PSHA) and a Deterministic Seismic Hazard Analysis (DSHA) to develop seismic design response spectrum and design acceleration parameters for comparison to the general procedure spectrum and design parameters.

3.1 PROBABILISTIC SEISMIC HAZARD ANALYSIS

The input for a Probabilistic Seismic Hazard Analysis (PSHA) consists of three significant components:

1) Identification of earthquake sources, locations, and physical characteristics (e.g., dip angle, rupture width, length, etc.);
2) Characterization of the seismicity rate for each seismic source using an appropriate model (e.g., exponential or normal distribution); and,
3) Selection of empirical attenuation relationships that describe how the characteristics of the strong ground motions change as the waves propagate from the seismic source to a given site location.

These components include aleatory and epistemic uncertainties associated with our limited knowledge and understanding of the fault sources and their predicted behavior. Aleatory uncertainty describes the probabilistic randomness associated with estimating fault behavior and earthquakes. Epistemic uncertainty is associated with our incomplete knowledge or understanding of the seismic model or parameters. The PSHA method combines and incorporates these uncertainties to obtain a probabilistic ground motion, which is defined by the likelihood of an earthquake of a specific magnitude occurring within a specific length of time.

A logic tree is used to evaluate these uncertainties in a PSHA. A logic tree assigns each model parameter a "tree branch" and a relative weight (some fraction of 1.0), based on the level of confidence in that quantified parameter. Multiple levels of tree branches can be assigned corresponding to levels of confidence associated with factors such as fault location, appropriate recurrence model, or probability of activity. The seismic hazard is then calculated by summing up the weighted hazards, each calculated independently from the branches of the logic tree.

Probabilistic seismic hazard analyses are typically completed in one of two ways to generate ground surface earthquake characteristics:

1) A PSHA is completed using empirical attenuation relationships for estimating ground motion parameters (e.g., peak acceleration, acceleration response spectra) on bedrock. A dynamic soil response model is then used to simulate the propagation of representative earthquake motions from a defined bedrock layer through a soil column, with pertinent soil properties identified through a

geotechnical investigation at the site. This modeling provides the characteristics of the design earthquake motions at specified depths of interest, usually at the ground surface or at depths representative of the proposed foundations.

2) The PSHA is completed using attenuation relationships derived from historical earthquake recording stations at soil sites. The individual attenuation relationships provide ground surface characteristics as a function of the site conditions at the recording station. In this procedure, the ground surface motions (i.e., PGA, PGV, response spectra) are obtained directly from the PSHA results.

Due to the relatively stiff soils underlying the site (V_{s30}= 760 feet/sec), we have used next generation attenuation relationships (NGA) that provide ground surface motions for the purposes of our seismic hazard evaluation. The ground surface motions for the CSZ were determined by applying the spectral amplification rations determined for the local crustal events.

3.1.1 PROBABILISTIC CONSIDERATIONS

The probability of occurrence of an earthquake of a specific magnitude at a given location is commonly expressed by its return period, i.e., the average length of time between successive occurrences of an earthquake of that size or larger at that location. The return period of a design earthquake can be calculated once a project design life and some measure of the acceptable risk that the design earthquake might occur or be exceeded are specified. For this project, a design life of 50 years and an acceptable probability of exceedance of 2% have been considered, in accordance with the requirements of the 2014 OSSC. The relationship between the return period, the design life, and the exceedance probability is such that the choice of a 50-year design life and a 2% probability of exceedance result in a return period of approximately 2,475 years.

3.2 DETERMINISTIC SEISMIC HAZARD ANALYSIS

PSI performed a screening for the Deterministic Seismic Hazard Analysis (DSHA) concurrently with the PSHA to estimate the ground motions at the site, and to help define the risk-targeted maximum considered earthquake (MCE_R) in accordance with Section 21.2.2 of ASCE 7 (2010). A DSHA is completed by estimating ground motions for characteristic magnitude earthquakes at the location of active seismic sources in the region. Typically, the characteristic earthquakes are analyzed using an average of the same attenuation relationships used for the PSHA for consistency.

The deterministic spectral response acceleration at each period is defined as the largest 84th percentile, 5 percent damped spectral response acceleration in the direction of maximum horizontal response computed, i.e., the maximum rotated component (MRC), at that period for characteristic earthquakes on all known active faults within the region. The ordinates of the deterministic ground motions response spectrum should not be taken as lower than the corresponding ordinates of the response spectrum (i.e., the "Deterministic Lower Limit") determined in accordance with Figure 21.2-1, where F_a and F_v are determined using Tables 11.4-1 and 11.4-2, respectively.

Deaggregation of the PSHA indicates that the seismic sources contributing the most seismic hazard to this project site are: the M_w 9.0 megathrust CSZ earthquake and the M_w 6.9 intraplate CSZ earthquake. The DSHA was evaluated with respect to the "Deterministic Lower Limit", which was calculated based on ASCE 7-10, Figure 21.2-1. PSI concluded that the DSHA was higher than the "Deterministic Lower Limit".

3.3 RESULTS

ASCE 7 (2010) defines the site-specific MCE_R as the lower of the probabilistic MCE_R and the deterministic MCE_R. The ground motion associated with the probabilistic MCE_R is defined as a 2 percent in 50-year hazard level spectrum with 5 percent damping. The probabilistic MCE_R was determined to be less than the deterministic MCE_R. The probabilistic MCE_R has been adjusted by the risk-targeted coefficients (C_{RS} and C_{R1}) in Chapter 22 of ASCE 7-10, and reduced by a factor of 2/3, in accordance with Section 21.3, to obtain the design response spectrum, S_a.

As indicated in ASCE 7-10, when the site-specific procedure is used to determine the ground motion in accordance with Section 21.3, the parameter S_{DS} shall be taken as the spectral acceleration, S_a, obtained from the site specific spectra at a period of 0.2s, except that it shall not be taken less than 90 percent of the peak spectral acceleration, S_a, at any period larger than 0.2 s. The parameter S_{D1} shall be taken as the greater of the spectral acceleration, S_a, at a period of 1 s or two times the spectral acceleration, S_a, at a period of 2 sec. The parameters S_{MS} and S_{M1} shall be taken as 1.5 times S_{DS} and S_{D1}, respectively. The value obtained as described above shall not be less than 80 percent of the values determined in accordance with ASCE 7-10 Section 11.4.3 for S_{MS} and S_{M1} and Section 11.4.4 for S_{DS} and S_{D1}. The results of the evaluation are shown on Figures 9 through 12, and summarized in Table 5 which presents the comparison of the response spectra. The recommended spectrum is also graphically depicted on Figure 12.

Table 5: Recommended Site-Specific Design Response Spectrum

Spectral Period	2% in 50 Year Mean Prob. (1)	Risk Coeff. (C_R) (2)	Prob. MCE$_R$ (3)	84th Percentile Mean Det. (4)	Det. Lower Limit (5)	Site-Specific MCE$_R$ (6)	2/3 Site-Specific MCE$_R$ (7)	General Design Response (8)	80% of General Design Response (9)	Recommended Site-Specific Design Response Spectrum (10)
(seconds)	*(g)*	*(g)*	*(g)*	*(g)*	*(g)*	*(g)*	*(g)*	*(g)*	*(g)*	*(g)*
0.000	**0.637**	**0.903**	**0.576**	**1.271**	**0.661**	**0.576**	**0.384**	**0.293**	**0.235**	**0.293**
0.087	1.050	0.903	0.948	1.814	1.584	0.948	0.632	0.603	0.482	0.603
0.100	1.122	0.903	1.013	1.920	1.652	1.013	0.675	0.649	0.519	0.649
0.124	1.228	0.903	1.109	2.083	1.652	1.109	0.739	0.733	0.587	0.733
0.200	**1.421**	**0.903**	**1.283**	**2.402**	**1.652**	**1.283**	**0.855**	**0.733**	**0.587**	**0.733**
0.300	1.368	0.899	1.230	2.554	1.652	1.230	0.820	0.733	0.587	0.733
0.400	1.314	0.896	1.177	2.654	1.652	1.177	0.784	0.733	0.587	0.733
0.435	1.274	0.894	1.139	2.663	1.565	1.139	0.759	0.733	0.587	0.733
0.500	1.214	0.892	1.083	2.677	1.341	1.083	0.722	0.733	0.587	0.722
0.600	1.094	0.888	0.971	2.570	1.264	0.971	0.648	0.733	0.587	0.648
0.619	1.080	0.887	0.958	2.553	1.174	0.958	0.639	0.733	0.587	0.639
0.700	1.028	0.884	0.909	2.485	1.043	0.909	0.606	0.648	0.519	0.606
0.800	0.963	0.881	0.848	2.369	0.939	0.848	0.565	0.567	0.454	0.565
0.900	0.888	0.877	0.779	2.223	0.854	0.779	0.519	0.504	0.403	0.504
1.000	**0.830**	**0.873**	**0.725**	**2.098**	**0.783**	**0.725**	**0.483**	**0.454**	**0.363**	**0.454**
1.100	0.766	0.873	0.668	1.945	0.722	0.668	0.446	0.413	0.330	0.413
1.200	0.714	0.873	0.623	1.810	0.671	0.623	0.415	0.378	0.303	0.378
1.300	0.665	0.873	0.581	1.690	0.626	0.581	0.387	0.349	0.279	0.349
1.400	0.621	0.873	0.542	1.583	0.587	0.542	0.361	0.324	0.259	0.324
1.500	0.584	0.873	0.510	1.486	0.552	0.510	0.340	0.303	0.242	0.303
1.600	0.549	0.873	0.479	1.380	0.522	0.479	0.320	0.284	0.227	0.284
1.700	0.519	0.873	0.453	1.286	0.494	0.453	0.302	0.267	0.214	0.267
1.800	0.492	0.873	0.430	1.202	0.470	0.430	0.287	0.252	0.202	0.252
1.900	0.467	0.873	0.408	1.129	0.447	0.408	0.272	0.239	0.191	0.239
2.000	0.445	0.873	0.388	1.064	0.427	0.388	0.259	0.227	0.182	0.227
2.100	0.417	0.873	0.364	1.001	0.408	0.364	0.242	0.216	0.173	0.216
2.200	0.392	0.873	0.342	0.944	0.391	0.342	0.228	0.206	0.165	0.206
2.300	0.368	0.873	0.322	0.894	0.376	0.322	0.214	0.197	0.158	0.197
2.400	0.348	0.873	0.304	0.848	0.361	0.304	0.203	0.189	0.151	0.189
2.500	0.330	0.873	0.288	0.807	0.348	0.288	0.192	0.182	0.145	0.182
2.600	0.313	0.873	0.273	0.769	0.335	0.273	0.182	0.175	0.140	0.175
2.700	0.296	0.873	0.259	0.734	0.324	0.259	0.173	0.168	0.134	0.168
2.800	0.280	0.873	0.245	0.702	0.313	0.245	0.163	0.162	0.130	0.162
2.900	0.266	0.873	0.232	0.673	0.303	0.232	0.155	0.157	0.125	0.155
3.000	0.253	0.873	0.221	0.645	0.293	0.221	0.147	0.151	0.121	0.147
3.100	0.242	0.873	0.212	0.620	0.285	0.212	0.141	0.146	0.117	0.141
3.200	0.233	0.873	0.203	0.596	0.276	0.203	0.136	0.142	0.113	0.136
3.300	0.224	0.873	0.196	0.574	0.268	0.196	0.131	0.138	0.110	0.131
3.400	0.217	0.873	0.189	0.554	0.261	0.189	0.126	0.133	0.107	0.126

Table 5: Recommended Site-Specific Design Response Spectrum (Continued)

Spectral Period	2% in 50 Year Mean Prob. (1)	Risk Coeff. (C_R) (2)	Prob. MCE_R (3)	84th Percentile Mean Det. (4)	Det. Lower Limit (5)	Site-Specific MCE_R (6)	2/3 Site-Specific MCE_R (7)	General Design Response (8)	80% of General Design Response (9)	Recommended Site-Specific Design Response Spectrum (10)
(seconds)	_(g)_	_(g)_	_(g)_	_(g)_	_(g)_	_(g)_	_(g)_	_(g)_	_(g)_	_(g)_
3.500	0.209	0.873	0.183	0.535	0.254	0.183	0.122	0.130	0.104	0.122
3.600	0.203	0.873	0.177	0.517	0.247	0.177	0.118	0.126	0.101	0.118
3.700	0.196	0.873	0.171	0.500	0.241	0.171	0.114	0.123	0.098	0.114
3.800	0.188	0.873	0.164	0.484	0.235	0.164	0.110	0.119	0.096	0.110
3.900	0.182	0.873	0.159	0.469	0.000	0.159	0.106	0.116	0.093	0.106
4.000	0.176	0.873	0.153	0.454	0.000	0.153	0.102	0.113	0.091	0.102

Table 5 Notes:
(1) From EZ-Frisk PSHA output.
(2) From ASCE 7-10 Figures 22-17 and 22-18.
(3) = (1) x (2).
(4) From EZ-Frisk DSHA output.
(5) Calculated based on Fa and Fv per ASCE 7-10.
(6) The Lesser of (3) and (4).
(7) = (6) x2/3
(8) Calculated based on Sds, Sd1, T_L per ASCE 7-10, Section 11.4.5.
(9) = (8) x 0.8
(10) Generally = (7); capped by (8) and not less than (9).

3.4 SEISMIC HAZARD DISCUSSION

Based on our review of geologic maps, subsurface information, and fault research, there is limited evidence of Quaternary displacement of the Portland Hills Fault. The Portland Hills Fault is located approximately 0.1 km to the southwest. Due to the relatively close proximity of this known fault to the site the presence of other residual fault underlying the site cannot be ruled out. Based on the limited information about this fault and its low displacement rate (estimated to be less than 0.2 millimeters per year), it is our opinion that the potential for fault rupture at the site is low to moderate.

The project site is located approximately 86 kilometers (53 miles) upstream from the nearest mapped area of tsunami inundation; therefore, based on the location and elevation of the site, the risk of damage by tsunamis and or seiches at the site is absent.

Soil Liquefaction Potential and Settlement

Soil liquefaction is a mechanism by which loose, saturated, granular materials, such as sands and low-plasticity silts, temporarily lose strength during and immediately after a seismic event. Liquefaction occurs when saturated granular soils are subjected to cyclic loading, which distorts the soil structure and causes loosely packed groups of particles to collapse, increasing pore water pressure in the soil mass. As pore water pressure increases, the soil begins to lose strength and may even behave as a viscous liquid in the most extreme cases. As strength is lost, there is an increased risk of settlement, and an increased risk of lateral spreading and/or slope instability on sloping sites.

Based on IMS-1, as discussed in the *Geology* section of this report, the project site is mapped on the boundary of Liquefaction Hazard Category 2 and 3 for relative liquefaction hazards, which indicates that there may be 20 to 30 feet of “liquefiable material”, and that the groundwater table is between 15 and 30 feet below the ground surface.

A liquefaction settlement screening for the site has been completed. Based on PSI’s analyses of the field investigation results, it is estimated that settlement due to liquefaction will be on the order of 2½ to 3½ inches in a major earthquake (i.e., an earthquake with a moment magnitude M_w of 7.55 and an acceleration of 0.58g, based on the peak ground acceleration (PGA_M) from the site-specific ground motion hazard analysis. PSI estimates that differential settlement between the two foundations of the tank may be on the order of 1 inch. Both static and seismically-induced settlements should be considered in the foundation design of the proposed butane tank.

4 CONCLUSIONS AND RECOMMENDATIONS

The subsurface explorations indicate that the site is predominantly covered at the surface by asphaltic concrete pavement and gravel fill, and underlain by approximately 20 to 25 feet of fill materials (i.e., dredge fill), underlain by mostly silty sand and silty clay.

It is PSI's opinion that the proposed butane tank, pipe bridge, and related equipment can be supported by either shallow foundations or drilled piers at this project site, provided PSI's recommendations are followed. PSI estimates that a total static settlement of approximately 2 inches, and liquefaction-induced settlements on the order of 3 to 4 inches should be considered for footing design for the proposed butane tank; and flexible connections should be considered for managing anticipated settlements.

4.1 SITE PREPARATION

4.1.1 DEMOLITION AND SITE STRIPPING

Following demolition of the existing storage building at the location of the proposed butane tank, PSI recommends that the existing buried piping, where encountered, that is not completely removed or rerouted from below the proposed building footprints should be permanently capped, and filled with grout to prevent seepage or underground soil erosion in the future. Concrete structures and remnants of previous structures encountered during site excavation and site construction operations should be completely removed.

PSI recommends that, prior to construction, unsuitable materials be stripped and removed from the site, or stockpiled in non-settlement sensitive areas of the project site (e.g. landscaping areas or berms). Unsuitable materials include vegetation/organics, organic soils, undocumented fills, construction debris, etc. Unsuitable materials have the potentially to undergo high and variable volume changes when subjected to loads, resulting in detrimental performance of structures placed on or in these materials. Based on the results of PSI's field exploration, it is expected that approximately 20 to 25 feet of undocumented fills may be present across the site.

While PSI recommends that the undocumented fill soils be entirely removed from within the planned construction area, some or all of the fill soils could be left in place for support of the planned structures, providing the Owner accepts the risks associated in doing so. These risks include variable support characteristics and the possibility that organics or other unsuitable soil layer(s) could be present below or within fill deposits, resulting in an increased risk of detrimental settlement of the floor slab (and/or buried utilities) occurring. If these risks are unacceptable, then all fill soils must be removed as recommended and be replaced with structural fill.

Should some of all of the existing fill material be left in place, PSI recommends that at least 2 feet of the material below the planned butane tank foundations be removed and replaced with structural fill (as specified below).

The thickness of the fill is likely to vary throughout the site and other, possibly more extensive, deposits could be encountered during the site work activities. The exact depth of removal of these soils should be observed by PSI during the stripping activities.

4.1.2 WET WEATHER CONSTRUCTION

Where silty or clayey soils are exposed, the subcontractor must use care to protect the subgrade from disturbance by construction traffic, particularly during wet weather. Permanent cut and fill slopes should be limited to 2Horizontal:1Vertical (2H:1V) or flatter to minimize erosion and the risk of slope instability.

It may be prudent to use working blankets and haul roads constructed of imported granular material to provide equipment support and to protect the underlying subgrade during wet-weather conditions. Clean, coarse-graded fragmental rock with less than 5% passing the U.S. Standard No. 200 sieve (washed analysis), such as 4-inch-minus crushed rock (4”-0), capped with a leveling course of clean finer-graded rock, such as ¾-inch-minus (¾”-0), works well for this purpose. A typical haul-road section consists of 18 inches of 4”-0 crushed rock, overlaid by a 6-inch leveling course of ¾”-0 crushed rock. The sections may be reduced by 25 to 50% for areas of light construction activities that are anticipated to be subjected to limited truck traffic. The 4”-0 crushed rock thickness may be reduced by utilizing a geotextile fabric. PSI recommends the use of a geotextile fabric (overlapped by at least 12 inches at joints) between the granular material and the underlying subgrade as a separation to limit the movement of fines into the crushed rock. The use of a fabric tends to reduce maintenance of working blankets or haul roads during construction. PSI recommends the use of Mirafi 140N (or equivalent) geotextile fabric for separation. Where practicable, PSI recommends that the fill be placed so that the construction equipment remains on newly-placed fill soils and not on the exposed subgrade during fill placement.

Where silty and clayey (fine grained) soils are concerned, it has also been PSI’s experience that despite during warm, dry weather, the moisture content of the upper few feet of the fine grained soils that mantle the site will decrease, below this depth the moisture content of the soil tends to remain relatively unchanged and above the optimum moisture content required for proper compaction. As a result, the subcontractor must employ construction equipment and procedures that prevent disturbance and softening of the subgrade soils. The use of excavation equipment equipped with smooth-edged buckets for excavation, along with the concurrent placement of granular work pads tends to minimize the potential for subgrade disturbance. Subgrade disturbed during construction activities should be over-excavated to firm soil and backfilled with structural fill.

4.1.3 SUBGRADE PREPARATION

After the surficial and unsuitable materials have been stripped, PSI should observe the subgrade to identify any soft, unstable areas. Where organic, soft or otherwise unsuitable soils are identified, these unsuitable soils should be completely removed and replaced with structural fill. In areas where unsuitable soils are encountered and overexcavation occurs below footings, the overexcavation and structural fill should extend laterally a minimum distance that is equal to the depth of the excavation below the footing. The Contractor should provide a contingency for the repair of soft areas identified by the Geotechnical Engineer. Geotextile fabric and/or geotextile grid may be utilized to provide stabilization of the subgrade; however, more extensive subgrade stabilization measures may be needed upon observation of the subgrade.

4.1.4 FILL MATERIALS

Proper control of placement and compaction of new fills should be monitored by PSI. Fill materials should be placed in individual lifts not exceeding 12 inches in un-compacted thickness. Each lift is to be compacted to a minimum of 95 percent of the maximum dry density within 2 percent of the optimum moisture content, as determined in accordance with ASTM D1557 (modified Proctor). A sufficient number of in-place density tests should be performed on each lift of fill, as determined by the geotechnical engineer.

Tested structural fill materials that do not achieve either the required dry density or moisture content range shall be recorded, the location noted, and reported to the Contractor and Owner. A re-test of the area should be performed after the Contractor performs remedial measures.

Structural Fill

Fill placed at the project site should be installed as properly compacted structural fill. If imported structural fill is required, PSI recommends using granular material, especially if placement and compaction take place in wet weather. Imported granular material for structural fill should consist of pit-run or quarry-run rock, crushed rock, crushed gravel, or sand. The imported material should be well-graded between coarse and fine material, angular, have a plasticity index of 8 or less, and have less than 5 percent by weight passing the U.S. Standard No. 200 Sieve (75-μm).

Structural fill should be placed in lifts with a maximum un-compacted thickness of 12 inches, and compacted to not less than 95 percent of the maximum dry density within 2 percent of optimum moisture content, as determined by ASTM D1557 (modified Proctor). A sufficient number of in-place density tests should be performed on each lift of the fill.

Utility Trench Backfill

Utilities trenches should be backfilled with granular structural fill such as sand, sand and gravel, fragmented rock, or recycled concrete with constituents less than 2 inches in maximum diameter, and less than 5 percent passing the U.S. Standard No. 200 sieve (washed analysis).

Utility trench backfill should be placed in lifts with a maximum un-compacted thickness of 12 inches. Utility trenches should be compacted to not less than 95 percent of the maximum dry density within 2 percent of optimum moisture content, as determined by ASTM D1557 (modified Proctor), in the upper 3 feet of the final surface grade, and to about 90 percent below 3 feet. A sufficient number of in-place density tests should be performed on each lift of the fill. Compaction by jetting or flooding should not be permitted.

Drain Rock

Drain rock, or "free-draining" material, should have less than 2% passing the U.S. Standard No. 200 (75-μm) sieve (washed analysis). Examples of materials that would satisfy this requirement include ¾-inch to ¼-inch or 1½-inch to ¾-inch crushed rock.

4.2 EXCAVATIONS AND SLOPES

4.2.1 TEMPORARY SLOPES

It is PSI's opinion that temporary excavation slopes should be limited to 2Horizonal:1Vertical (2H:1V). To protect temporary excavation slopes from erosion caused by rainfall and subsequent runoff, the slopes should be covered with waterproof sheeting, and all surface drainage should be directed away from the excavation. In addition, PSI recommends that surcharge loads due to construction traffic, material laydown, excavation spoils, etc., not be allowed within a horizontal distance of one-half of the height from the top of the cut (i.e., H/2, where H is the height of the cut). In this regard, PSI recommends the use of fencing or barricades along the top of the cut to prevent this area from being subjected to surcharge loads.

The Contractor should recognize that the above recommendations will not guarantee that failure of temporary cut slopes will not occur.

4.2.2 TRENCH EXCAVATIONS

Excavations should be made in accordance with applicable Federal and State Occupational Safety and Health Administration regulations. Trenches in the near-surface silty soils at the site will likely require to be sloped due to the potential for caving. Actual inclinations will ultimately depend on the soil conditions encountered during earthwork. While PSI may provide certain approaches for trench excavations, the Contractor should be responsible for selecting the excavation technique, monitoring the trench excavations for safety, and providing shoring, as required, to protect personnel and adjacent improvements. The information provided below is for use by the Owner and Engineer and should not be interpreted to mean that PSI is assuming responsibility for the Contractor's actions or site safety.

The Contractor should be aware that excavation and shoring should conform to the requirements specified in the applicable local, state, and federal safety regulations, such as OSHA Health and Safety Standards for Excavations, 29 CFR Part 1926, or successor regulations. PSI understands that such regulations are being strictly enforced, and if not followed, the Contractor may be liable for substantial penalties.

Excavation and construction operations may expose the on-site soils to inclement weather conditions. The stability of exposed soils may deteriorate due to a change in moisture content or the action of heavy or repeated construction traffic. Accordingly, foundation and pavement area excavations must be protected from the elements and from the action of repetitive or heavy construction loadings. In addition, it is recommended that surcharge loads due to construction traffic, material laydown, excavation spoils, etc., not be allowed within a horizontal distance of H/2 from the top of the cut, where H is the height of the cut.

Permanent cut and fill slopes should be limited to 2H:1V or flatter to minimize erosion and the risk of slope instability.

4.3 FOUNDATIONS

4.3.1 SHALLOW FOUNDATIONS

It is PSI's opinion that the equipment at the project site can be supported by shallow foundations, provided PSI's recommendations are followed.

Foundation support for the new structures can be provided by conventional spread footings. Spread footings can be designed for a net, allowable bearing pressure of up to 3,000 psf, where these foundations are placed on at least 12 inches of structural fill, overlying firm, or medium dense existing subgrade soils. Where unsuitable or soft native soils have been encountered at the subgrade elevation, these soils should overexcavated, and replaced with properly placed and compacted structural fill. In areas where overexcavation occurs below footings, the overexcavation and structural fill should extend laterally a minimum distance that is equal to the depth of the excavation below the footing.

PSI recommends that column footings have a minimum width of 24 inches, even if those dimensions result in stresses below the allowable bearing capacity. The purpose of limiting the footing size is to prevent excessive shear deformation and to provide for vertical stability. Footings should be provided with at least 18 inches of embedment below the lowest adjacent exterior final grade.

Horizontal forces can be resisted partially or completely by frictional forces developed between the base of the spread footings and the underlying native soils. The total shearing resistance between the foundation footprint and the soil should be taken as the normal force (i.e., the sum of all vertical forces, dead load plus real live load, times the coefficient of friction between the soil and the base of the footing). PSI recommends utilizing an ultimate coefficient of friction value of 0.40 for design. If additional lateral resistance is required, passive earth pressures against embedded footings can be computed using a pressure based on an equivalent fluid with a unit weight of 250 pcf. This value is based on backfill around footings being cast against the native silty, clayey soils. Where the backfill around the footings is cast against properly compacted structural fill, a passive pressure of 450 pcf may be utilized. These values are considered ultimate values, and an appropriate factor of safety should be utilized in the design.

4.3.2 DRILLED PIER FOUNDATIONS

The proposed butane tank pedestals and other equipment such as equipment skids can be supported by cast-in-place reinforced concrete drilled piers. It is anticipated that groundwater (near the location of the proposed butane tank) will be near a depth of approximately 7 to 10 feet below the existing site grades at this site.

Due to the potential for relatively high groundwater and the likelihood of caving soils, PSI recommends that drilled piers be installed with a temporary steel casing to control caving within the overburden soils. The Contractor should provide PSI with a work plan prior to construction so that PSI may comment on the necessity of casing the drilled shafts, regardless of the elevation of groundwater or pier tip depth (i.e., casing will likely be required to successfully install the drilled piers). Concrete will likely need to be placed using tremie pipe methods after the steel reinforcing cage is in place, and before the temporary casing is removed, particularly beneath the groundwater level.

PSI should observe the excavation at the time of the installation of each of the proposed drilled pier foundations. Drilled piers that bear within the existing fill soils, PSI recommends that the Contractor perform load testing on a test pier near each tank pile cap. Recommendations for drilled pier load testing is further described below.

Table 6 contains PSI's recommended end bearing and skin friction capacities for the drilled piers. Please note that the upper 2 feet of skin friction must be neglected from pile capacity calculations. PSI has included a factor of safety of 2.0 in the recommended skin and end bearing calculations contained in Table 6.

Table 6: Recommended Drilled Pier Skin Friction and End-Bearing Parameters

Approximate Depth Range (feet)	Approximate Elevation Range (feet above MSL)	General Soil Type	Recommended Skin Friction Capacity (psf)	Recommended End-Bearing Capacity (psf)
CPT-01 (East Support of Butane Tank)				
0 to 2	35 to 33	Existing Fill / Asphalt	Neglect	N/A
2 to 6	33 to 29	Native Sand / Silty Sand	150	20,000
6 to 14	29 to 21	Native Silty Sand / Sandy Silt	150	10,000
14 to 22	21 to 13	Native Clay / Silty Clay	150	6,000
22 to 46	13 to -11	Native Sandy Silt / Silty Sand	300	20,000
Greater than 46	Less than -11	Native Gravel	2,500	60,000
CPT-02 (West Support of Butane Tank)				
0 to 2	34 to 32	Existing Fill / Asphalt	Neglect	N/A
2 to 8	32 to 26	Native Clay / Silty Clay	50	4,500
8 to 14	26 to 20	Native Clay / Silty Clay	100	9,000
14 to 22	20 to 12	Native Silty Sand / Sandy Silt	150	25,000
22 to 33½	12 to ½	Native Silty Sand / Sandy Silt	750	30,000
33½ to 43	½ to -9	Native Silty Sand / Sandy Silt	300	20,000
Greater Than 46	Less than -9	Native Gravel	1,000	60,000

Table 6: Recommended Drilled Pier Skin Friction and End-Bearing Parameters

Approximate Depth Range (feet)	Approximate Elevation Range (feet above MSL)	General Soil Type	Recommended Skin Friction Capacity (psf)	Recommended End-Bearing Capacity (psf)
CPT-03 (Equipment Pad Area)				
0 to 2	32 to 30	Existing Fill	Neglect	N/A
2 to 8	30 to 24	Native Sand / Silty Sand	75	20,000
8 to 10	24 to 22	Native Clay	25	2,000
10 to 16	22 to 6	Native Sand / Silty Sand	200	30,000
16 to 32	6 to 0	Native Clay / Silty Clay	400	15,000
32 to 42	0 to -10	Native Silty Sand / Sandy Silt	500	40,000
Greater than 42	Less than -10	Native Gravel	1,000	60,000

RECOMMENDED LPILE SOIL PARAMETERS

PSI prepared the following recommended LPILE soil parameters based on our interpretation of the subsurface conditions encountered at the sounding locations, and geotechnical reports near this project site. The following LPILE soil parameters may be used in the design of laterally loaded piles for this project. These recommended LPILE soil parameters are to be utilized beginning at the proposed bottom of abutment pile cap and proceeding downward. The values in Table 7 were estimated from empirical relationships presented in FHWA-91-048 (Reese & Wang) and are suitable for use in analysis by COM624P and LPILE:

Table 7: Recommended LPILE Soil Parameters

Approximate Depth Range (feet)	General Soil Type	Average Effective Undrained Shear Strength, c (psf)	Estimated Effective Angle of Internal Friction (degrees)	Estimated Total/ Submerged Unit Weight (pcf)	Recommended K_s Value (pci)*
CPT-01 (East Support of Butane Tank)					
0 to 2	Existing Fill / Asphalt	0	28	110 / 48	Neglect
2 to 6	Native Sand / Silty Sand	0	34	115 / 53	50 (Submerged: 35)
6 to 14	Native Silty Sand / Sandy Silt	0	32	110 / 48	25 (Submerged: 20)
14 to 22	Native Clay / Silty Clay	1,000	0	100 / 38	240 (E_{50} = 0.008)
22 to 46	Native Sandy Silt / Silty Sand	0	34	120 / 58	80 (Submerged: 55)
Greater than 46	Native Gravel	0	42	130 / 68	500 (Submerged: 180)
CPT-02 (West Support of Butane Tank)					
0 to 2	Existing Fill / Asphalt	0	28	110 / 48	Neglect
2 to 8	Native Clay / Silty Clay	550	0	105 / 43	40 (E_{50} = 0.014)
8 to 14	Native Clay / Silty Clay	1,000	0	115 / 53	240 (E_{50} = 0.008)
14 to 22	Native Silty Sand / Sandy Silt	0	32	120 / 58	65 (Submerged: 45)
22 to 33½	Native Silty Sand / Sandy Silt	0	34	125 / 63	95 (Submerged: 65)
33½ to 43	Native Silty Sand / Sandy Silt	0	36	125 / 63	125 (Submerged: 80)
Greater Than 46	Native Gravel	0	42	130 / 68	500 (Submerged: 180)

Table 7: Recommended LPILE Soil Parameters (Continued)

Approximate Depth Range (feet)	General Soil Type	Average Effective Undrained Shear Strength, c (psf)	Estimated Effective Angle of Internal Friction (degrees)	Estimated Total/ Submerged Unit Weight (pcf)	Recommended K_s Value (pci)*
CPT-03 (Equipment Pad Area)					
0 to 2	Existing Fill	0	28	110 / 48	Neglect
2 to 8	Native Sand / Silty Sand	0	30	115 / 53	45 (Submerged: 30)
8 to 10	Native Clay	400	0	95 / 33	35 (E_{50} = 0.019)
10 to 16	Native Sand / Silty Sand	0	34	120 / 58	80 (Submerged: 55)
16 to 32	Native Clay / Silty Clay	2,000	0	120 / 58	680 (E_{50} = 0.006)
32 to 42	Native Silty Sand / Sandy Silt	0	36	125 / 63	165 (Submerged: 100)
Greater than 42	Native Gravel	0	42	130 / 68	500 (Submerged: 180)

Groundwater Elevation anticipated at 7½ feet bgs.

The design engineer should neglect the upper 2 feet of soil at each pier location from the lateral load calculations.

PILE GROUP EFFICIENCY RECOMMENDATIONS

Pile groups subjected to vertical loads do not have the same capacity as the sum of the capacity of the individual piles. The amount of reduction in the pile capacity or the group efficiency factor will depend on the pile spacing or the distance between the adjacent piles. For axially loaded piles designed in a group and bearing in cohesionless soils, the efficiency factor may be taken as 0.67, when the center-to-center distance between the piles is equal to 2 pile diameters.

PILE LOAD TESTS

Prior to construction of the production piles, and where the drilled shafts are planned to bear within the existing fill soils, PSI recommends installing 2 test piers near each end of the proposed tank pile caps. Load bearing properties of the piles should be evaluated by performing a load test in general accordance with the, "Standard Method of Testing Piles under Axial Compressive Load," (ASTM D-1143). We recommend that a minimum of two load tests be performed under the direct supervision of PSI to verify the estimated drilled pier capacity.

4.4 SETTLEMENT

The foundation loads, and live loads will cause settlement due to consolidation, or compression, of the underlying soils. PSI anticipates that the maximum saddle loads will be up to 480 kips. Based on the anticipated loads and PSI's recommended net allowable bearing capacity, PSI anticipates the proposed spread footings will bear at a depth of approximately 30 inches below the existing site grades. Should drilled piers be utilized for the support of the proposed tank, PSI anticipates these foundation elements will bear on the order of 10 to 14 feet bgs.

PSI estimates that the static settlement (non-seismic) of the shallow foundations of the proposed butane tank that is designed and constructed in accordance with the recommendations in this report will be on the order of 2 inches, assuming the foundations will consist of square spread footings. This estimated settlement is based on the load on the footing is sustained (actual) dead load or long-term live load. Lesser actual bearing pressures should produce less settlement. Should drilled piers be utilized for the support of the proposed tank, the estimated static settlement of these deep foundations will be on the order of ½ inch or less (assuming at least a three pier by three pier pile cap is designed).

Some differential settlement between footings should be expected due to differences in their size and loading conditions and the variability in subsurface conditions across the loaded footprint. Differential settlements are difficult to quantify; however, PSI anticipates the differential settlements will likely be limited to less than about one half the total settlement or approximately 1 inch between the proposed butane tank saddle foundations with spread footing and ¼ inch between saddle foundations founded on piers. Settlement of the footings is also expected to occur rapidly, essentially as the new structural loads are placed and shortly thereafter. Should drilled piers be utilized for the support of the proposed tank, the estimated differential settlement will be approximately half of the total settlement.

Where structures are planned that cannot tolerate 2 inches of static settlement and are desired to be founded on square spread footings, PSI recommends proportioning these spread footings such that the bearing capacity of the foundations of these structures are 2,000 psf. PSI estimates the settlement due to this induced bearing load will be on the order of 1 inch, with differential settlements of less than ½ inch between saddle footings.

Where structures are settlement sensitive and cannot tolerate 1 inch of settlement, PSI recommends these structures be supported by drilled piers. The settlement of the drilled piers are a function of the depth of these foundation elements. After the drilled piers are designed, PSI should be contacted for further settlement analysis and consulting.

Please note that settlements discussed in this section are static settlements and to not include seismic liquefaction induced settlements. To obtain total settlement potential for the site static and liquefaction induces settlements should be added together.

4.5 DRAINAGE

Pavement surfaces and open space areas should be sloped such that surface water runoff is collected and routed to suitable discharge points. PSI also recommends that ground surfaces adjacent to the proposed improvements be sloped to facilitate positive drainage away from these structures and the related foundations.

4.6 DESIGN REVIEW AND CONSTRUCTION MONITORING

After plans and specifications are complete, PSI should review the final design and specifications so that the earthwork and foundation recommendations are properly interpreted and implemented. It is considered imperative that the Geotechnical Engineer and/or their representative be present during earthwork operations and foundation installations to observe the field conditions with respect to the design assumptions and specifications. PSI will not be responsible for changes in the project design or project information it was not provided, or interpretations and field quality control observations made by others. PSI would be pleased to provide these services for this project.

5 GEOTECHNICAL RISK AND REPORT LIMITATIONS

The concept of risk is an important aspect of the geotechnical evaluation. The primary reason for this is that the analytical methods used to develop geotechnical recommendations do not comprise an exact science. The analytical tools which geotechnical engineers use are generally empirical and must be used in conjunction with engineering judgment and experience. Therefore, the solutions and recommendations presented in the geotechnical evaluation should not be considered risk-free and, more importantly, are not a guarantee that the interaction between the soils and the proposed structure will perform as planned. The engineering recommendations presented in the preceding sections constitute PSI's professional estimate of those measures that are necessary for the proposed structure to perform according to the proposed design based on the information generated and referenced during this evaluation, and PSI's experience in working with these conditions.

The recommendations submitted for the proposed 90,000-gallon butane storage tank are based on the information provided to PSI. If there are any revisions to the plans for this project or if deviations from the subsurface conditions noted in this report are encountered during construction, PSI must be notified immediately to determine if changes to PSI's recommendations are required. If PSI is not retained to perform these functions, PSI cannot be responsible for the impact of those conditions on the performance of the project.

The stratification shown on the CPT logs represent the conditions only at the actual sounding locations. Variations may occur and should be expected between sounding locations. The stratification represents the approximate boundary between subsurface materials; however, the actual transition may be gradual, abrupt, or not clearly defined. In the absence of foreign substances, it is difficult to distinguish between native soils and clean fill soil.

The Geotechnical Engineer warrants that the findings, recommendations, specifications, or professional advice contained herein have been made in accordance with generally accepted professional geotechnical engineering practices in the local area. No other warranties are implied or expressed.

After the plans and specifications are complete, PSI should be retained to review the final design plans and specifications. This review is required to verify that the engineering recommendations are appropriate for the final configuration, and that they have been properly incorporated into the design documents. This report has been prepared for the exclusive use of Sunoco Logistics Partners, LP for specific application to the proposed 90,000-gallon butane storage tank project at the existing Philips 66 facility at the Portland Terminal in Portland, Oregon.

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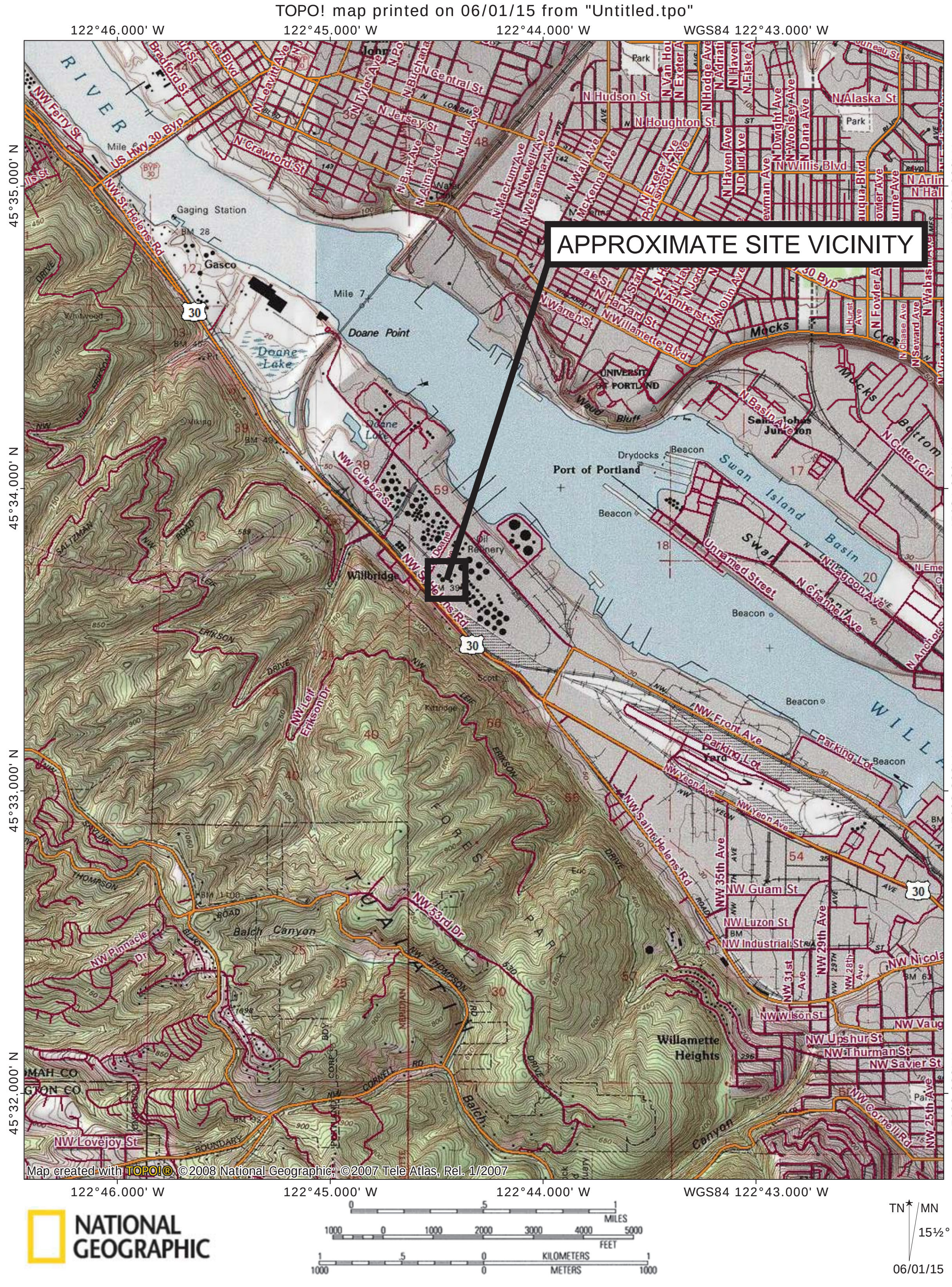
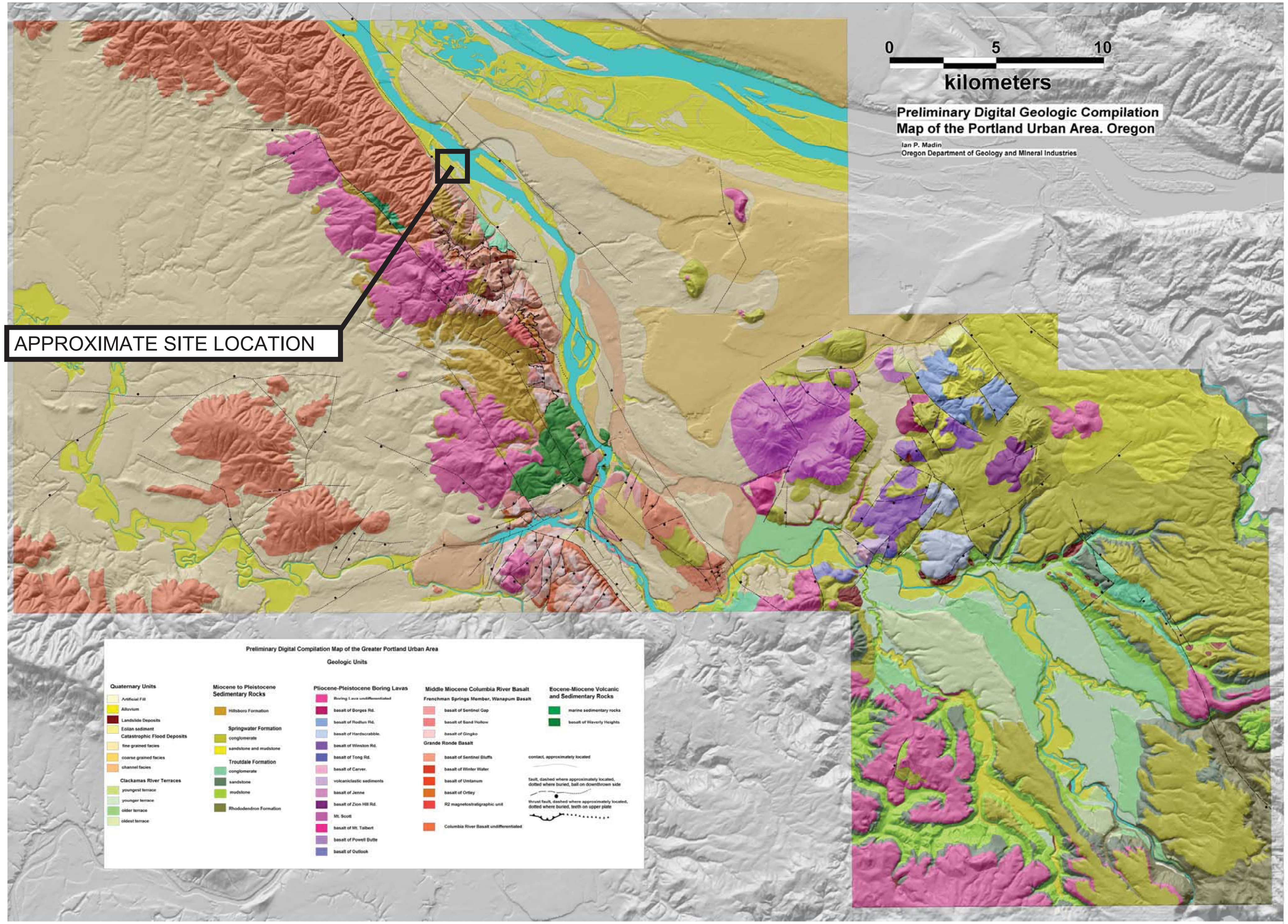
FIGURES

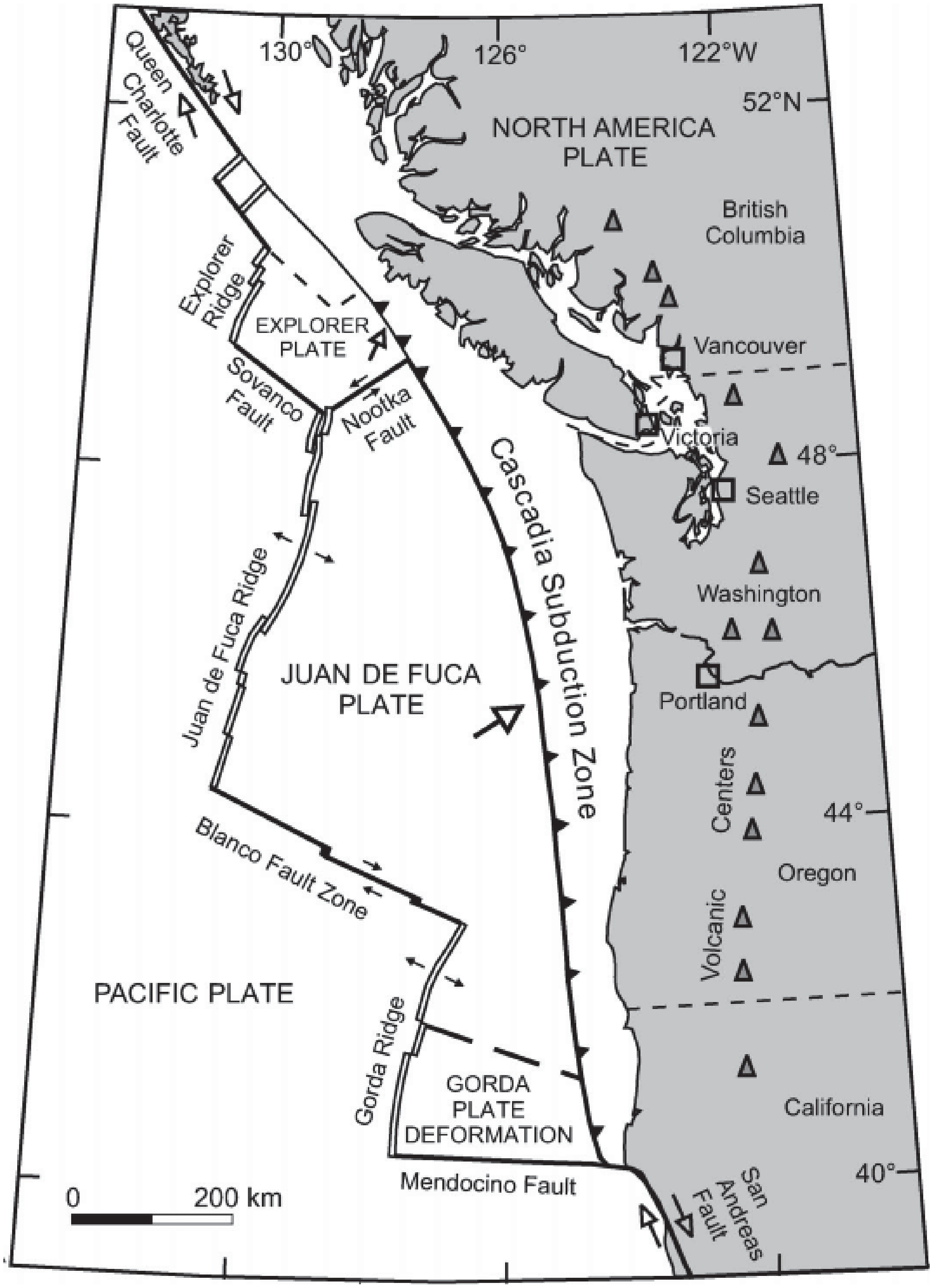


psi Information To Build On Engineering • Consulting • Testing	DATE: OCTOBER 2016	PROPOSED 90,000-GALLON BUTANE TANK PHILLIPS 66 - PORTLAND TERMINAL 5528 NW DOANE AVENUE PORTLAND, OREGON	PSI PROJECT NUMBER 0704999
PSI, INC. **6032 N. CUTTER CIRCLE, SUITE 480** **PORTLAND, OREGON 97217** **(503) 289-1778**	DRAWN BY: ESR	**SITE VICINITY MAP**	**FIGURE 1**

psi Information To Build On Engineering • Consulting • Testing	DATE: OCTOBER 2016	PROPOSED 90,000-GALLON BUTANE TANK PHILLIPS 66 - PORTLAND TERMINAL 5528 NW DOANE AVENUE PORTLAND, OREGON	PSI PROJECT NUMBER 0704999
PSI, INC. **6032 N. CUTTER CIRCLE, SUITE 480** **PORTLAND, OREGON 97217** **(503) 289-1778**	DRAWN BY: JDB	**SITE EXPLORATION MAP**	**FIGURE 2**

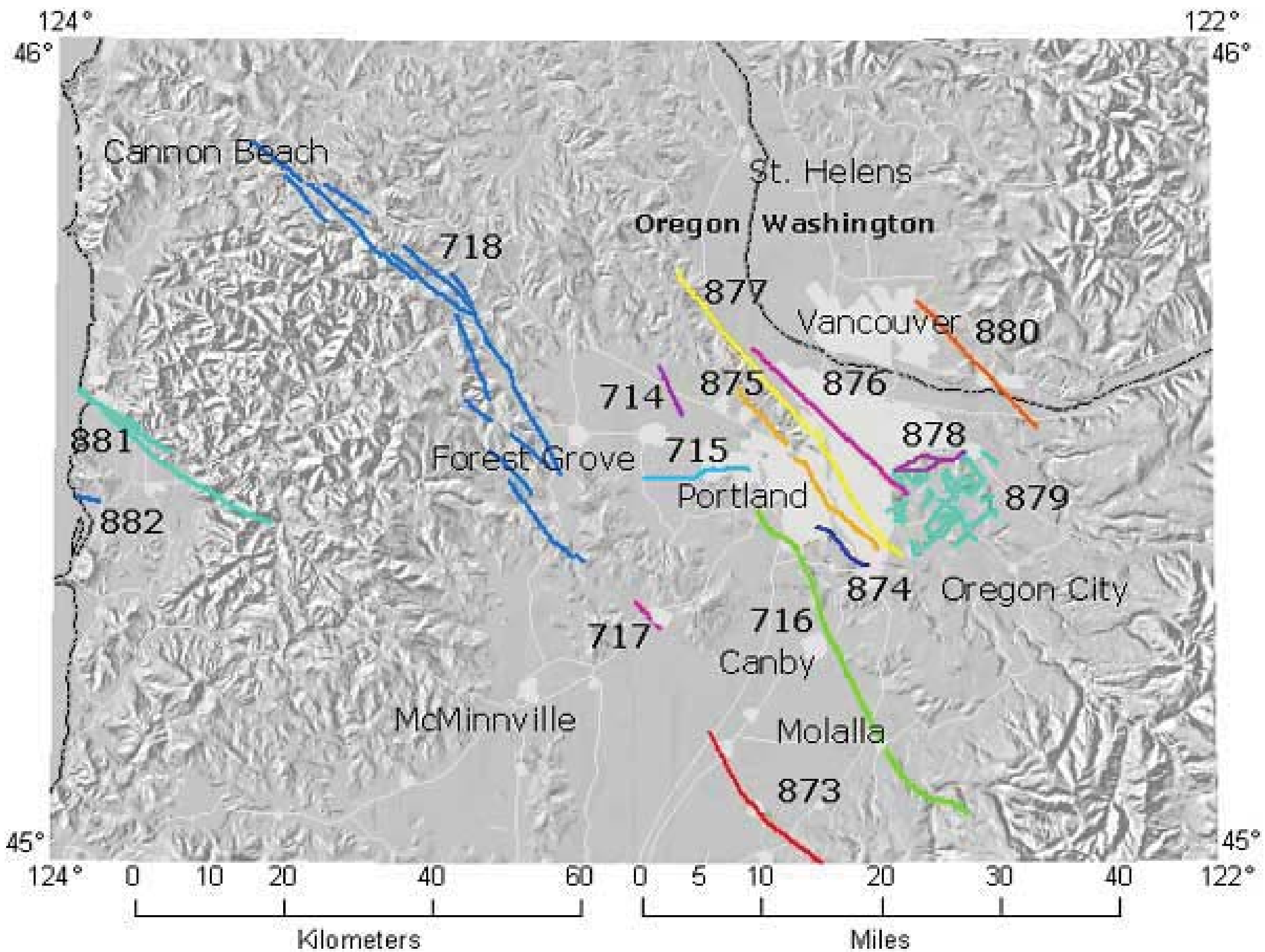


psi Information To Build On *Engineering • Consulting • Testing*	DATE: OCTOBER 2016	PROPOSED 90,000-GALLON BUTANE TANK PHILLIPS 66 - PORTLAND TERMINAL 5528 NW DOANE AVENUE PORTLAND, OREGON	PSI PROJECT NUMBER 0704999
PSI, INC. 6032 N. CUTTER CIRCLE, SUITE 480 PORTLAND, OREGON 97217 (503) 289-1778	DRAWN BY: ESR	**GEOLOGIC MAP OF PORTLAND**	**FIGURE 3**



TECTONIC MAP OF PACIFIC NORTHWEST, SHOWING ORIENTATION AND EXTENT OF CASCADIA SUBDUCTION ZONE
(MODIFIED FROM DRAGERT AND OTHERS, 1994)

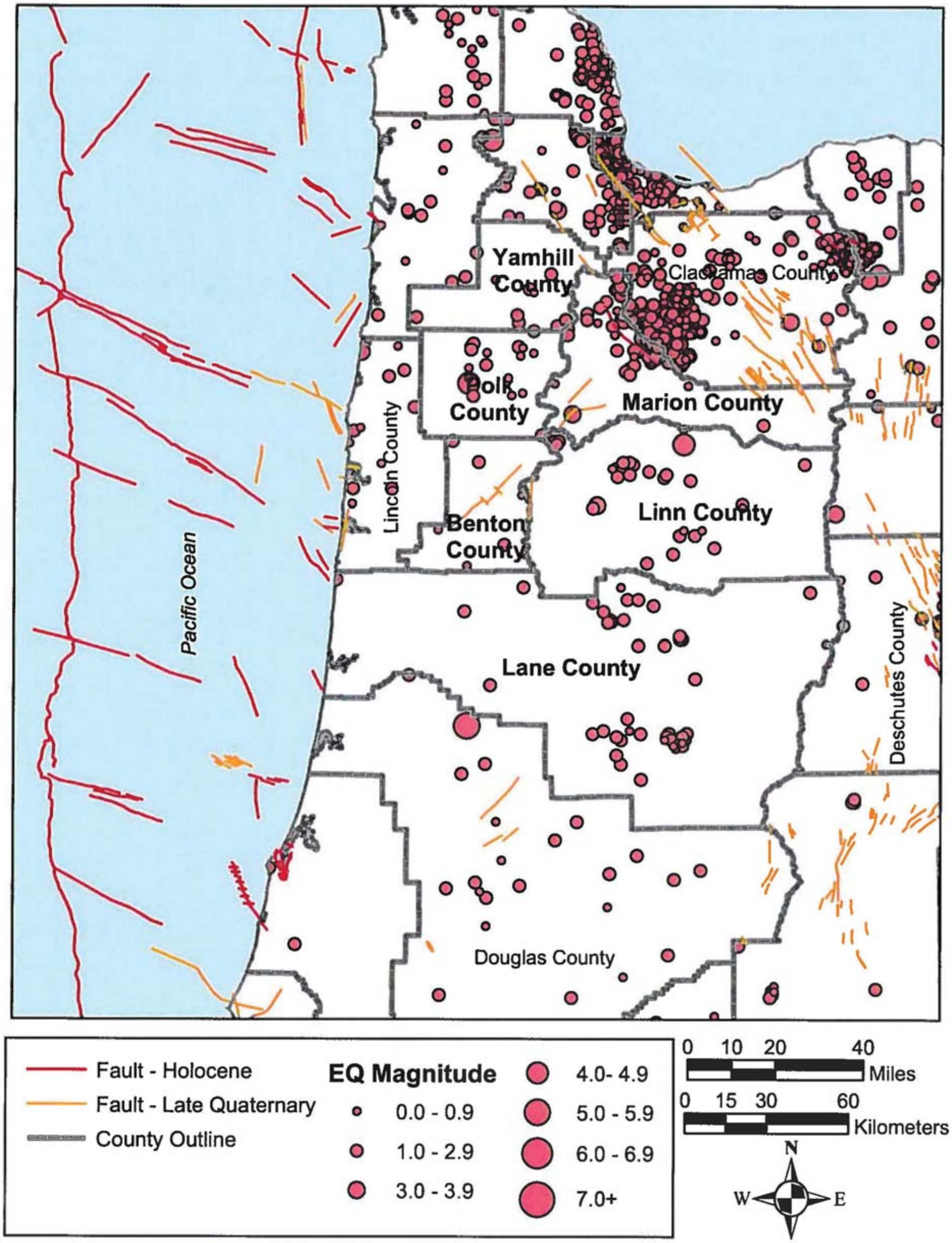
psi Information To Build On Engineering • Consulting • Testing	DATE: OCTOBER 2016	PROPOSED 90,000-GALLON BUTANE TANK PHILLIPS 66 - PORTLAND TERMINAL 5528 NW DOANE AVENUE PORTLAND, OREGON	PSI PROJECT NUMBER 0704999
PSI, INC. 6032 N. CUTTER CIRCLE, SUITE 480 PORTLAND, OREGON 97217 (503) 289-1778	DRAWN BY: ESR	**TECTONIC MAP OF THE PACIFIC NORTHWEST**	**FIGURE 4**



Number	Name
714	Helvetia fault
715	Beaverton fault zone
716	Canby-Molalla fault
717	Newberg fault
718	Gales Creek fault zone
873	Mount Angel fault
874	Bolton fault
875	Oatfield fault
876	East Bank fault
877	Portland Hills fault
878	Grand Butte fault
879	Damascus-Tickle Creek fault zone
880	Lacamas Lake fault
881	Tillamook Bay fault zone
882	Happy Camp fault

Last modified January 27, 2006
URL http://earthquake.usgs.gov/regional/qfaults/or/van.html

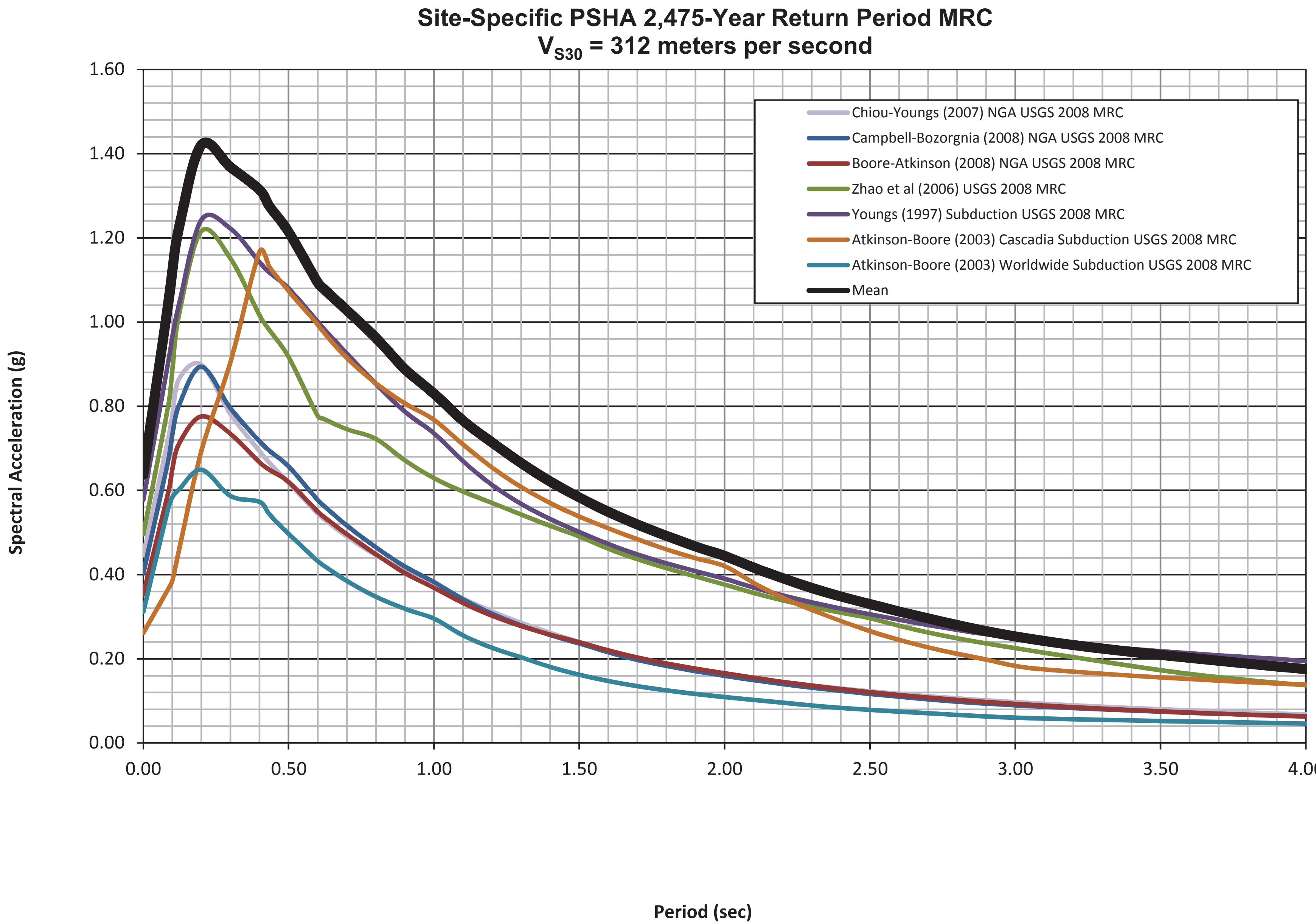
psi Information To Build On Engineering • Consulting • Testing	DATE: OCTOBER 2016	PROPOSED 90,000-GALLON BUTANE TANK PHILLIPS 66 - PORTLAND TERMINAL 5528 NW DOANE AVENUE PORTLAND, OREGON	PSI PROJECT NUMBER: 0704999
PSI, INC. 6032 N. CUTTER CIRCLE, SUITE 480 PORTLAND, OREGON 97217 (503) 289-1778	DRAWN BY: ESR	**USGS VANCOUVER - PORTLAND FAULT MAP**	**FIGURE 5**



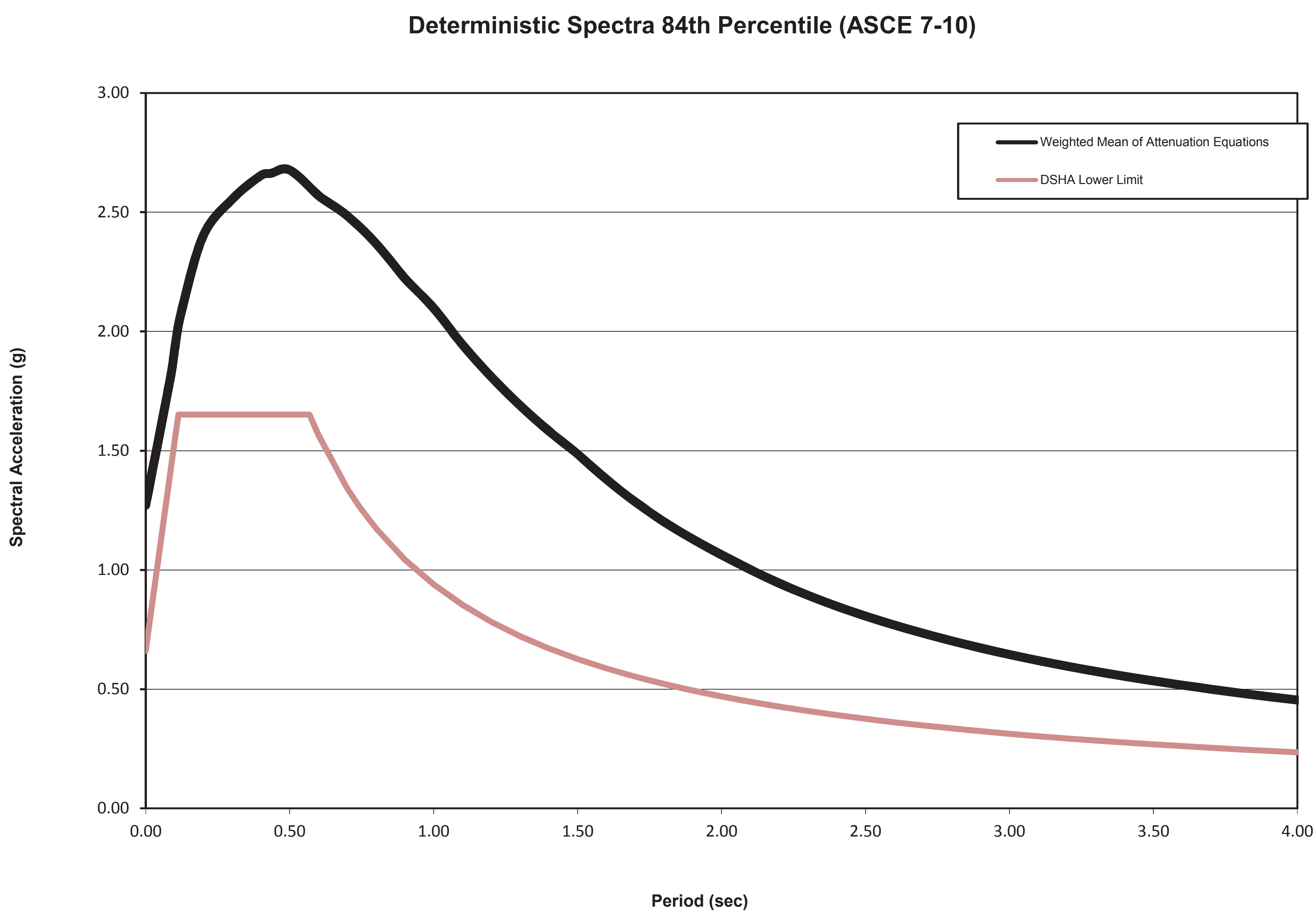
NOTE: Active faults on this map are defined as those that have moved in the last 780,000 years (Geomatrix Consultants, Inc., 1995). Faults active in the last 20,000 years are colored red. Faults active between 20, 000 and 780,000 years ago are colored gold.

SOURCE: Oregon Department of Geology and Mineral Industries IMS-24

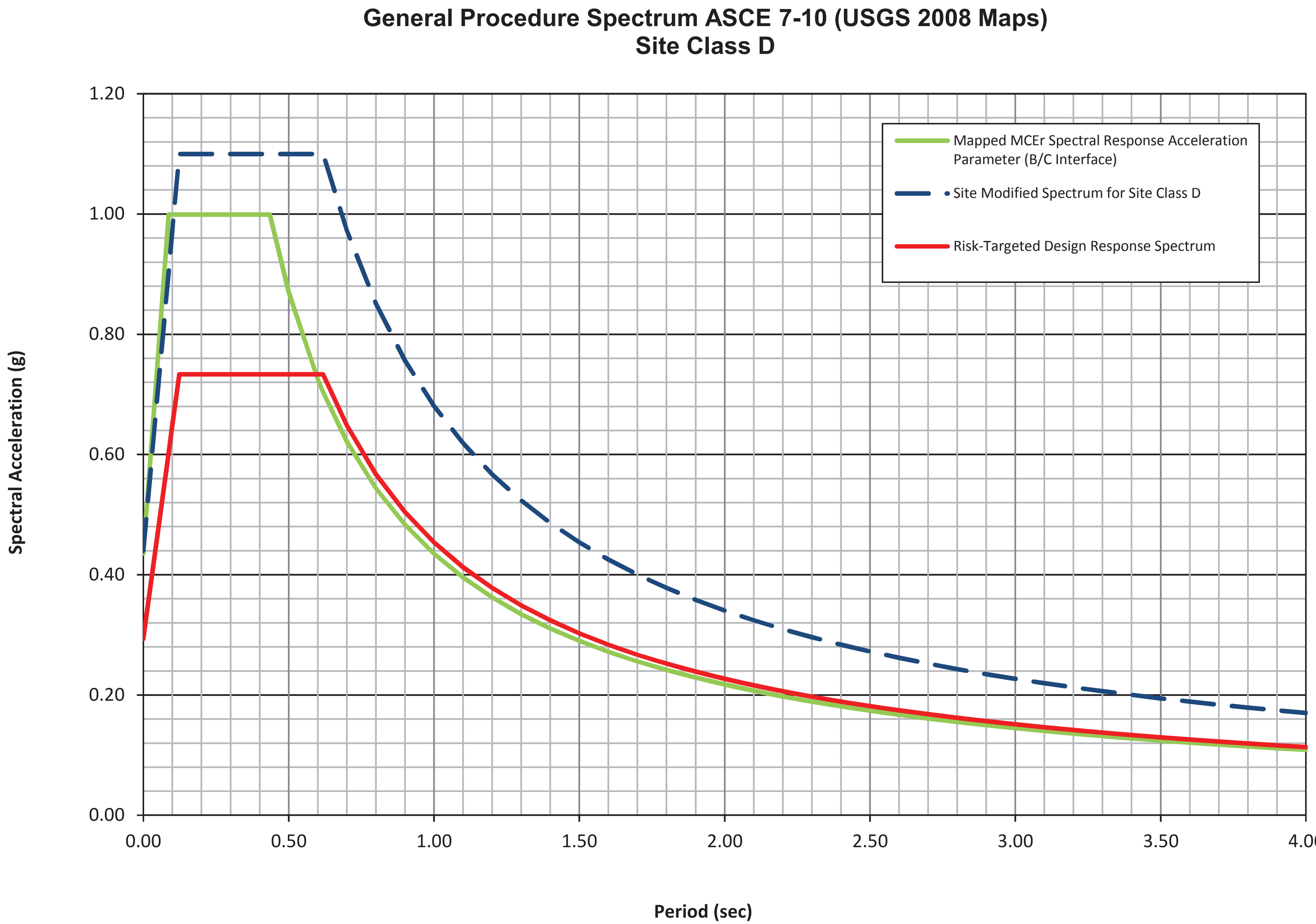
psi Information To Build On Engineering • Consulting • Testing	DATE: OCTOBER 2016	PROPOSED 90,000-GALLON BUTANE TANK PHILLIPS 66 - PORTLAND TERMINAL 5528 NW DOANE AVENUE PORTLAND, OREGON	PSI PROJECT NUMBER 0704999
PSI, INC. **6032 N. CUTTER CIRCLE, SUITE 480** **PORTLAND, OREGON 97217** **(503) 289-1778**	DRAWN BY: ESR	**HISTORIC SEISMICITY**	**FIGURE 6**



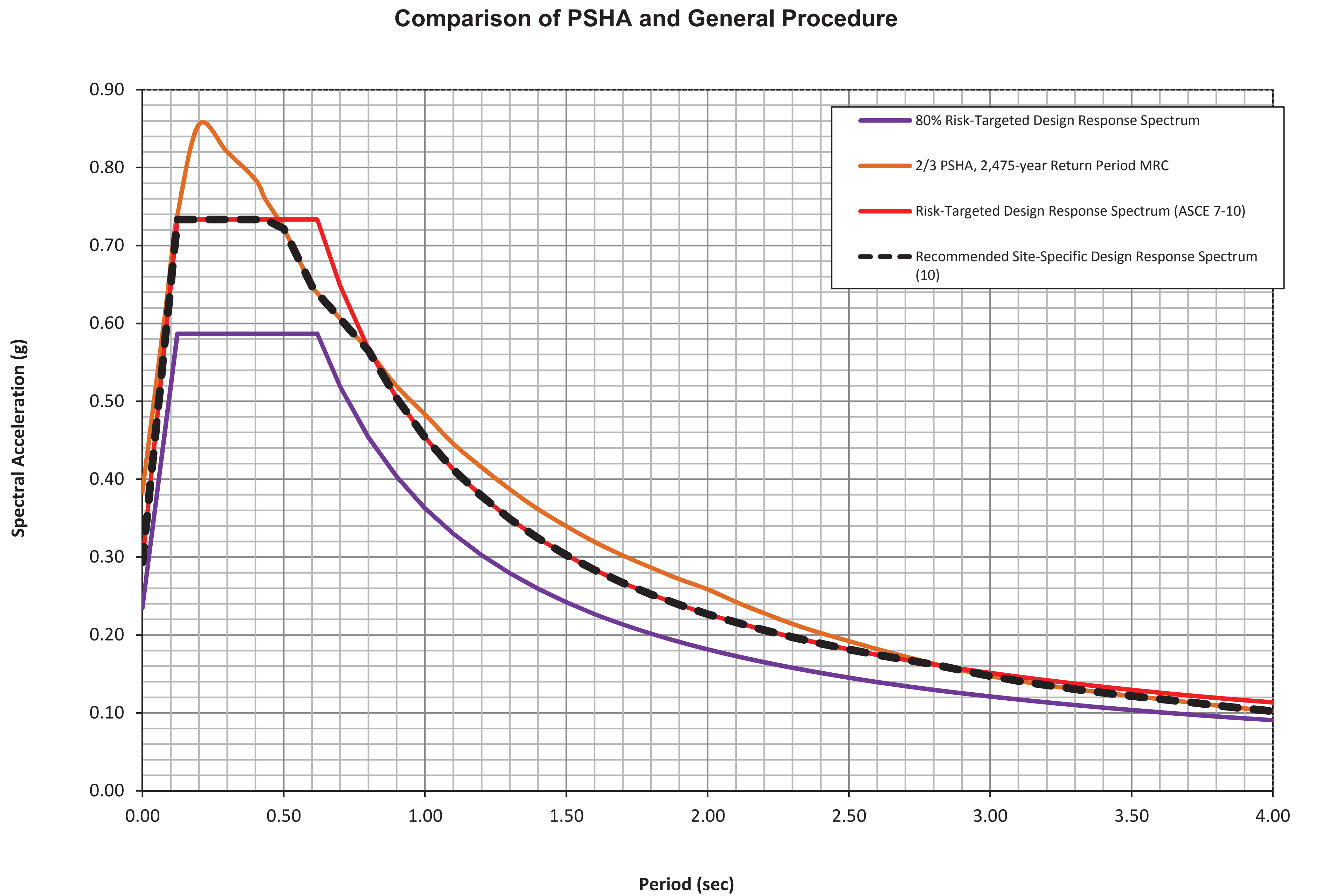
psi Information To Build On Engineering • Consulting • Testing	DATE: OCTOBER 2016	PROPOSED 90,000-GALLON BUTANE TANK PHILLIPS 66 - PORTLAND TERMINAL 5528 NW DOANE AVENUE PORTLAND, OREGON	PSI PROJECT NUMBER 0704999
PSI, INC. 6032 N. CUTTER CIRCLE, SUITE 480 PORTLAND, OREGON 97217 (503) 289-1778	DRAWN BY: ESR	**SITE-SPECIFIC PSHA 2,475-Year Return Period**	**FIGURE 7**



psi Information To Build On Engineering • Consulting • Testing	DATE: OCTOBER 2016	PROPOSED 90,000-GALLON BUTANE TANK PHILLIPS 66 - PORTLAND TERMINAL 5528 NW DOANE AVENUE PORTLAND, OREGON	PSI PROJECT NUMBER 0704999
PSI, INC. 6032 N. CUTTER CIRCLE, SUITE 480 PORTLAND, OREGON 97217 (503) 289-1778	DRAWN BY: ESR	**SITE-SPECIFIC DSHA**	**FIGURE 8**



psi Information To Build On Engineering • Consulting • Testing	DATE: OCTOBER 2016	PROPOSED 90,000-GALLON BUTANE TANK PHILLIPS 66 - PORTLAND TERMINAL 5528 NW DOANE AVENUE PORTLAND, OREGON	PSI PROJECT NUMBER 0704999
PSI, INC. 6032 N. CUTTER CIRCLE, SUITE 480 PORTLAND, OREGON 97217 (503) 289-1778	DRAWN BY: ESR	**GENERAL PROCEDURE**	**FIGURE 9**



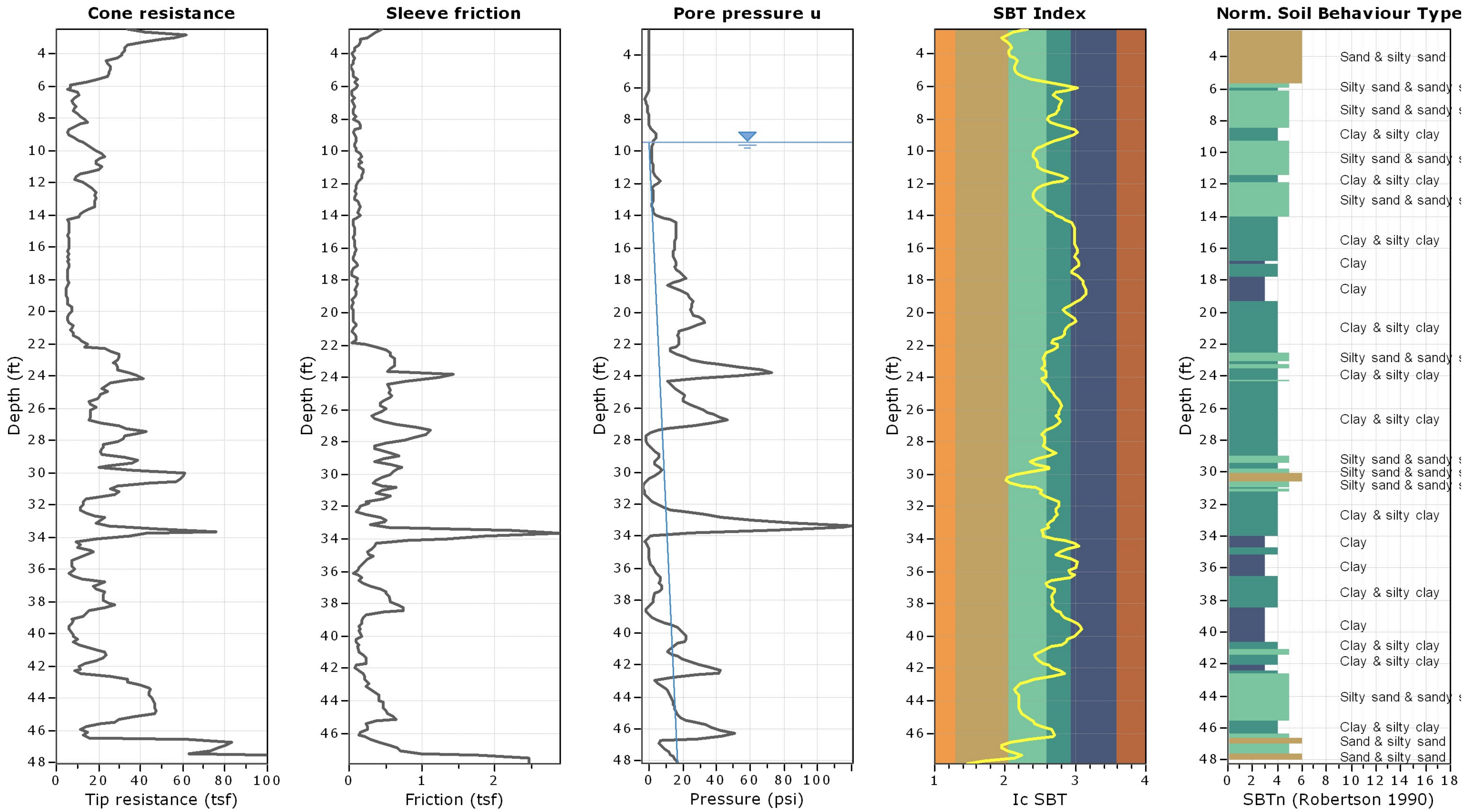
psi Information To Build On Engineering • Consulting • Testing	DATE: OCTOBER 2016	PROPOSED BUTANE STORAGE TANK CHEVRON USA 5924 NW FRONT AVENUE PORTLAND, OREGON	PSI PROJECT NUMBER 0704999
PSI, INC. 6032 N. CUTTER CIRCLE, SUITE 480 PORTLAND, OREGON 97217 (503) 289-1778	DRAWN BY: ESR	**RECOMMENDED SITE-SPECIFIC DESIGN RESPONSE SPECTRUM**	**FIGURE 10**

APPENDIX A – CPT LOGS

Professional Service Industries, Inc. (PSI)
6032 North Cutter Circle, Suite 480
Portland, Oregon 97217
(503) 289-1778

CPT: CPT-01
Total depth: 48.06 ft, Date: 10/27/2016
Surface Elevation: 184.00 ft
Coords: X:0.00, Y:0.00
Cone Type: Uknown
Cone Operator: Uknown

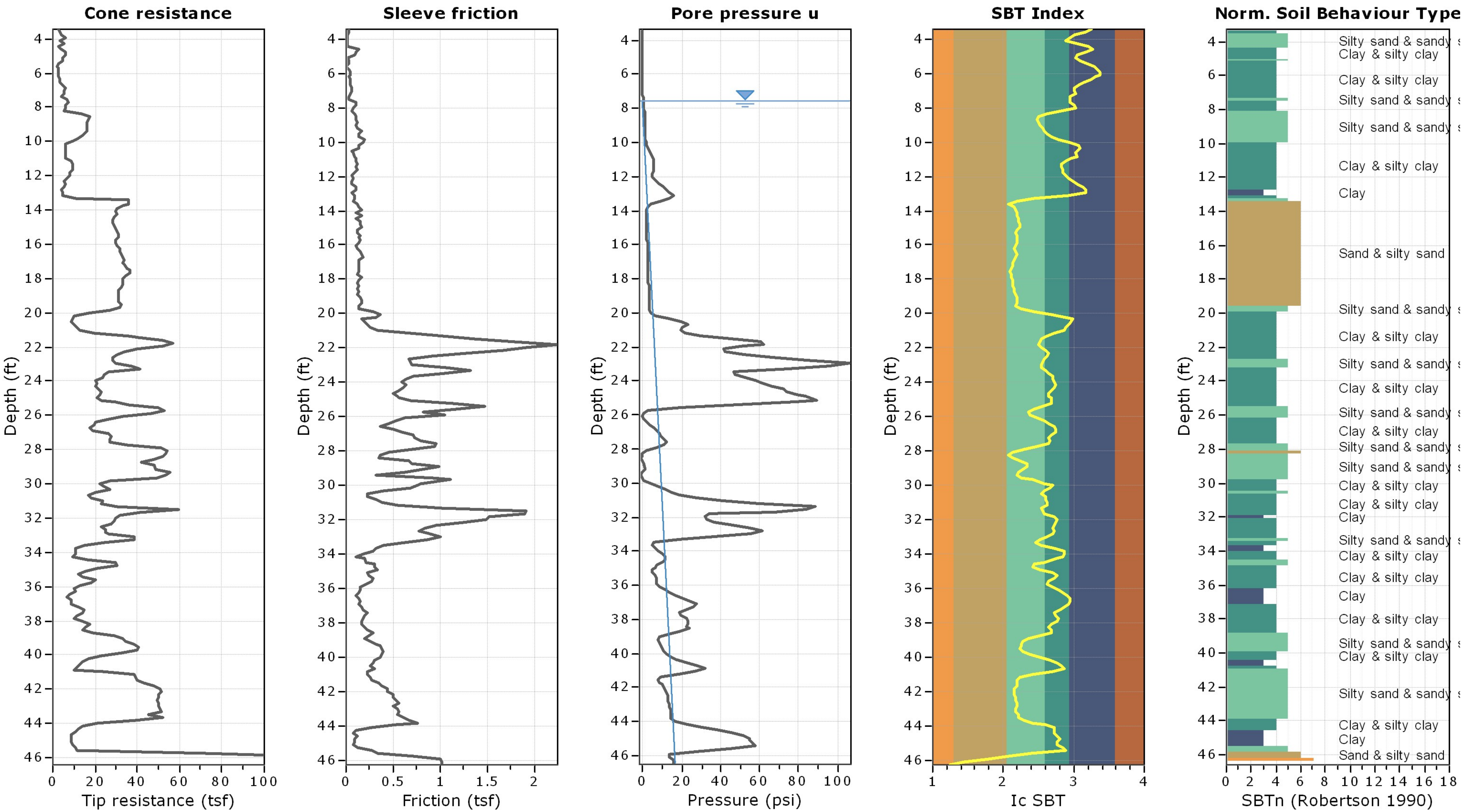
Project: P66 - Proposed Butane Tank
Location: Portland, Oregon



Professional Service Industries, Inc. (PSI)
6032 North Cutter Circle, Suite 480
Portland, Oregon 97217
(503) 289-1778

CPT: CPT-02
Total depth: 46.42 ft, Date: 10/27/2016
Surface Elevation: 184.00 ft
Coords: X:0.00, Y:0.00
Cone Type: Uknown
Cone Operator: Uknown

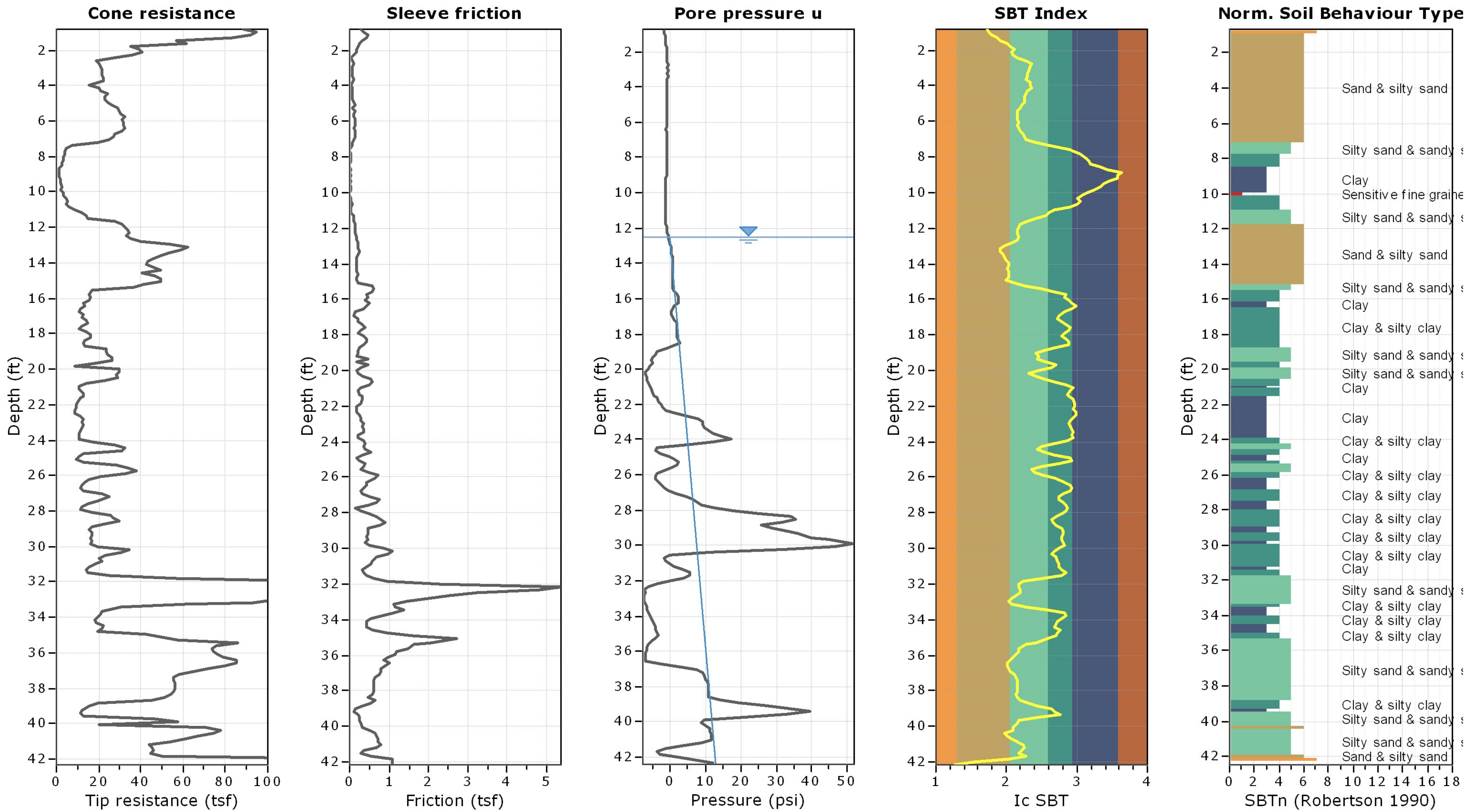
Project: **P66 - Proposed Butane Tank**
Location: **Portland, Oregon**



Professional Service Industries, Inc. (PSI)
6032 North Cutter Circle, Suite 480
Portland, Oregon 97217
(503) 289-1778

CPT: CPT-03
Total depth: 42.32 ft, Date: 10/27/2016
Surface Elevation: 184.00 ft
Coords: X:0.00, Y:0.00
Cone Type: Uknown
Cone Operator: Uknown

Project: **P66 - Proposed Butane Tank**
Location: **Portland, Oregon**



APPENDIX B – LIQUEFACTION ANALYSIS RESULTS

Professional Service Industries, Inc.
6032 N. Cutter Circle
Suite 480
Portland, Oregon 97217

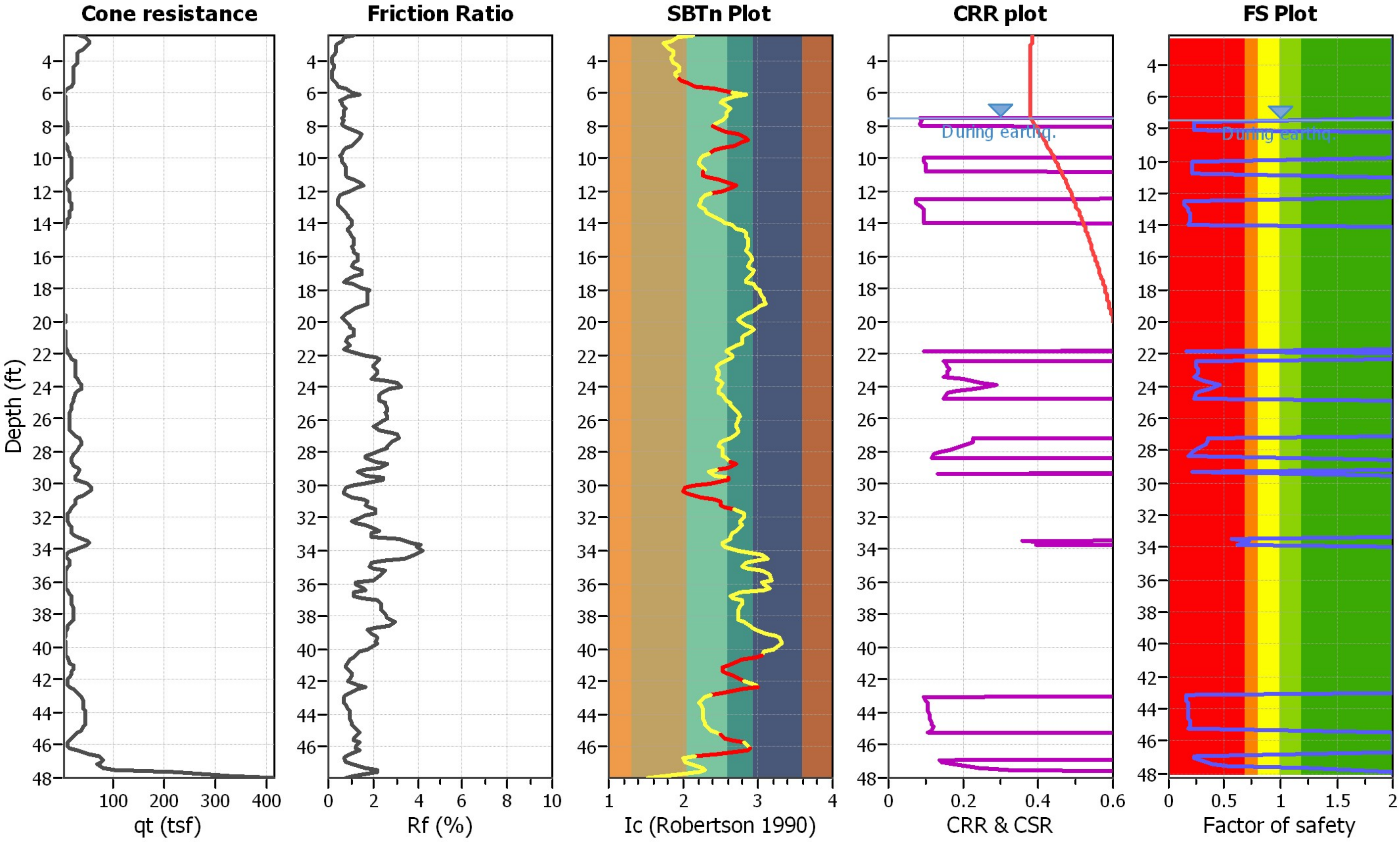
LIQUEFACTION ANALYSIS REPORT

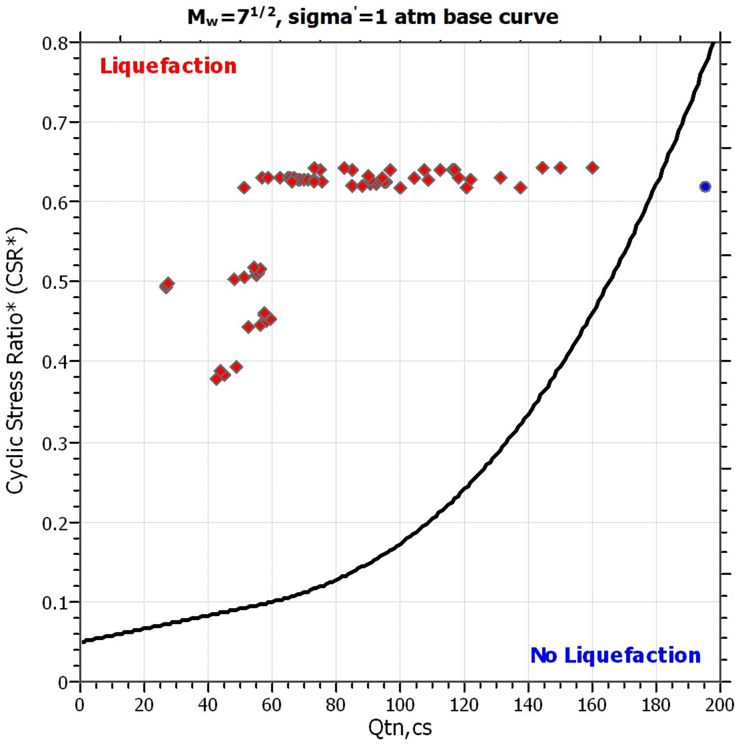
Project title : Philips 66 Portland Terminal **Location : Portland, OR**

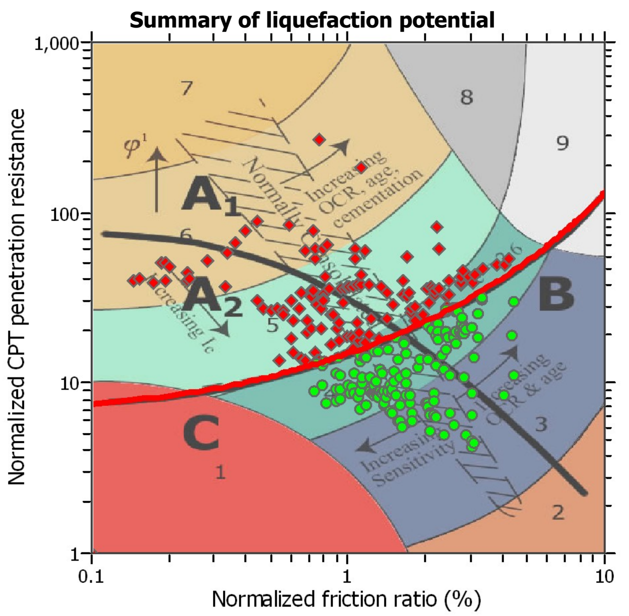
CPT file : CPT-01

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	9.40 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	7.50 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.55	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.58	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

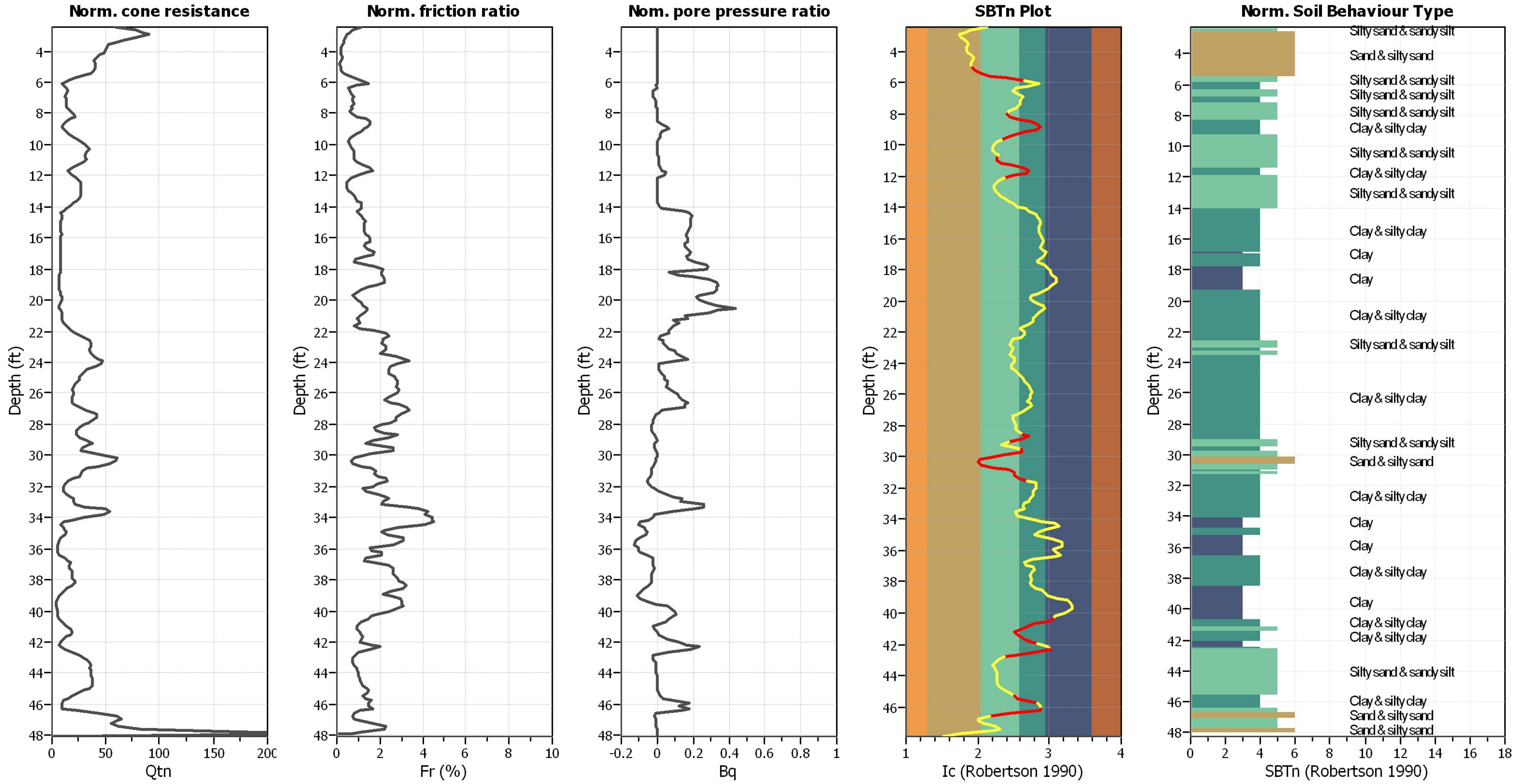






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots (normalized)



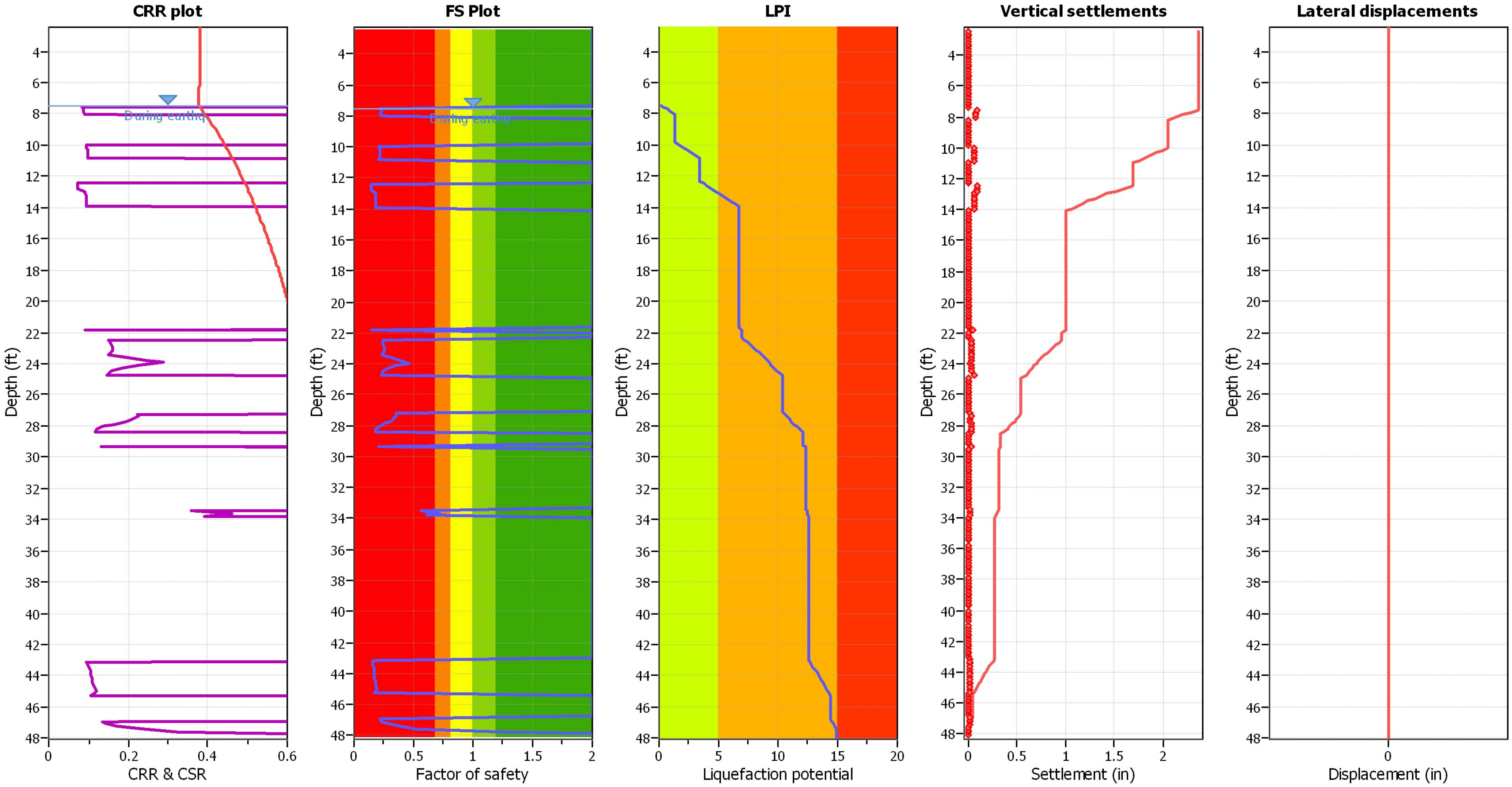
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	7.50 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.55	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.58	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	9.40 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	7.50 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.55	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.58	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	9.40 ft	Fill height:	N/A	Limit depth:	N/A

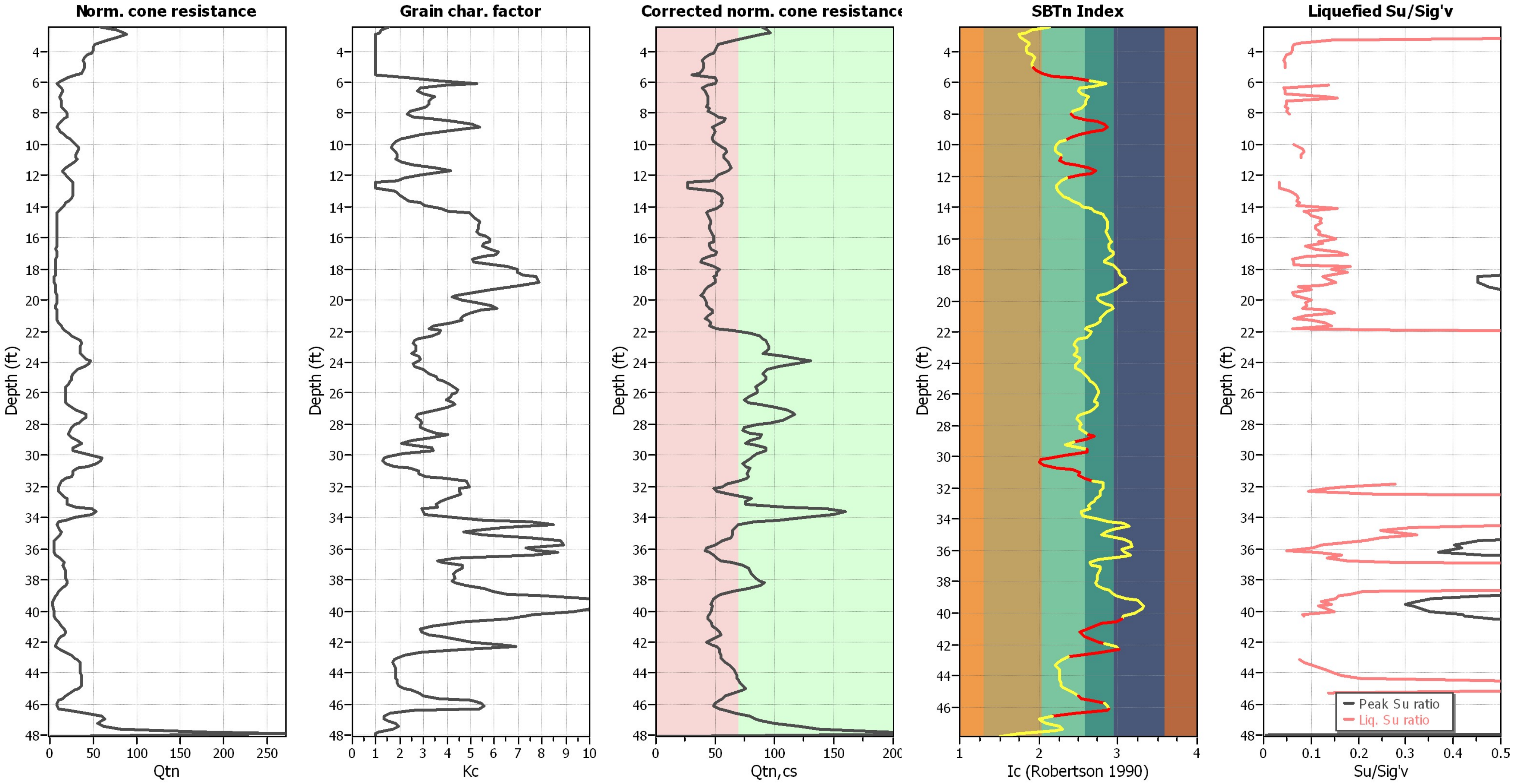
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	7.50 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.55	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.58	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	9.40 ft	Fill height:	N/A	Limit depth:	N/A

Professional Service Industries, Inc.
6032 N. Cutter Circle
Suite 480
Portland, Oregon 97217

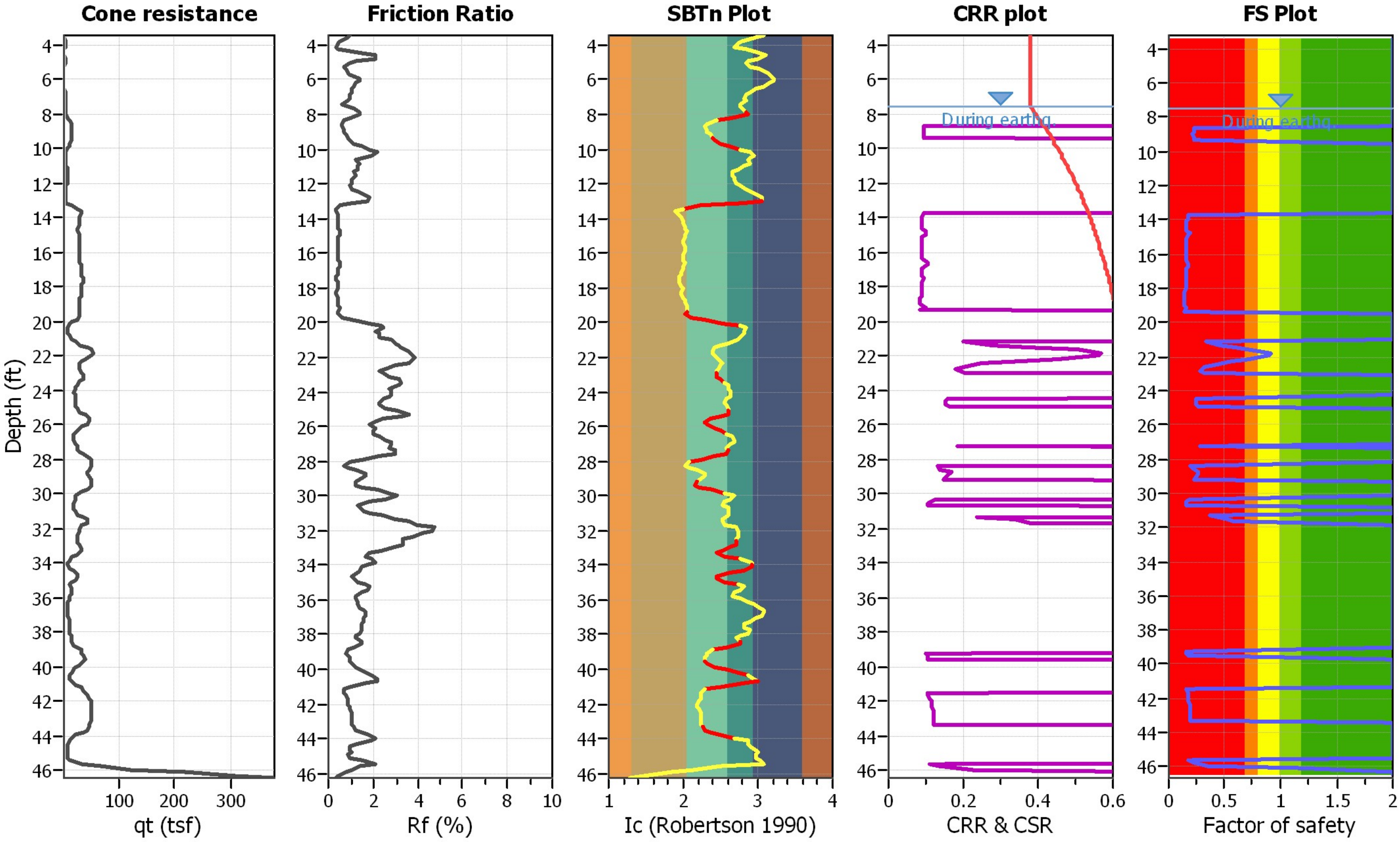
LIQUEFACTION ANALYSIS REPORT

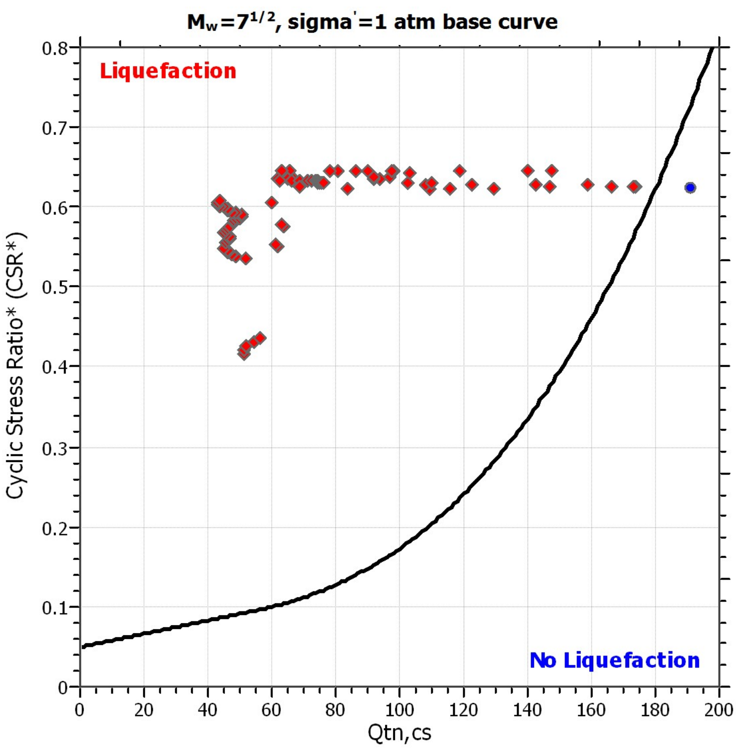
Project title : Philips 66 Portland Terminal **Location : Portland, OR**

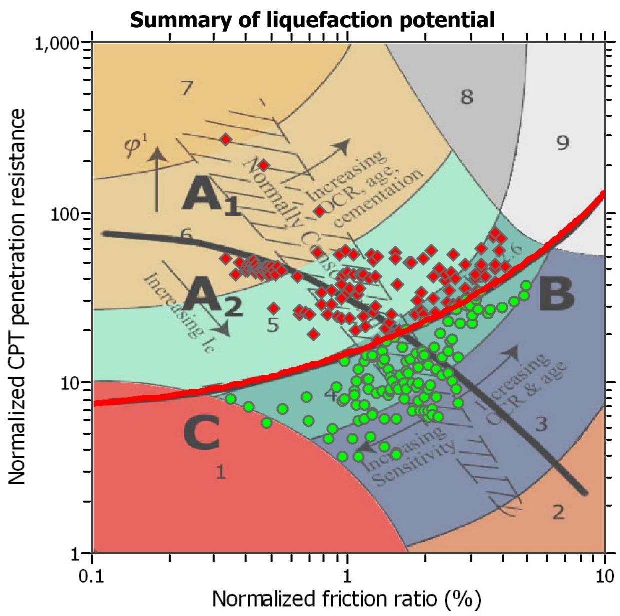
CPT file : CPT-02

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	7.60 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	7.50 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.55	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.58	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

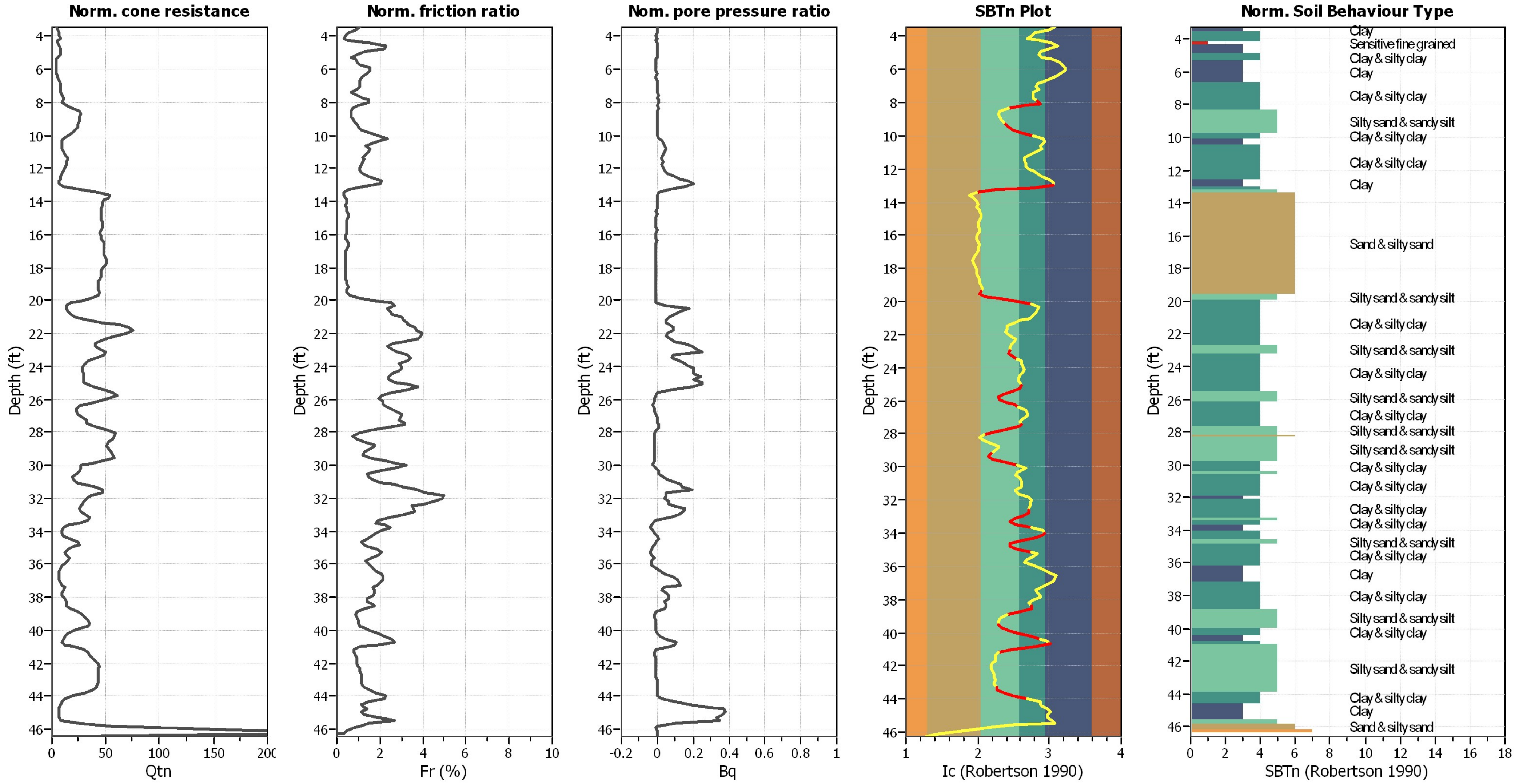






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots (normalized)



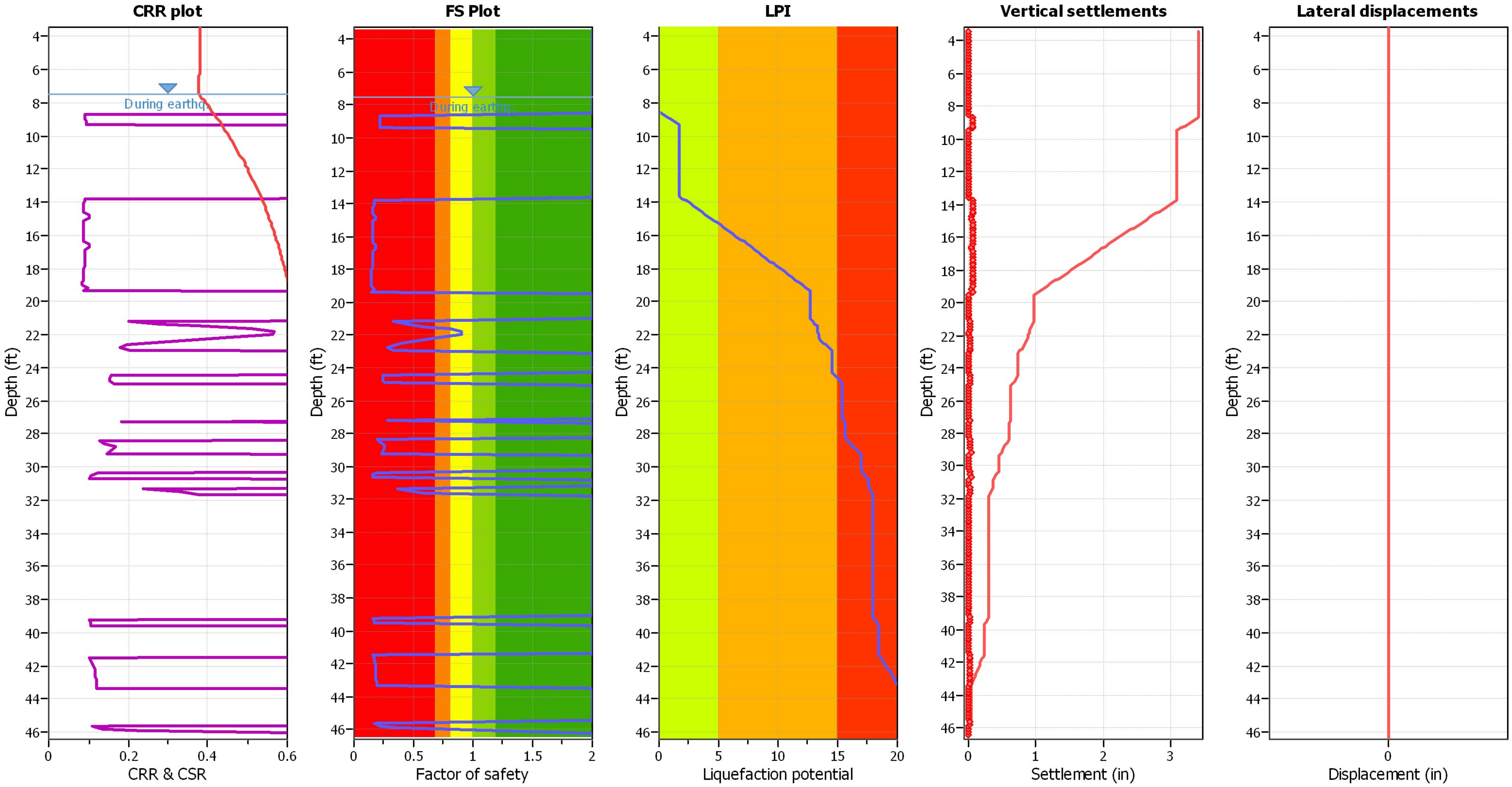
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	7.50 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.55	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.58	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	7.60 ft	Fill height:	N/A	Limit depth:	N/A

SBTn legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	7.50 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.55	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.58	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	7.60 ft	Fill height:	N/A	Limit depth:	N/A

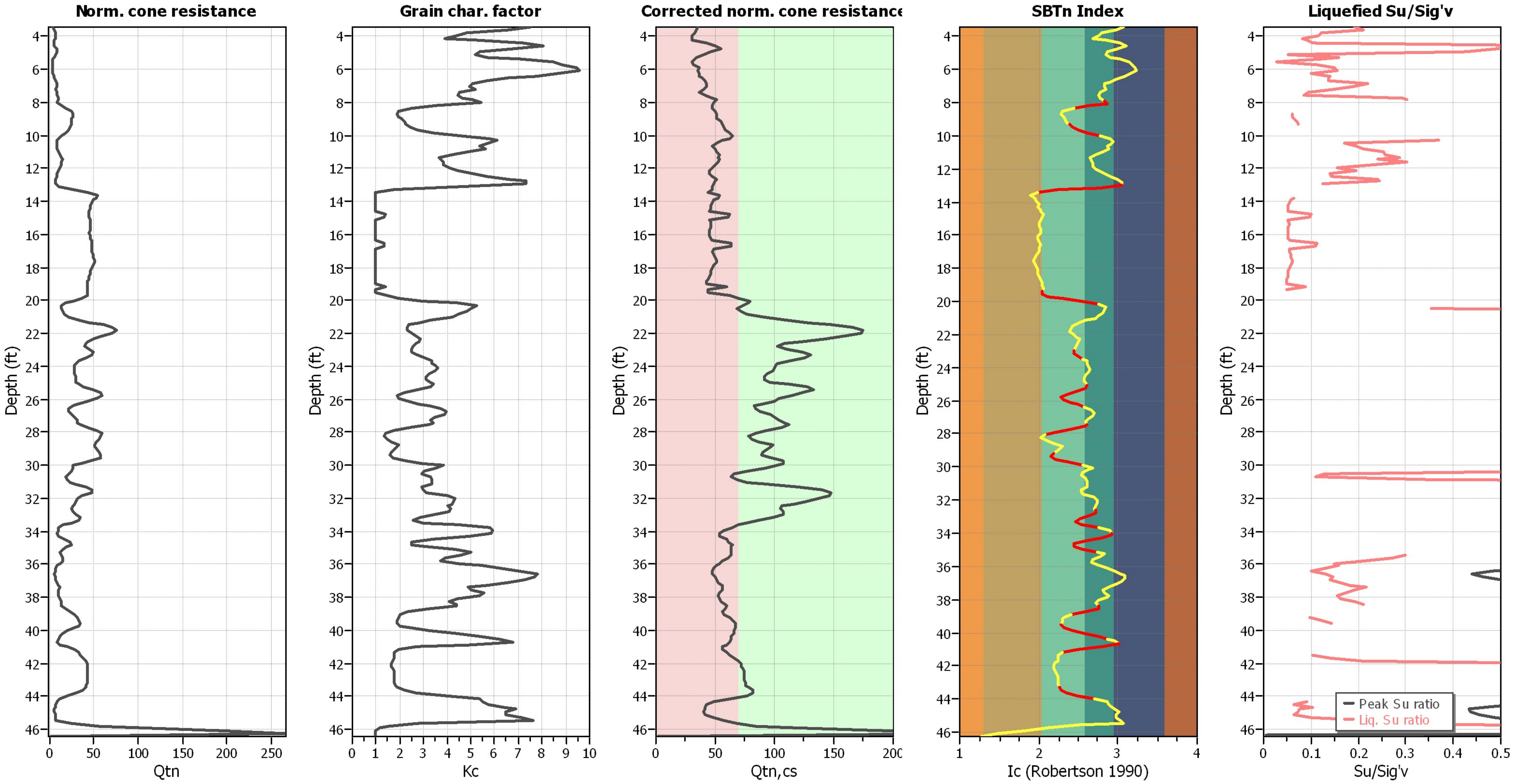
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

Check for strength loss plots (Robertson (2010))



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	7.50 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.55	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.58	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	7.60 ft	Fill height:	N/A	Limit depth:	N/A

Professional Service Industries, Inc.
6032 N. Cutter Circle
Suite 480
Portland, Oregon 97217

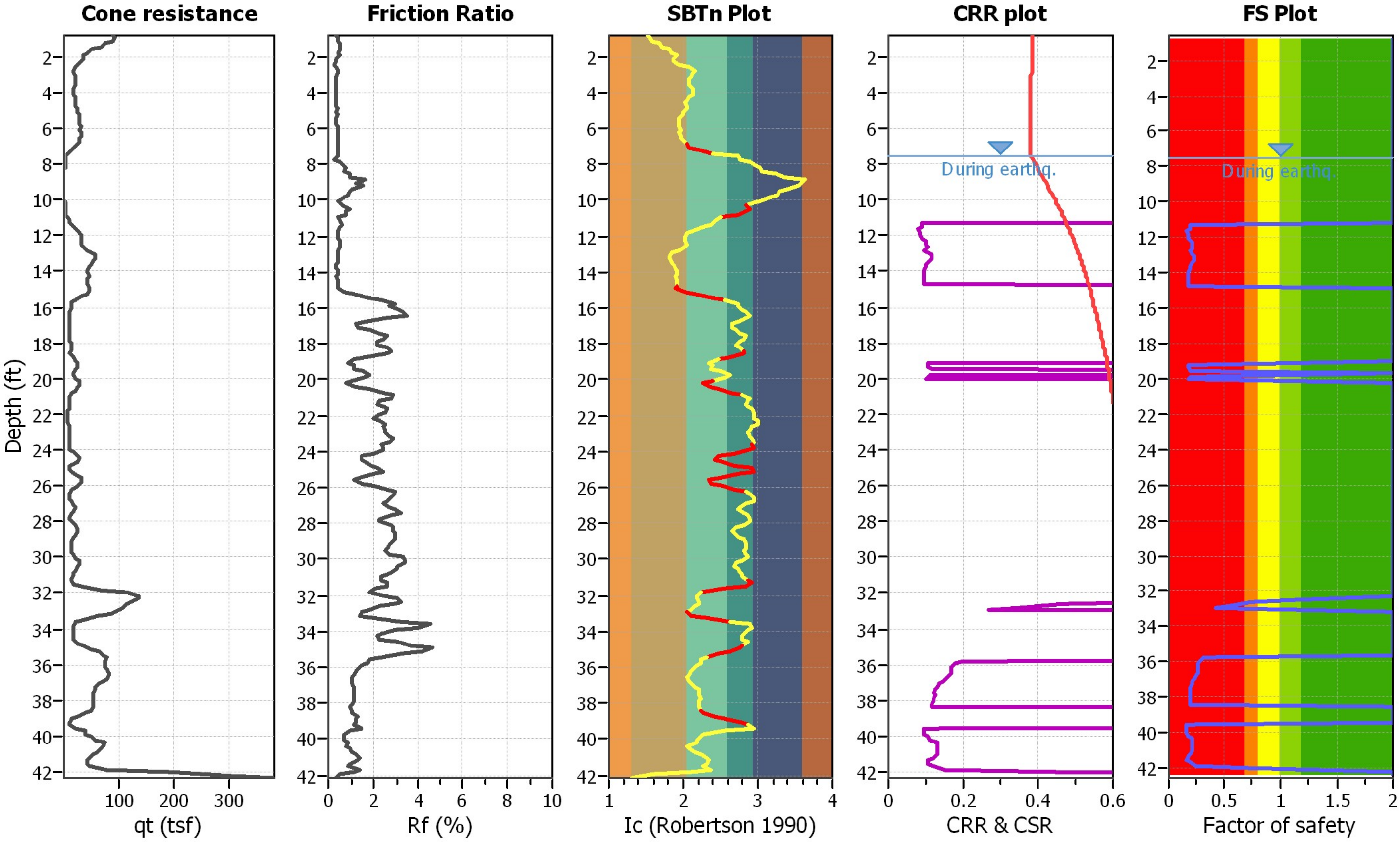
LIQUEFACTION ANALYSIS REPORT

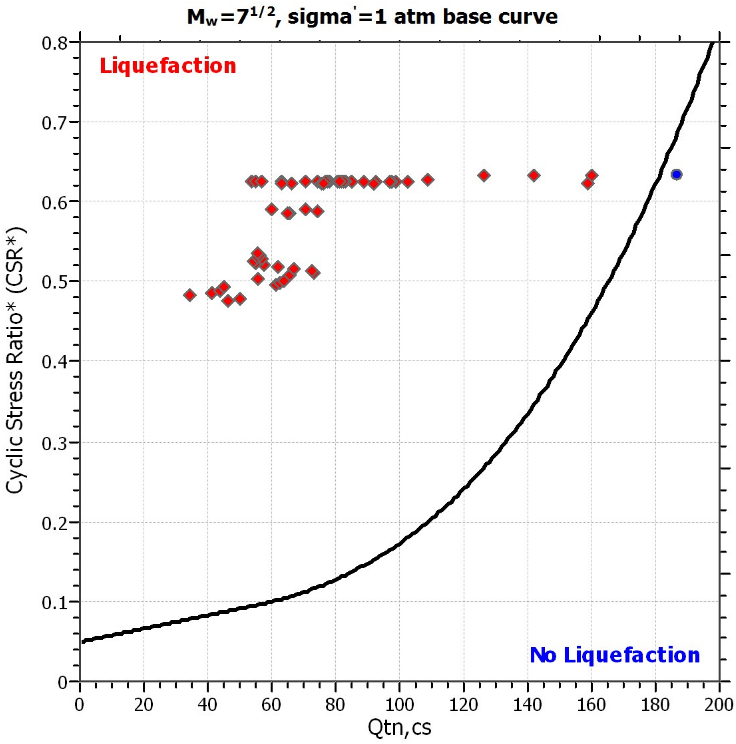
Project title : Philips 66 Portland Terminal **Location : Portland, OR**

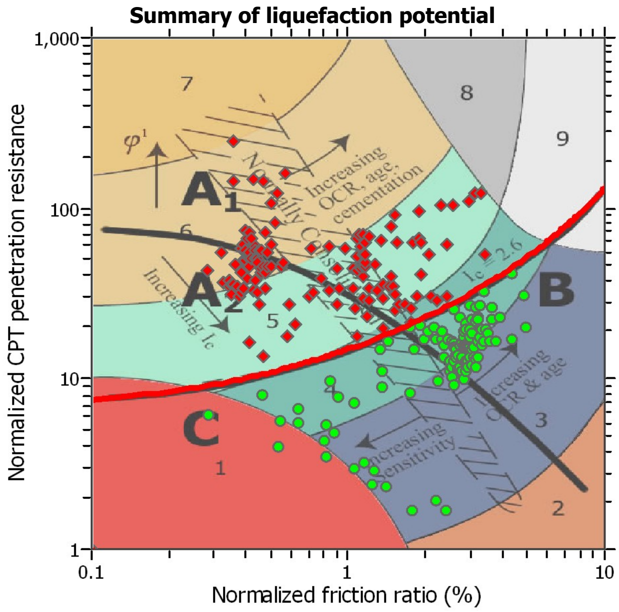
CPT file : CPT-03

Input parameters and analysis data

Analysis method:	NCEER (1998)	G.W.T. (in-situ):	12.50 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	NCEER (1998)	G.W.T. (earthq.):	7.50 ft	Fill height:	N/A	Limit depth applied:	No
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	N/A
Earthquake magnitude M_w:	7.55	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method based
Peak ground acceleration:	0.58	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

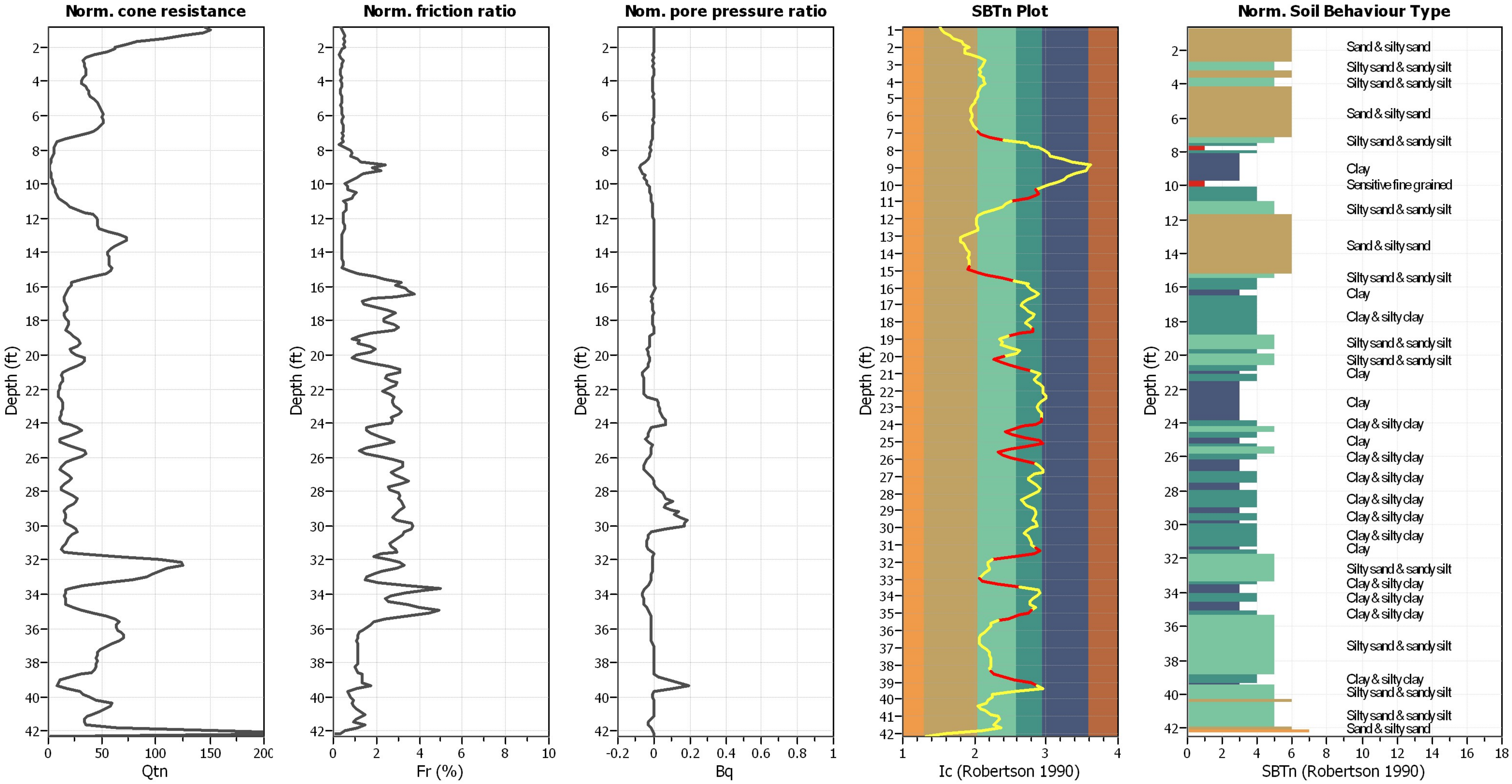






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and strength loss possible depending on soil plasticity, brittleness/sensitivity, strain to peak undrained strength and ground geometry

CPT basic interpretation plots (normalized)



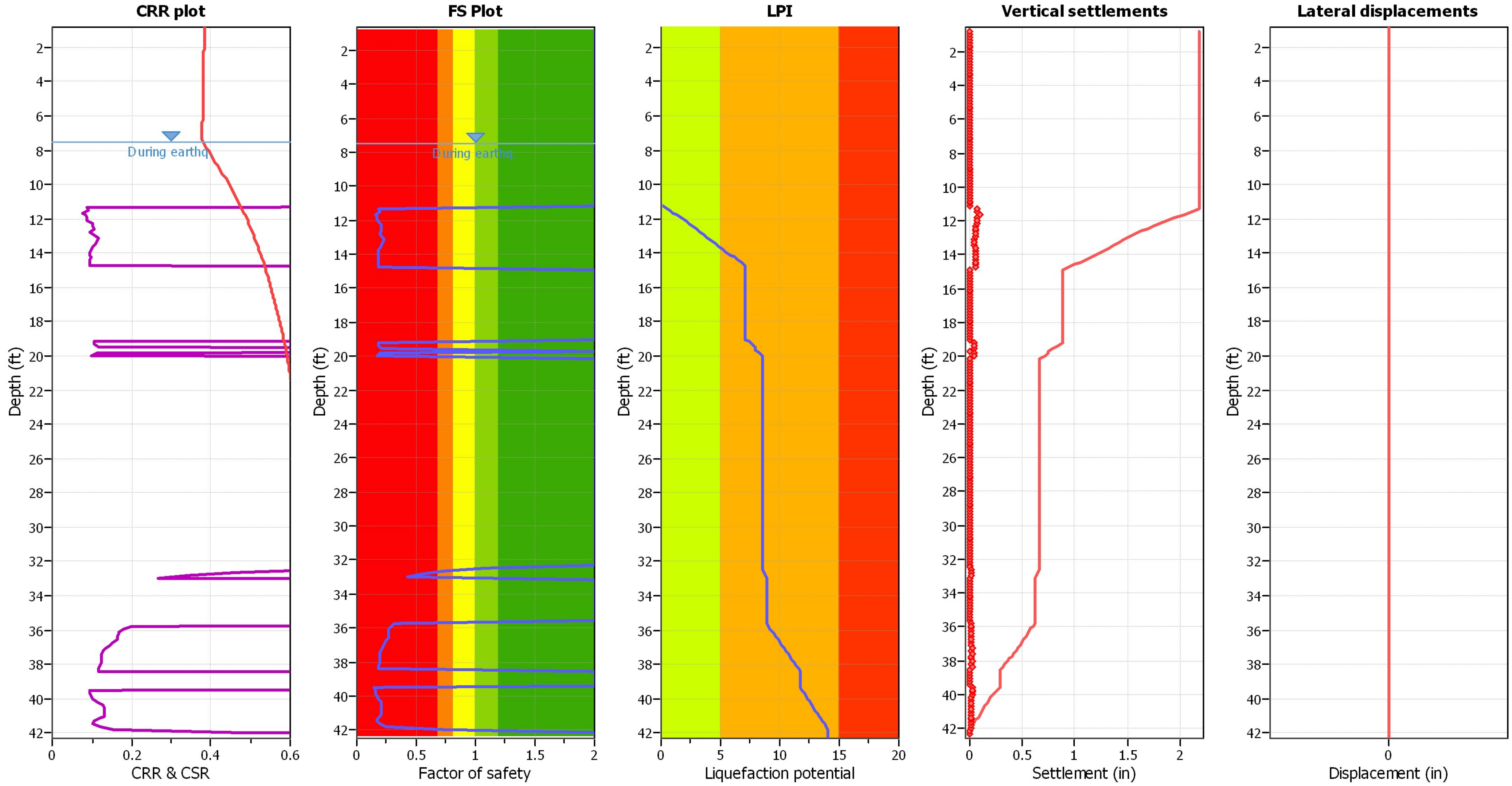
Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	7.50 ft	Fill weight:	N/A		
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes		
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes		
Earthquake magnitude M_w:	7.55	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only		
Peak ground acceleration:	0.58	Use fill:	No	Limit depth applied:	No		
Depth to water table (insitu):	12.50 ft	Fill height:	N/A	Limit depth:	N/A		

SBTn legend

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

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	7.50 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.55	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.58	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	12.50 ft	Fill height:	N/A	Limit depth:	N/A

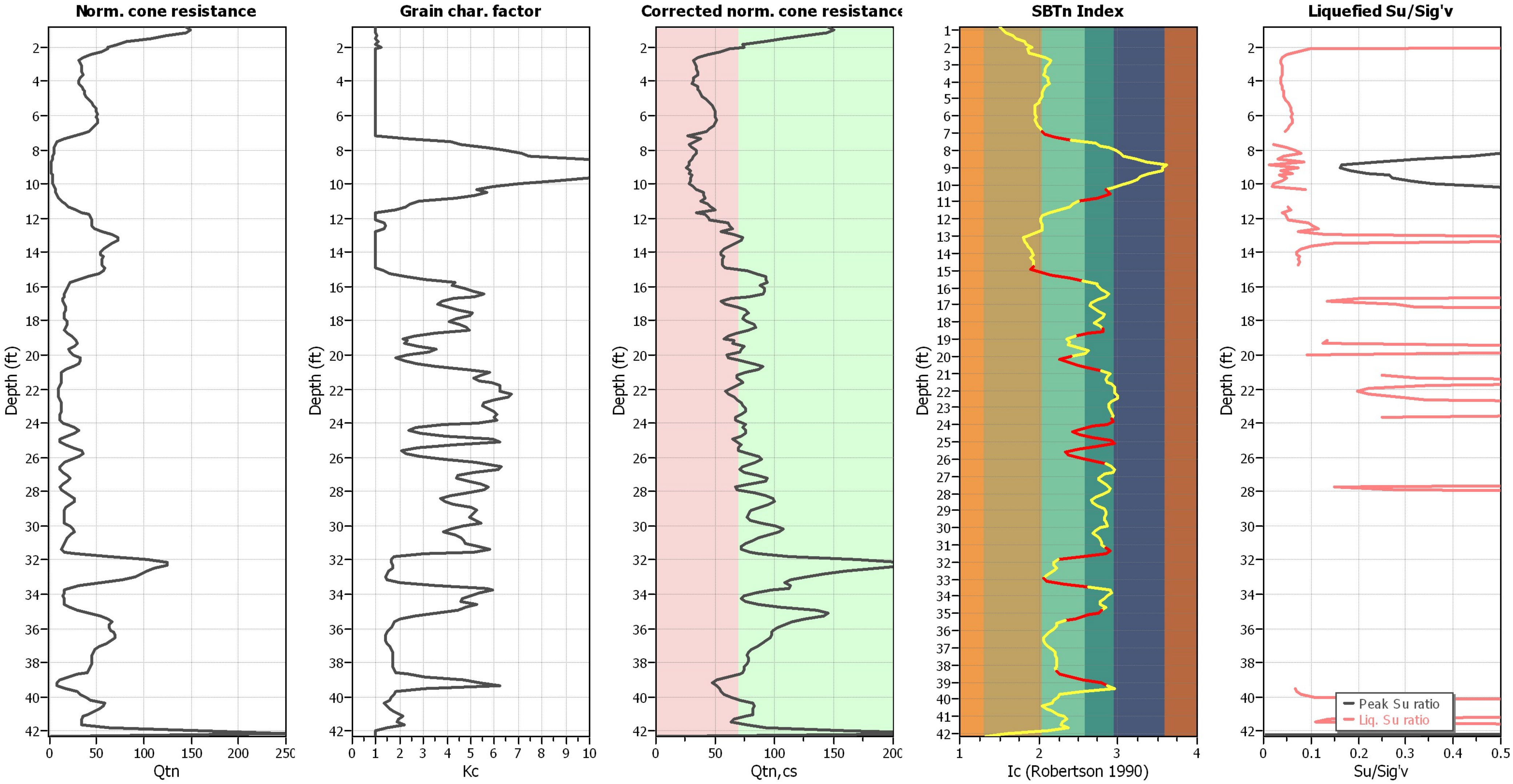
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

Check for strength loss plots (Robertson (2010))

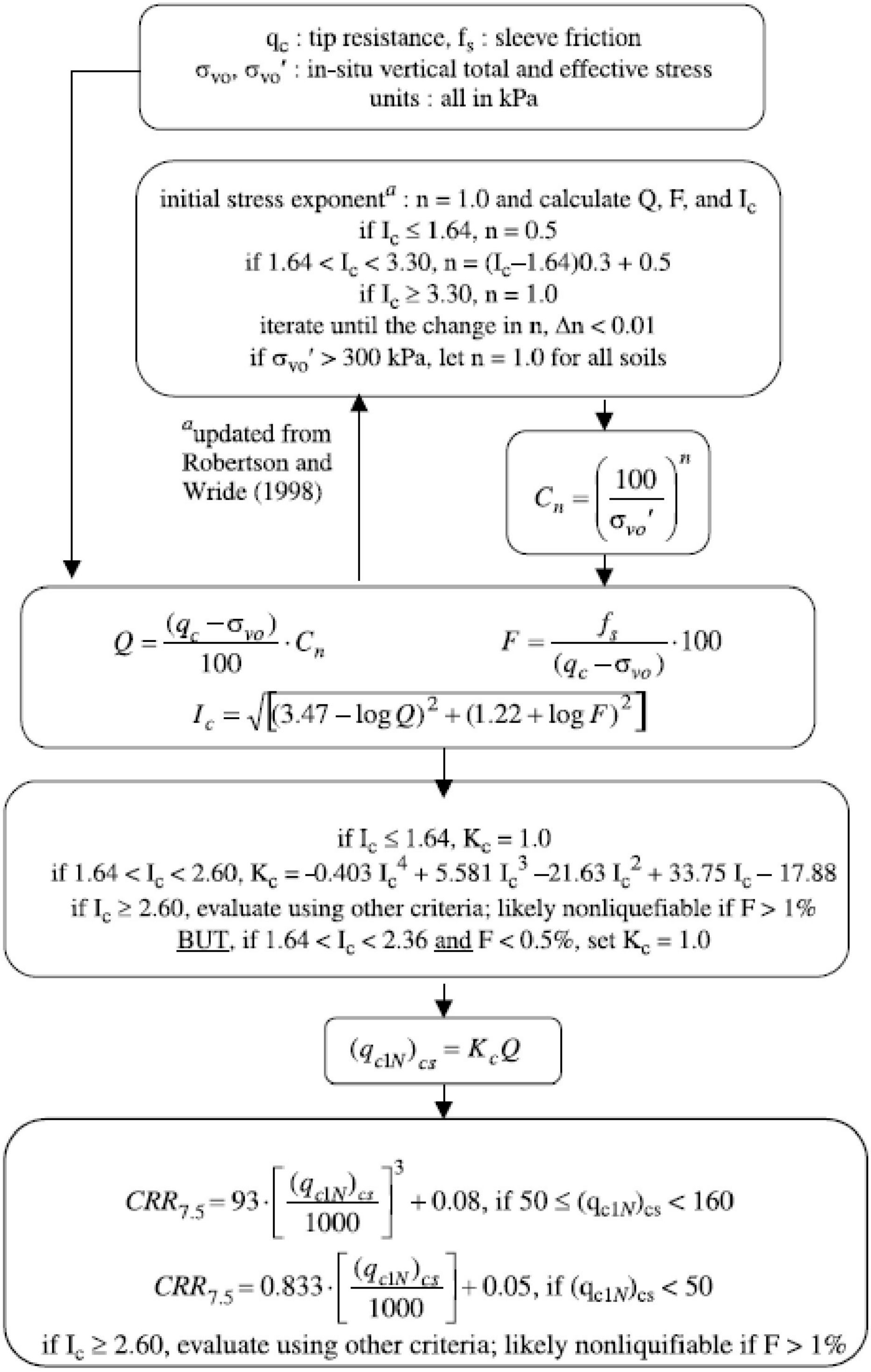


Input parameters and analysis data

Analysis method:	NCEER (1998)	Depth to water table (erthq.):	7.50 ft	Fill weight:	N/A
Fines correction method:	NCEER (1998)	Average results interval:	3	Transition detect. applied:	Yes
Points to test:	Based on Ic value	Ic cut-off value:	2.60	K_σ applied:	Yes
Earthquake magnitude M_w:	7.55	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.58	Use fill:	No	Limit depth applied:	No
Depth to water table (insitu):	12.50 ft	Fill height:	N/A	Limit depth:	N/A

Procedure for the evaluation of soil liquefaction resistance, NCEER (1998)

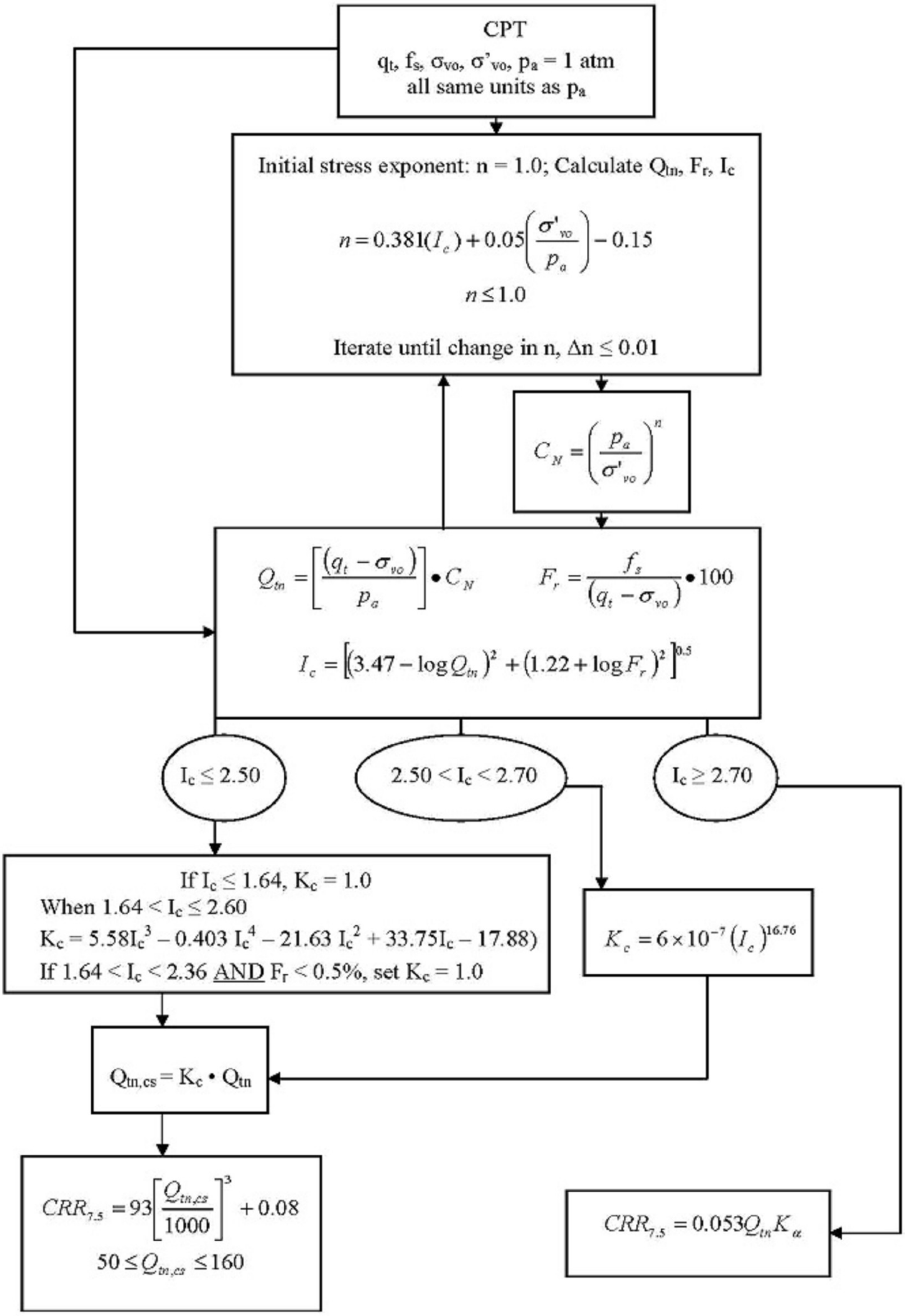
Calculation of soil resistance against liquefaction is performed according to the Robertson & Wride (1998) procedure. The procedure used in the software, slightly differs from the one originally published in NCEER-97-0022 (Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils). The revised procedure is presented below in the form of a flowchart[1]:



[1] "Estimating liquefaction-induced ground settlements from CPT for level ground", G. Zhang, P.K. Robertson, and R.W.I. Brachman

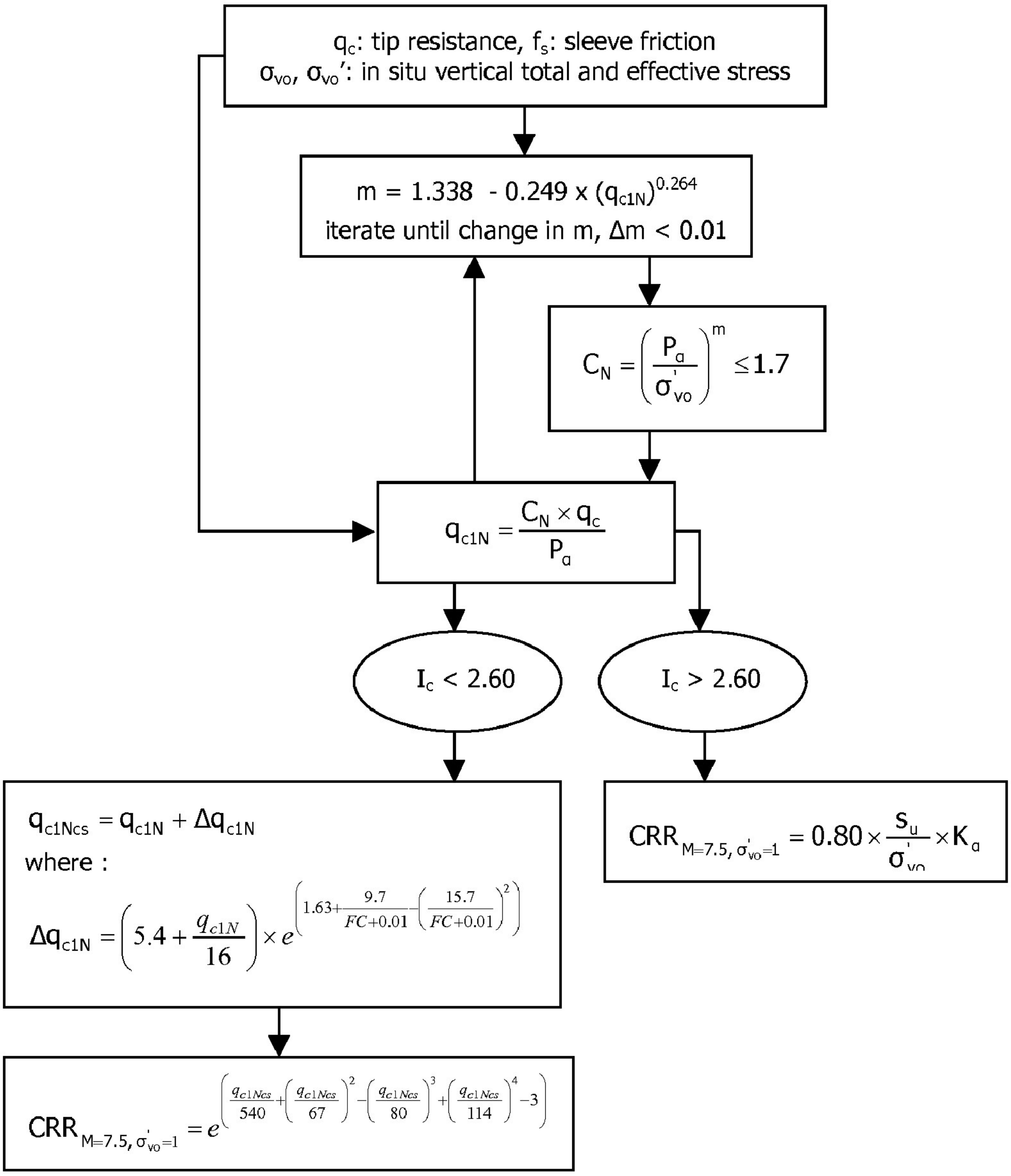
Procedure for the evaluation of soil liquefaction resistance (all soils), Robertson (2010)

Calculation of soil resistance against liquefaction is performed according to the Robertson & Wride (1998) procedure. This procedure used in the software, slightly differs from the one originally published in NCEER-97-0022 (Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils). The revised procedure is presented below in the form of a flowchart[1]:

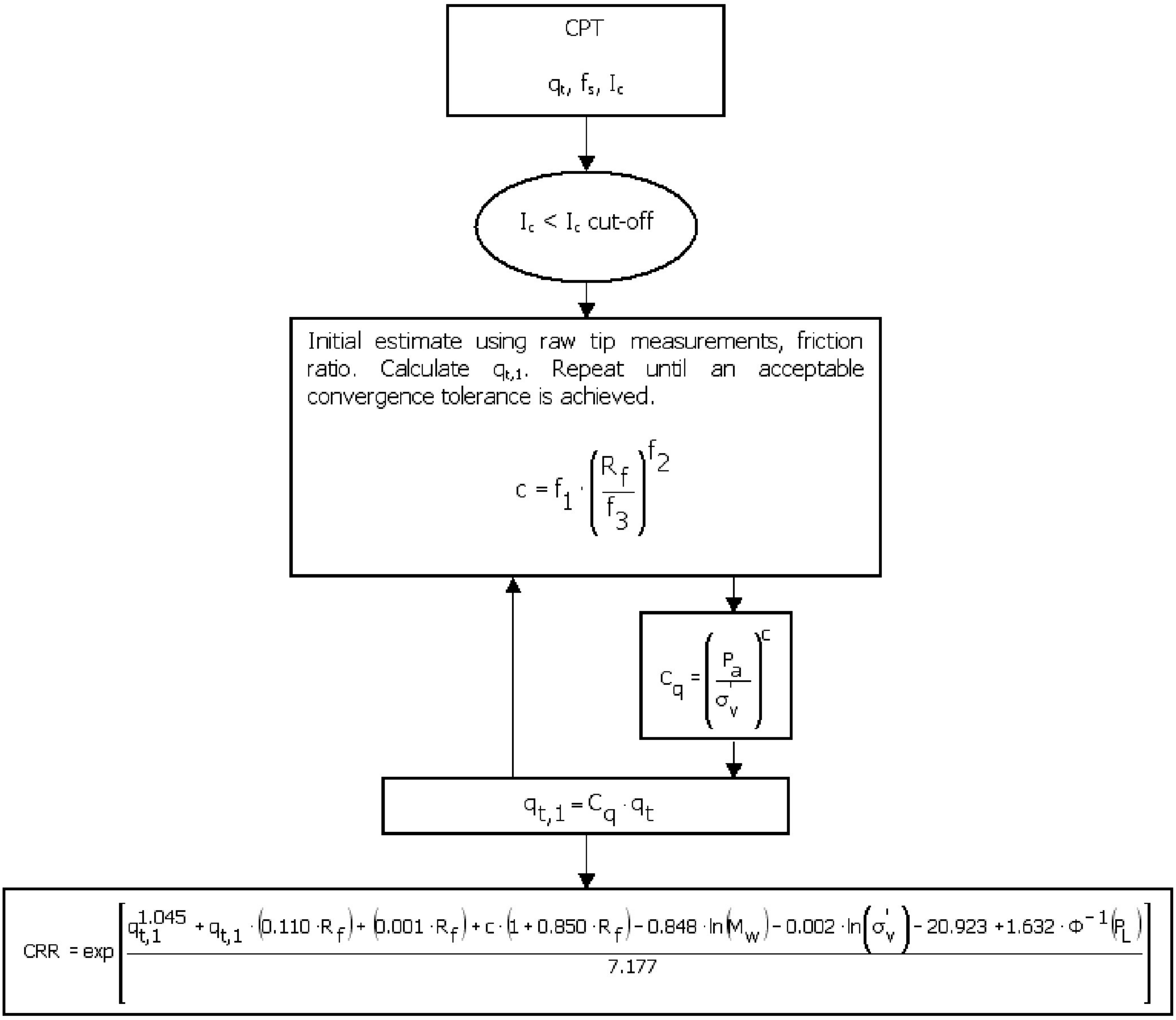


[1] P.K. Robertson, 2009. "Performance based earthquake design using the CPT", Keynote Lecture, International Conference on Performance-based Design in Earthquake Geotechnical Engineering – from case history to practice, IS-Tokyo, June 2009

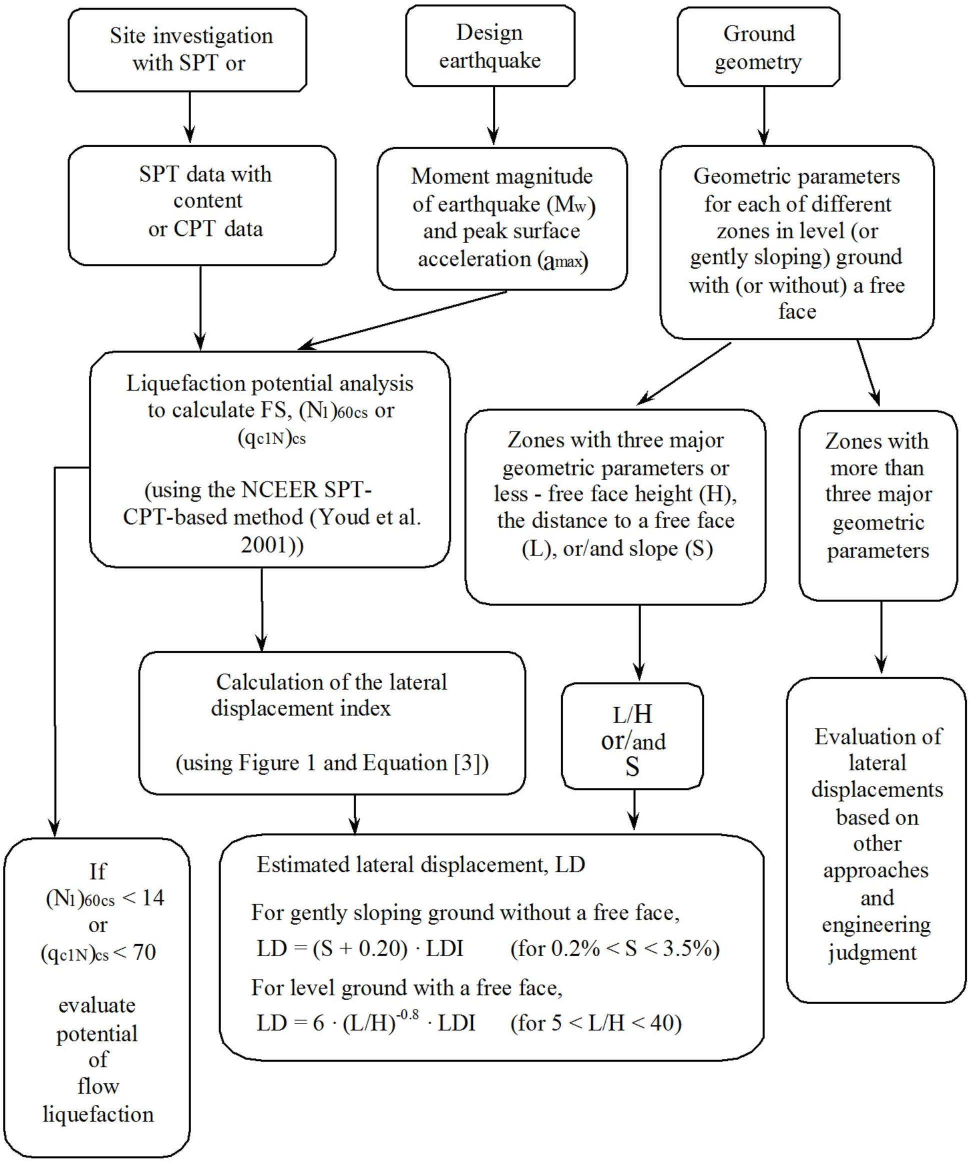
Procedure for the evaluation of soil liquefaction resistance, Idriss & Boulanger (2008)



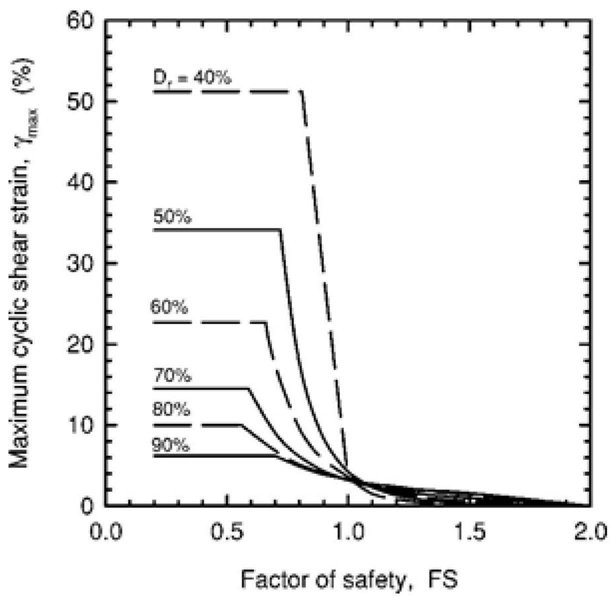
Procedure for the evaluation of soil liquefaction resistance (sandy soils), Moss et al. (2006)



Procedure for the evaluation of liquefaction-induced lateral spreading displacements



[1] Flow chart illustrating major steps in estimating liquefaction-induced lateral spreading displacements using the proposed approach



[1] **Figure 1**

$$LDI = \int_0^{Z_{max}} \gamma_{max} dz$$

[1] **Equation [3]**

[1] "Estimating liquefaction-induced ground settlements from CPT for level ground", G. Zhang, P.K. Robertson, and R.W.I. Brachman

Procedure for the estimation of seismic induced settlements in dry sands

Average shear stress, τ_{av}

$$\tau_{av} = CSR^{*} \cdot \sigma_{v0}' = 0.65 \cdot \frac{\alpha_{max}}{g} \cdot \sigma_{v0} \cdot \tau_{a}$$

↓

Estimate small shear strain modulus, G_0

$$G_0 = 0.0188 \cdot \left[10^{(0.55 I_c - 1.68)}\right] \cdot (q_t - \sigma_v)$$

↓

Estimate shear strain amplitude, γ (based on Pradel (1998))

$$\gamma = \left[\frac{1 + \alpha \cdot e^{bR}}{1 + \alpha}\right] \cdot R \cdot 100\ (\%)$$

$$R = \frac{\tau_{av}}{G_0}\ (\text{Note } \tau_{av} \text{ and } G_0 \text{ same units})$$

$$\alpha = 0.0389 \cdot \left(\frac{\sigma_v}{p_a}\right) + 0.124$$

$$b = 6400 \cdot \left(\frac{\sigma_v}{p_a}\right)$$

↓

Estimate volumetric strain in 15 cycles

$$\varepsilon_{vol(15)} = \gamma \cdot \left[\frac{(N_1)_{60,cs}}{20}\right]^{-1.20}$$

$$(N_1)_{60,cs} = \frac{Q_{tn,cs}}{8.5 \cdot \left[1 - \frac{I_c}{4.6}\right]}$$

↓

Volumetric strain in design earthquake

$$\Sigma_{vol} = \varepsilon_{vol(15)} \cdot \left[\frac{N_c}{15}\right]^{0.45}$$

$$N_c = (M - 4)^{2.17}$$

↓

Seismic settlement, s

$$s = 2 \cdot \int_0^{GWT} \Sigma_{vol} \cdot dz$$

Robertson, P.K. and Lisheng, S., 2010, "Estimation of seismic compression in dry soils using the CPT" FIFTH INTERNATIONAL CONFERENCE ON RECENT ADVANCES IN GEOTECHNICAL EARTHQUAKE ENGINEERING AND SOIL DYNAMICS, Symposium in honor of professor I. M. Idriss, San Diego, CA

Liquefaction Potential Index (LPI) calculation procedure

Calculation of the Liquefaction Potential Index (LPI) is used to interpret the liquefaction assessment calculations in terms of severity over depth. The calculation procedure is based on the methology developed by Iwasaki (1982) and is adopted by AFPS.

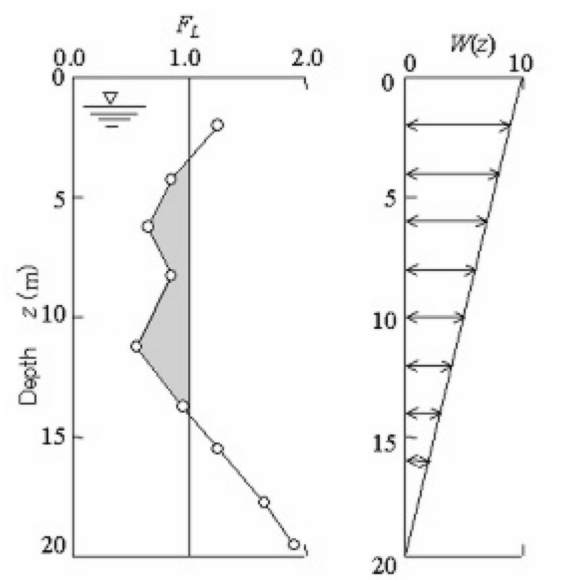
To estimate the severity of liquefaction extent at a given site, LPI is calculated based on the following equation:

$$\textbf{LPI} = \int_0^{20} (10 - 0{,}5_z) \times F_l \times d_z$$

where:
F_L = 1 - F.S. when F.S. less than 1
F_L = 0 when F.S. greater than 1
z depth of measurment in meters

Values of LPI range between zero (0) when no test point is characterized as liquefiable and 100 when all points are characterized as susceptible to liquefaction. Iwasaki proposed four (4) discrete categories based on the numeric value of LPI:

- LPI = 0 : Liquefaction risk is very low
- 0 < LPI <= 5 : Liquefaction risk is low
- 5 < LPI <= 15 : Liquefaction risk is high
- LPI > 15 : Liquefaction risk is very high



Graphical presentation of the LPI calculation procedure

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- Robertson, P.K. and Lisheng, S., 2010, “Estimation of seismic compression in dry soils using the CPT” FIFTH INTERNATIONAL CONFERENCE ON RECENT ADVANCES IN GEOTECHNICAL EARTHQUAKE ENGINEERING AND SOIL DYNAMICS, *Symposium in honor of professor I. M. Idriss,* SAN diego, CA

- R. E. S. Moss, R. B. Seed, R. E. Kayen, J. P. Stewart, A. Der Kiureghian, K. O. Cetin, CPT-Based Probabilistic and Deterministic Assessment of In Situ Seismic Soil Liquefaction Potential, Journal of Geotechnical and Geoenvironmental Engineering, Vol. 132, No. 8, August 1, 2006

Important Information about Your
Geotechnical-Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical-engineering study conducted for a civil engineer may not fulfill the needs of a construction contractor or even another civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client. No one except you should rely on your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you* — should apply the report for any purpose or project except the one originally contemplated.

Read the Full Report

Serious problems have occurred because those relying on a geotechnical-engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

A Geotechnical-Engineering Report Is Based on a Unique Set of Project-Specific Factors

Geotechnical engineers consider many unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk-management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical engineering report that was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical-engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light-industrial plant to a refrigerated warehouse,
- elevation, configuration, location, orientation, or weight of the proposed structure,
- composition of the design team, or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical-engineering report is based on conditions that existed at the time the study was performed. *Do not rely on a geotechnical-engineering report* whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, droughts, earthquakes, or groundwater fluctuations. *Always* contact the geotechnical engineer before applying the report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ—sometimes significantly—from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final,* because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations *only* by observing actual subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.*

A Geotechnical Engineering Report Is Subject to Misinterpretation

Other design team members' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Lower that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical-engineering report. Reduce that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

Give Contractors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical-engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure contractors have sufficient time* to perform additional study. Only then might you be in a position to give contractors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and contractors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform a *geoenvironmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical-engineering report does not usually relate any geoenvironmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own geoenvironmental information, ask your geotechnical consultant for risk management guidance. *Do not rely on an environmental report prepared for someone else.*

Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the *express purpose* of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold-prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, many mold-prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical-engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold-prevention consultant; ***none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.***

Rely, on Your GBA-Member Geotechncial Engineer for Additional Assistance

Membership in the Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk confrontaton techniques that can be of genuine benefit for everyone involved with a construction project. Confer with your GBA-member geotechnical engineer for more information.

GEOPROFESSIONAL BUSINESS ASSOCIATION

8811 Colesville Road/Suite G106, Silver Spring, MD 20910
Telephone: 301/565-2733 Facsimile: 301/589-2017
e-mail: info@geoprofessional.org www.geoprofessional.org

Appendix B
Tanks

B. Form 2: Checklist for Tanks to Comply with OAR 340-300 (TNK)

B.1 TNK1

1. *Submit a plan view of the tank farm, to scale, including cross-sections and dimensions of all berms.*

A site map of the tank farm is shown in Appendix D. The plan does not include cross-sections or dimensions of the berm surrounding Tank Farm 3 at this time.

B.2 TNK2

2. *For each tank, provide tank age, any previous inspection records, contents, dimensions (height and diameter) and type of anchorage to the concrete foundation. If the tank was built prior to 1988, there were not standards for anchorage or design (Ref. CalARP). If a tank is empty, provide details of how long since it was used and whether or not it is permanently out of service. From the results of the geotechnical investigations or reports verify the site class (A-F) with the appropriate seismic risk. For the parallel treatment of tanks compared to the requirement of Risk Classification IV (Per OAR 340-300-0003(a)(a) and Table 1.5.1, ASCE 7), the analogous treatment for tanks would be the "SUG III" Classification.*

The terminal and lubricant distribution facility includes 116 aboveground storage tanks. Most of the larger tanks on the site are ground supported steel tanks. Many of these tanks do not have anchors connecting the tanks to a foundation and are classified as unanchored. Some of the tanks located in the lube cell areas are steel tanks that are slightly elevated on steel leg pedestals with anchors. Table B-1 lists the site's tanks and some relevant tank characteristics based on available information. The information in the table is intended to partially satisfy the information requested as part of Oregon DEQ's Form 2: Checklist for Tanks to Comply with OAR 340-300 Part 2. At this time, a comprehensive catalog of the tanks and the seismic-relevant tank properties has not been able to be developed. This facility has existed in some capacity for over 100 years. Many of the facility's oldest tanks are permanently out of service. The facility has many tanks that were built throughout the 20th century. Many of the facility's tanks do not have anchors connecting the tank to a foundation and are, therefore, classified as unanchored. The facility's site class has been identified by the geotechnical engineer as Site Class F due to the site's liquefaction potential.

Table B-1. Facility Tank Information Summary.

Location within Facility	Tank Identifier	Contents	Max Operating Capacity (bbls)
Tank Farm 1	T-36	Slop Oil	522
Tank Farm 1	T-1471	Hydraulic Tractor Fluid	464
Tank Farm 1	T-2561	Out of Service	39,644

Table B-1. Facility Tank Information Summary.

Location within Facility	Tank Identifier	Contents	Max Operating Capacity (bbls)
Tank Farm 1	T-2579	Hydraulic Tractor Fluid	480
Tank Farm 1	T-2669	Out of Service	11,361
Tank Farm 1	T-2713	Lube Oil	2,820
Tank Farm 1	T-2714	Lube Oil	2,820
Tank Farm 1	T-2783	Out of Service	23,458
Tank Farm 1	T-2784	Diesel	34,750
Tank Farm 1	T-2917	Lube Oil	15,470
Tank Farm 1	T-3623	HiTec Add	485
Tank Farm 1	T-3639	Lube Oil	3,147
Tank Farm 1	T-4369	Lube Oil	485
Tank Farm 1	T-4441	Diesel Lubricity	485
Tank Farm 1	VRU Contactor	Gasoline	20
Tank Farm 1	Stainless Steel Totes	Diesel Winter Add (1)	5
Tank Farm 1	Stainless Steel Totes	Red Dye (2)	16
Tank Farm 2	T-2915	Gasoline	84,600
Tank Farm 2	T-2916	Diesel	40,850
Tank Farm 2	T-2982	Out of Service	11,602
Tank Farm 2	T-2983	Lube Oil	7,614
Tank Farm 2	T-3407	Gasoline	80,571
Tank Farm 2	T-3408	Gasoline	45,321
Tank Farm 2	T-3409	Gasoline	27,417
Tank Farm 2	T-3410	Dn-Ethanol	7,197
Tank Farm 2	T-3411	Gasoline	7,197
Tank Farm 2	T-3412	Dn-Ethanol	7,197
Tank Farm 2	T-3413	Gasoline	7,197
Tank Farm 2	T-4259	Trans-mix	5,483
Tank Farm 3	T-3414	Lube Oil	5,288
Tank Farm 3	T-3415	Lube Oil	5,288
Tank Farm 3	T-3416	Lube Oil	5,288
Tank Farm 3	T-3417	Lube Oil	5,288
Tank Farm 3	T-3579	Diesel	80,571
Tank Farm 3	T-3739	Lube Oil	5,288
Tank Farm 3	T-3740	Lube Oil	7,197
Tank Farm 3	T-3761	Diesel	80,571
Tank Farm 3	T-4244	Lube Oil	485
Tank Farm 3	T-4245	Out of Service	485
Tank Farm 3	T-4252	Out of Service	11,190
Tank Farm 3	T-4253	Out of Service	11,190
Tank Farm 3	T-4254	Out of Service	11,190
Tank Farm 3	T-4255	Biodiesel	11,190
Tank Farm 3	T-4258	Recovery Oil	485
Tank Farm 3	T-4266	Recovery Oil	485

Table B-1. Facility Tank Information Summary.

Location within Facility	Tank Identifier	Contents	Max Operating Capacity (bbls)
Tank Farm 3	T-4302	Lube Oil	485
Tank Farm 3	T-4303	Out of Service	485
Tank Farm 3	T-4305	Out of Service	243
Tank Farm 3	T-4306	Lube Oil	5,314
Tank Farm 3	T-4318	Diesel	38,719
Tank Farm 3	T-4320	Lube Oil	1,014
Tank Farm 3	T-4321	Lube Oil	1,014
Tank Farm 3	T-4322	Lube Oil	1,014
Tank Farm 3	T-4323	Lube Oil	1,014
Tank Farm 3	F-103	Lube Oil	699
Tank Farm 3	F-104	Lube Oil	517
Tank Farm 3	PVL-0008	Butane	2,142
Tank Farm F	T-4335	Lube Oil	485
Tank Farm F	T-4336	Lube Oil	485
Tank Farm F	T-4337	Lube Oil	485
Tank Farm F	T-4436	Lube Oil	485
Tank Farm F	T-4437	Lube Oil	485
Tank Farm F	F-10	Lube Oil	162
Tank Farm F	F-11	Lube Oil	162
Tank Farm F	F-12	Lube Oil	162
Upper Lube Cell	T-3741	Lube Oil	485
Upper Lube Cell	T-3742	Lube Oil	485
Upper Lube Cell	T-3743	Lube Oil	485
Upper Lube Cell	T-3744	Lube Oil	485
Upper Lube Cell	T-3745	Lube Oil	485
Upper Lube Cell	T-3746	Lube Oil	485
Upper Lube Cell	T-3747	Lube Oil	485
Upper Lube Cell	T-3757	Lube Oil	485
Upper Lube Cell	T-3760	Lube Oil	485
Upper Lube Cell	T-4191	Lube Oil	485
Upper Lube Cell	T-4192	Lube Oil	485
Upper Lube Cell	T-4241	Lube Oil	485
Upper Lube Cell	T-4242	Lube Oil	485
Upper Lube Cell	T-4243	Lube Oil	485
Upper Lube Cell	T-4281	Lube Oil	485
Upper Lube Cell	T-4332	Lube Oil	485
Upper Lube Cell	T-4333	Lube Oil	485
Upper Lube Cell	T-4334	Lube Oil	485
Lower Lube Cell	T-4300	Lube Oil	699
Lower Lube Cell	T-4331	Lube Oil	699
Lower Lube Cell	T-4388	Lube Oil	289
Lower Lube Cell	T-4389	Lube Oil	289

Table B-1. Facility Tank Information Summary.

Location within Facility	Tank Identifier	Contents	Max Operating Capacity (bbls)
Lower Lube Cell	T-4390	Lube Oil	289
Lower Lube Cell	T-4391	Lube Oil	289
Lower Lube Cell	T-4392	Lube Oil	289
Lower Lube Cell	T-4393	Lube Oil	289
Lower Lube Cell	T-4394	Lube Oil	289
Lower Lube Cell	T-4395	Lube Oil	289
Lower Lube Cell	T-4397	Lube Oil	289
Lower Lube Cell	T-4398	Lube Oil	289
Lower Lube Cell	T-4399	Lube Oil	289
Lower Lube Cell	T-4400	Lube Oil	289
Lower Lube Cell	T-4400	Lube Oil	289
Lower Lube Cell	T-4401	Lube Oil	289
Lower Lube Cell	T-4402	Lube Oil	289
Lower Lube Cell	T-4403	Lube Oil	289
Lower Lube Cell	T-4404	Lube Oil	289
Lower Lube Cell	T-4405	Lube Oil	289
Lower Lube Cell	T-4406	Lube Oil	289
Lower Lube Cell	T-4407	Lube Oil	289
Lower Lube Cell	T-4408	Lube Oil	289

B.3 TNK3

3. *The first and preliminary inspection or assessment of the tank farm consists of a walk-through based on CalARP, with the seismic evaluations performed under the direction of an Oregon-registered civil, structural, or mechanical engineer (CalARP Section1.4). This includes a preliminary seismic assessment, using the seismic demand as provided in the initial geotechnical inspection/report required by the DEQ. This preliminary assessment would include possible liquefaction or lateral spreading, seismic settlement, and landslides (per CalARP 2.3, 2.4 and 2.5). This initial report provides some direction for the full tank assessment per API 653 and the rehabilitation or mitigation per API 650.*

The geotechnical engineer has identified the facility's site as having potentially liquefiable soil. The possibility of liquefaction-induced settlement in a seismic event appears to exist. Near the Willamette River waterfront, the possibility of lateral spreading in a seismic event also appears to exist. Please also see the response to TNK2 for a description of the understanding of the site's numerous storage tanks.

B.4 TNK4

4. *Per OAR 340-300-0004 (1)(a) retrofits, reconstruction or other mitigation measures must comply with ASCE 7 Risk Category IV. Per ASCE 7, Section 11.4.8; if the soil type is "F" a site-specific ground motion is used. Risk Category IV (ASCE 7, Table 1.5-1) implies that these tanks are "essential facilities". Per API 650, the seismic risk group would be SUG III (API 650, Annex E, Section E.3.1.1). With this seismic criterion, tank spills are limited to the MAUS (1 BBL/tank). Verify site classification and associated PGA/Spectra.*

In accordance with OAR 340-300-0004 (1)(a), it is anticipated that mitigation measures, where necessary, will be designed to achieve the performance objective of reducing the expected spill as a result of the Design level Earthquake to be below the MAUS. The facility's site class is identified as Site Class F by the geotechnical engineer. It is anticipated that site-specific ground motion procedures will be used for tank assessment and mitigation, however, additional engineering work (such as obtaining more detailed soil properties, etc) must be conducted by the geotechnical engineer prior to the development of site-specific ground motions.

B.5 TNK5

5. *For the comprehensive API 653 inspection, the inspectors must be certified by API. Provide copies of information as required in Annex D of API 653 (Section D.1 thru D.4). For the tank bottom inspectors, the procedures/personnel qualified must satisfy Appendix G (Sections G.1 thru G.5) of API 653. Obtain approval from DEQ before proceeding with the API 653 inspection process.*

Existing API 653 inspection reports can be submitted to Oregon DEQ. However, given the quantity of tanks at the facility and the short timeframe between publication of the assessment forms and the rules' deadline (~3 months between publication in March, 2024 and the June 1, 2024 deadline), the API 653 inspection reports have not been able to be collated at this time. Also, it is unclear if assessment Form 2 is requiring new API 653 inspections that go above-and-beyond typical inspections. OAR 340-300 mentions nothing about conducting condition assessments and mentions nothing about API 653 inspections. It is unclear if OAR 340-300 requires API 653 inspections.

B.6 TNK6

6. *Verify berm capacities are within allowable spill volumes, as stated in CFR 264.175(b). "Spill Prevention, Control, and Countermeasure Requirements" and that the secondary (e.g. berms) are sufficient to contain the entire contents of the largest tank or 10% of the total of all tanks (Containment, 40 CFR 264.175(b)) adding in precipitation, usually during the most severe 24-hour period.*

Based on the Phillips 66 Portland Terminal and Lubricants Spill Prevention and Control and Countermeasure (SPCC) Plan dated August 2020, each of the major tank farms have sufficient containment system capacity to accommodate the entirety of the product from the single largest tank within their containment area or 10-percent of the cumulative total volume of products within their containment area.

B.7 TNK7, TNK8, TNK9, TNK10

7. *[Referring to API 653] Section 4, "Suitability for Service" contains criteria and inspection activities. Evaluation questions and inspection procedures are provided for each of the following components. Each relevant question or evaluation shall be investigated.*
    a. *Roof tank (TNK7)*
    b. *Tank shell (TNK8)*
    c. *Tank bottom (TNK9)*
    d. *Tank foundation (TNK10)*

Please see the response to TNK5.

B.8 TNK11

8. *[Referring to API 653] Section 5, "Brittle Fracture Considerations" includes criteria and inspection activities for the assessment of existing tanks that might have a risk of brittle fracture. The assessment procedure of the 11 steps must be followed. Any deficiencies or issues shall be documented, and further action is required. Respond to each of the questions and provide all answers, if not applicable, respond N/A.*

Please see the response to TNK5.

B.9 TNK12

9. *[Referring to API 653] Section 6, "Inspections" – using the same numbering system of 6.1 thru 6.9 document all of the questions and include the reports in Section 6.9. This process must be completed for each tank within the facility. Tank inspections must be per Annex C. All tables must be submitted as presented in Table C.1 for in-service tanks and C.2 for out-of-service tanks. These checklists are to be followed, with any discrepancies listed. Tank bottom settlement shall be inspected/reported using Annex B. If the tanks have no existing corrosion historical rates, Annex H can be used, and the datasheets documented and reported. Tank inspection shall comply with Annex F, "Non-Destructive Examination." Tank qualification of tank bottom examination procedures, Annex G. Relevant sections of this annex should be applied as necessary. A report must follow the format of Section 6.9.2.*

Please see the response to TNK5.

B.10 TNK13

10. [Referring to API 653] Section 7, "Materials" – if any of this section 7.2 to 7.4 are applicable, explain/document for each tank. Respond to each of the questions and provide all answers; if not appliable, respond N/A.

Please see the response to TNK5.

Appendix C
Pipelines

ALPHA TECHNICAL GROUP INC.
Consulting Engineers
2929 N.W. 29th Avenue, Portland, OR. 97210
Phone (503) 227-3317 Fax (503) 227-3244

P66 Pipeline LLC- Portland Terminal
Seismic Vulnerability Assessment (SVA)
Piping Systems

Rev Date: May 9, 2024

REGISTERED PROFESSIONAL
ENGINEER
15,471
OREGON
JULY 25, 1991
JOHN DEPPA
EXPIRES 12/31/25

PRELIMINARY

Table of Contents

Appendix

1.0 Introduction & Scope

This report summarizes the results of a Seismic Vulnerability Assessment (SVA) of the P66 Pipeline's Portland Terminal's Product Piping Systems. This assessment is intended to directly address requirements of parts of the requirements outlined in the recently adopted Oregon DEQ "Fuel Tank Seismic Stability Rules" OAR 300. Alpha Technical Group Inc (ATGI) has designed various Piping System Improvements and updates to the facility's Piping & Instrumentation diagrams (PIDs) over many years.

2.0 Piping System(s) & Operations Overview

The product types and general routing of Terminals Product Piping System(s) are shown on drawing 0608-P-1100 located in the appendix. This shows current products in tanks which are subject to change over time depending on market conditions. Product transfer and related facilities are further described below:

<u>Gasoline:</u> Gasoline is received from vessels at the Marine Dock or from the Olympic Pipeline. It is currently sent to various storage tanks located in Tank Farm 1. Gasoline is typically loaded onto trucks at the main Truck Rack on Doane Ave. or onto barges at the Marine Dock. Gasoline is also shipped outbound to the Kinder Morgan Pipeline (serves Eugene, OR area).

<u>Diesel (including B100 Biodiesel)</u> Diesel is received from vessels at the Marine Dock or from the Olympic Pipeline. It is currently sent to various storage tanks located in Tank Farm 1/2/3. Diesel is typically loaded onto trucks at the main Truck Rack on Doane Ave., the Lane 4 Truck Rack (middle of site), and onto barges at the Marine Dock. Diesel is also shipped outbound to the Kinder Morgan Pipeline (serves Eugene, OR area).

<u>Ethanol:</u> Ethanol is received from trucks offload (middle of site). It is currently sent tank various storage tanks located in Tank Farm 2. Ethanol is then blended into gasoline and loaded onto trucks at the main Truck Rack on Doane Ave.

<u>Butane:</u> Butane is received from truck offloading (middle of site). It is currently sent to a single butane storage tank located south of Tank Farm 3. Butane is then blended into gasoline and loaded onto trucks at the main Truck Rack on Doane Ave. or onto barges at the Marine Dock.

<u>Transmix:</u> Transmix is a mixture of diesel and gasoline which is sometimes generated during typical terminal operations. It's stored in Tank 4259 located at Tank Farm 2 and then eventually loaded onto trucks.

<u>Lubricants:</u> Base Oil is received from vessels at the Marine Dock and currently sent to various storage tanks which are located in Tank Farms 2 and 3. Various additives are received from railcars near Tank Farm F. The base oils and additives are blended to make various lubricant oils (motor oil/ transmission fluid/etc.). Oils are stored in tanks at upper & lower lube cell areas and are bottled and stored in the warehouse.

3.0 Seismic Risk Assessment & Methodology

The ODEQ rules consider all petroleum products stored in tanks and piping as "oils" and thus limits the maximum allowable uncontained spill (MAUS) to 42 gallons. From a fire risk perspective, gasoline, ethanol, and butane are flammable liquids and thus have the highest risk of fires if released. Diesel (combustible) and lubricant oils have the lowest risk for fires if released.

<u>3.1 Assessment of Piping & Instrumentation System(s) functionality during Design Earthquake Event:</u>

This assessment will assess the ability of the Piping & Instrumentation System(s) to function during the design earthquake event and limit any piping releases outside of containment to MAUS levels. This assessment will be conducted by John Deppa PE and P66 Operations Staff using Process Hazard Analysis (PHA) procedures. This assessment will be completed after further geotechnical investigations are completed by Geo-Engineers Inc.- HOLD.

3.2 Assessment of Pipe Stress or Tank Nozzle failure during Design Earthquake Event:
This assessment will use pipe stress analysis to calculate loads from pipe onto tank nozzles during the design earthquake event. The most severe risk is that the pipe loads onto the tank nozzle themselves could rupture the tank shell itself and allow uncontrolled product release (i.e. base valve is no longer able to contain product in tank). Analysis will focus on the largest tanks within Tank Farms 1/2/3. The calculated loads onto nozzles will be provided to the Tank Structural Engineers(R-M) to assess the ability of the tank shells themselves to resist these loads and not rupture. This assessment will be completed after further geotechnical investigations by Geo-Engineers Inc.-HOLD.

4.0 Assessment Findings-HOLD

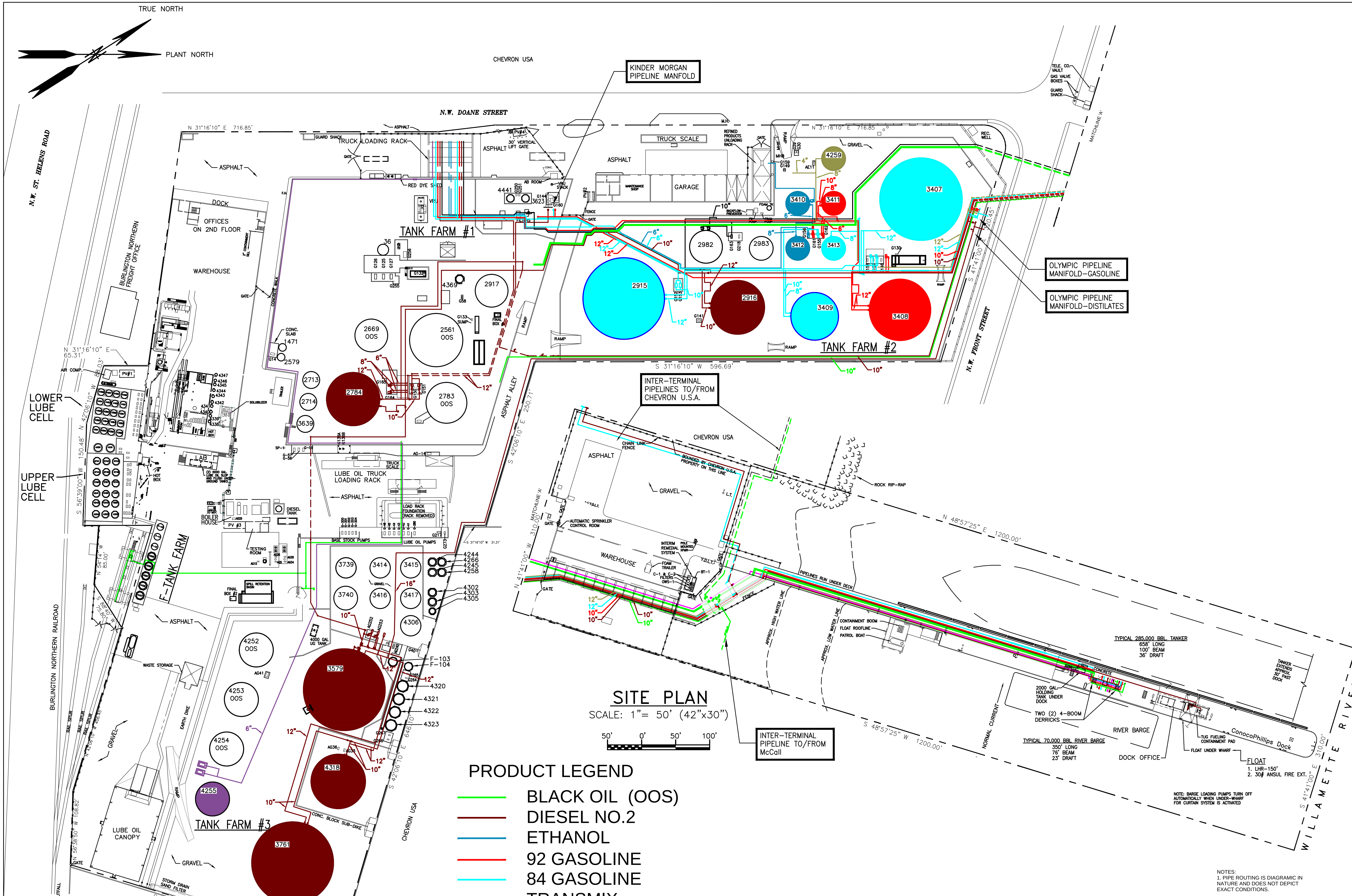
We look forward to discussing this report with you in more detail and hopefully can answer any questions you may have. If you have any other questions or require additional information, please feel free to contact me at your convenience.

Sincerely,

John Deppa, P.E., S.E.
Principal Engineer
.

Appendix 1

- Piping Systems Reference Drawings



NOTES:
1. PIPE ROUTING IS DIAGRAMIC IN NATURE AND DOES NOT DEPICT EXACT CONDITIONS.

REV. NO.	DATE	REVISED	DRAWN BY	CKD. BY
G	05/09/24	GENERAL REVISION	HVT	JPD
B	12/10/03	CHANGE TK 4318 TO DIESEL, TRANS. TANK UPDATE	DAB	
C	12/11/07	GENERAL REVISION	CWP	DAB
D	03/22/11	GENERAL REVISION	KLD	DAB
E	01/09/12	ADDED LINE SIZES	CWP	BDC
F	9-7-12	REMOVED TKS 4223,4327,4256 & 4257	LEJ	SSE

Phillips 66 Pipeline LLC

ALPHA TECHNICAL GROUP INC.
2929 NW 29th AVE, PORTLAND, OR. 97210
Tel:(503)227-3317/Fax:(503)227-3244

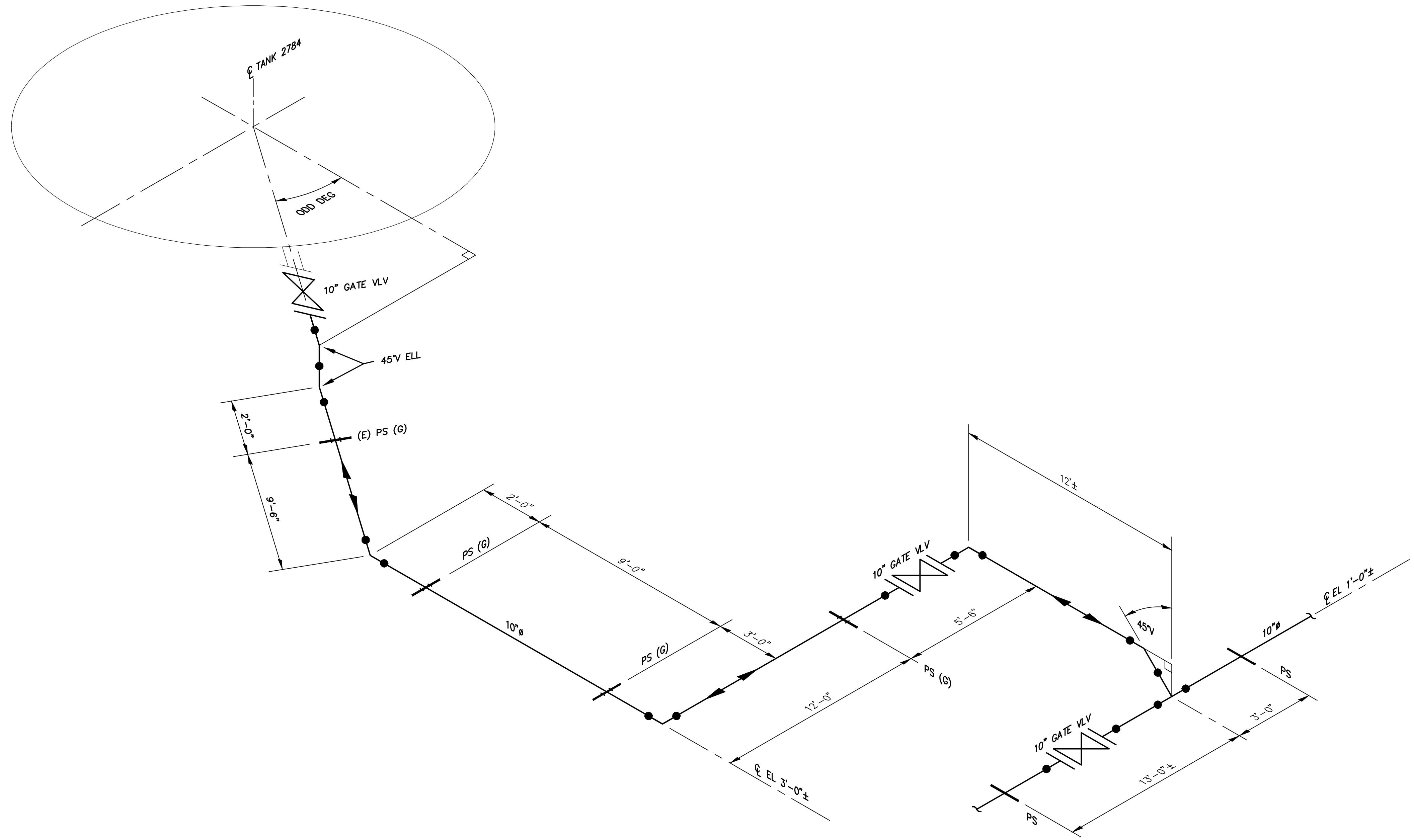
DRAWN BY D.A.BYNUM
CHKD BY W.H.BISHOP
APPD BY
SCALE 1"=50'

PORTLAND TERMINAL
PORTLAND, OREGON
PLOT PLAN
REFINED PRODUCTS
GENERAL PIPE ROUTING

DATE	DRAWING NO.	SHEET	REV.
04/11/00	0608–P–1000	1 OF 1	G

Appendix 2

- Pipe Stress Model Drawings



PIPE STRESS MODEL (TK 2784)

LEGEND:

1. "G" INDICATES PIPE GUIDE

NO.	REVISION	BY / CHKD	DATE / APP'D

ALPHA TECHNICAL GROUP INC.
2929 NW 29th AVE, PORTLAND, OR. 97210
Tel:(503)227-3317/Fax:(503)227-3244

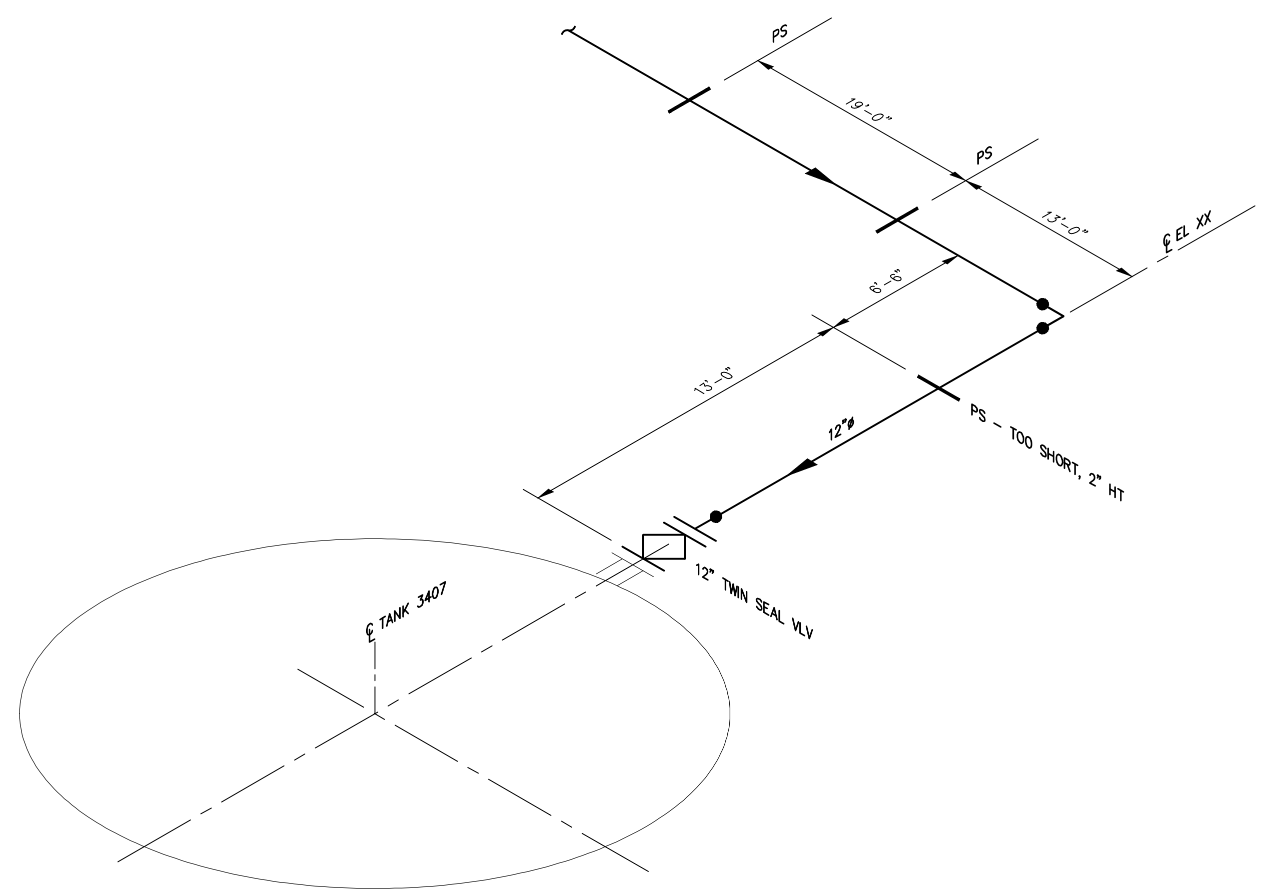
ATGI PROJECT NUMBER: 25-124

PHILLIPS 66 PIPELINE LLC

FOR BIDS	
FOR APPR	
FOR CONST	
DRAWN	HVT
CHECKED	JPD
APP'D	JPD

PORTLAND TERMINAL
PDX SEISMIC VULNERBILITY STUDY
TANK FARM #1
PIPE STRESS MODEL (T2784)
MULTNOMAH COUNTY, OREGON

SCALE: NONE
PROJECT NO: 25-124
FILE NAME (aka) DOCUMENT NUMBER: 25124-TF1-01



PIPE STRESS MODEL (TK 3407)

LEGEND:

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NO.	REVISION	BY / CHKD	DATE / APP'D

PHILLIPS 66 PIPELINE LLC

FOR BIDS
FOR APPR
FOR CONST
DRAWN HVT
CHECKED JPD
APP'D JPD

ALPHA TECHNICAL GROUP INC.
2929 NW 29th AVE, PORTLAND, OR. 97210
Tel:(503)227-3317/Fax:(503)227-3244

ATGI PROJECT-NUMBER 25-124

PORTLAND TERMINAL
PDX SEISMIC VULNERBILITY STUDY
TANK FARM #2
PIPE STRESS MODEL (T3407)
MULTNOMAH COUNTY, OREGON

SCALE NONE
PROJECT NO 25-124
FILE NAME (aka) DOCUMENT NUMBER
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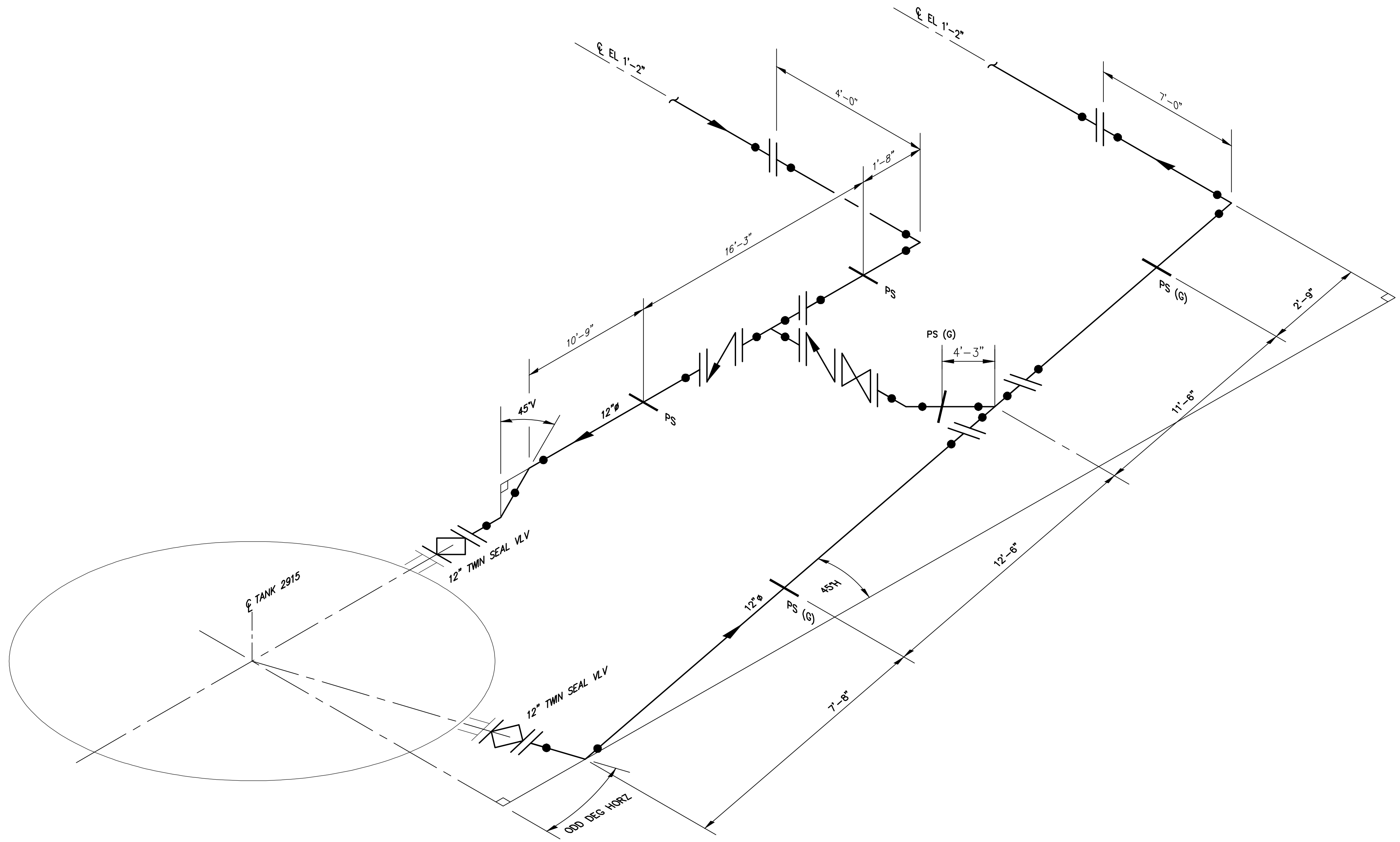


PIPE STRESS MODEL (TK 2915)

LEGEND:

1. "G" INDICATES PIPE GUIDE

ALPHA TECHNICAL GROUP INC.
2929 NW 29th AVE, PORTLAND, OR. 97210
Tel:(503)227-3317/Fax:(503)227-3244

ATGI PROJECT-NUMBER: 25-124

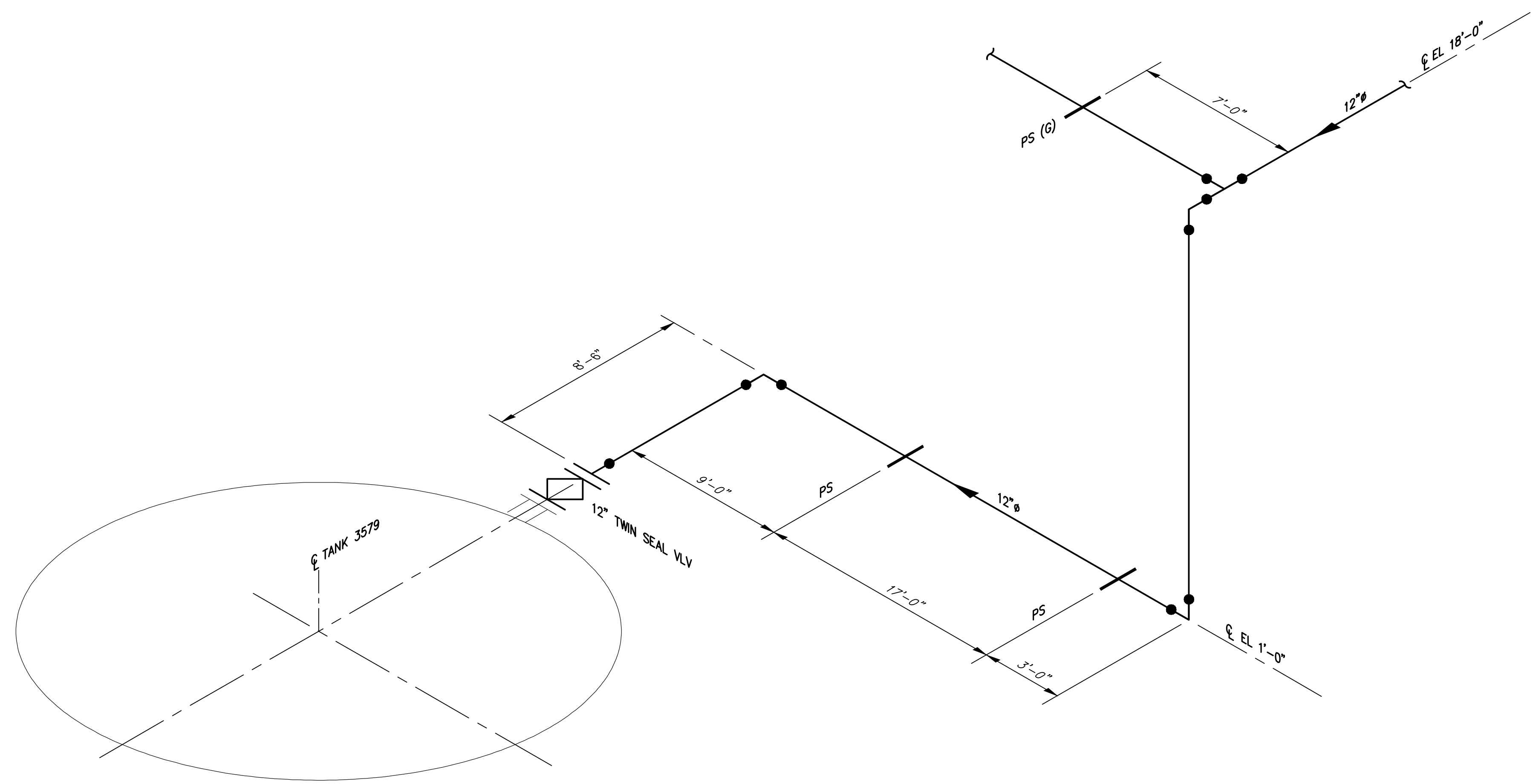
NO.	REVISION	BY / CHKD	DATE / APP'D

PHILLIPS 66 PIPELINE LLC

FOR BIDS
FOR APPR
FOR CONST
DRAWN HVT
CHECKED JPD
APP'D JPD

PORTLAND TERMINAL
PDX SEISMIC VULNERBILITY STUDY
TANK FARM #2
PIPE STRESS MODEL (T2915)
MULTNOMAH COUNTY, OREGON

SCALE NONE
PROJECT NO 25-124
FILE NAME (aka) DOCUMENT NUMBER
25124-TF2-02



PIPE STRESS MODEL (TK 3579)

LEGEND:

1. "G" INDICATES PIPE GUIDE

ALPHA TECHNICAL GROUP INC.
2929 NW 29th AVE, PORTLAND, OR. 97210
Tel:(503)227-3317/Fax:(503)227-3244

ATGI PROJECT NUMBER 25-124

NO.	REVISION	BY / CHKD	DATE / APP'D

PHILLIPS 66 PIPELINE LLC

FOR BIDS
FOR APPR
FOR CONST
DRAWN HVT
CHECKED JPD
APP'D JPD

PORTLAND TERMINAL
PDX SEISMIC VULNERBILITY STUDY
TANK FARM #3
PIPE STRESS MODEL (T3579)
MULTNOMAH COUNTY, OREGON

SCALE NONE
PROJECT NO 25-124
FILE NAME (aka) DOCUMENT NUMBER
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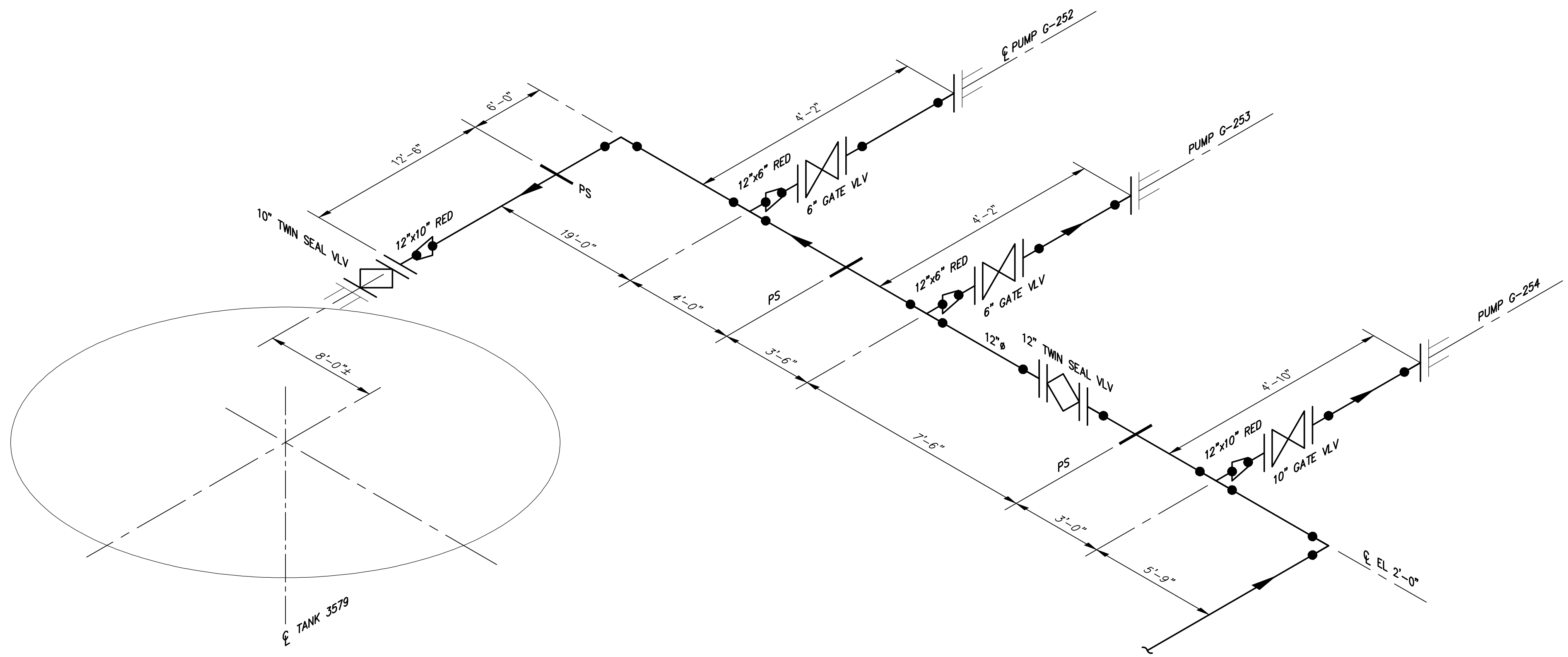

PIPE STRESS MODEL (TK 3579)

LEGEND:

1. "G" INDICATES PIPE GUIDE

NO.	REVISION	BY / CHKD	DATE / APP'D

ALPHA TECHNICAL GROUP INC.
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Tel:(503)227-3317/Fax:(503)227-3244

ATGI PROJECT NUMBER 25-124

PHILLIPS 66 PIPELINE LLC

FOR BIDS
FOR APPR
FOR CONST
DRAWN HVT
CHECKED JPD
APP'D JPD

PORTLAND TERMINAL
PDX SEISMIC VULNERBILITY STUDY
TANK FARM #3
PIPE STRESS MODEL (T3579)
MULTNOMAH COUNTY, OREGON

SCALE NONE
PROJECT NO 25-124
FILE NAME (aka) DOCUMENT NUMBER
25124-TF3-02

Appendix D
Dock

ALPHA TECHNICAL GROUP INC.
Consulting Engineers
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Phone (503) 227-3317   Fax (503) 227-3244

P66 Pipeline LLC- Portland Terminal
Seismic Vulnerability Assessment (SVA)
Marine Dock Facility

Rev Date: May 10, 2024

REGISTERED PROFESSIONAL
ENGINEER
15,471
OREGON
JULY 25, 1991
JOHN DEPPA
EXPIRES 12/31/25

PRELIMINARY

Table of Contents

1.0 Introduction & Scope

This report summarizes the results of a Seismic Vulnerability Assessment (SVA) of the P66 Pipeline's Portland Terminal's Marine Dock Facility. This assessment is intended to directly address the requirements outlined in the recently adopted Oregon DEQ "Fuel Tank Seismic Stability Rules" OAR 300. Alpha Technical Group Inc (ATGI) has performed many past inspections/analysis/studies to evaluate dock structural capacity for both gravity and lateral loading (including seismic). Repairs were designed and constructed in 2007 and 2012 and Building Permits were obtained from the City of Portland. A copy of the most recent 2021 Dock Inspection Report and drawings is included in the appendix.

2.0 Dock Structure & Operations Overview

The Marine Dock structure was originally constructed in approximately 1961. It is supported on pile supported bents at approximately 17.5 feet on center. Each support bent has batter piles and bracing to provide resistance to lateral loads due vessel berthing, mooring, and seismic. Most of the structural members are wood construction and these are typically replaced with steel when any new repairs are made.

3.0 Seismic Risk Assessment & Methodology

The product piping extends from shore to loading/unloading manifold near center of the dock as shown in Figure 1. Therefore, the area of the dock area from shore to the product manifolds has the highest risk of potential spills and this is what will be used for the assessment.

This assessment will evaluate the Marine Dock Structure during the design earthquake event. This assessment will be completed after further geotechnical investigations by Geo-Engineers Inc.-HOLD.

4.0 Assessment Findings-HOLD

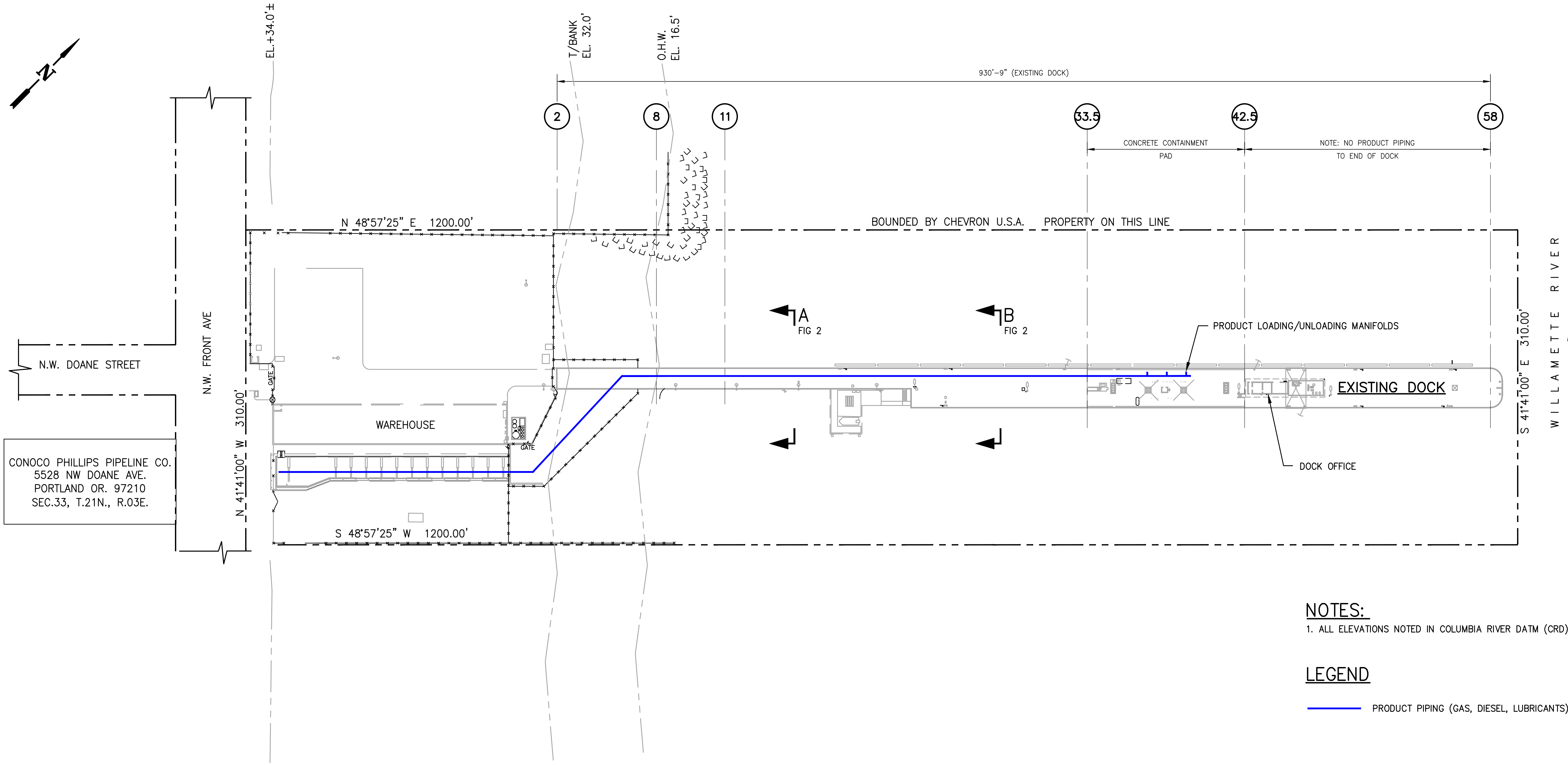
We look forward to discussing this report with you in more detail and hopefully can answer any questions you may have. If you have any other questions or require additional information, please feel free to contact me at your convenience.

Sincerely,

John Deppa, P.E., S.E.
Principal Engineer
.

Appendix 1

- Dock Site Plan (Figure 1)
- Dock Cross sections (Figure 2)



NOTES:

1. ALL ELEVATIONS NOTED IN COLUMBIA RIVER DATM (CRD)

LEGEND

PRODUCT PIPING (GAS, DIESEL, LUBRICANTS)

DOCK SITE PLAN

1"=50'-0"

25' 0' 50' 100'

SCALE: 1"=50' (22x34)
SCALE: 1"=100' (11x17)

ALPHA TECHNICAL GROUP INC.
2929 NW 29th AVE, PORTLAND, OR. 97210
Tel:(503)227-3317/Fax:(503)227-3244

ATGI PROJECT NUMBER 25-124

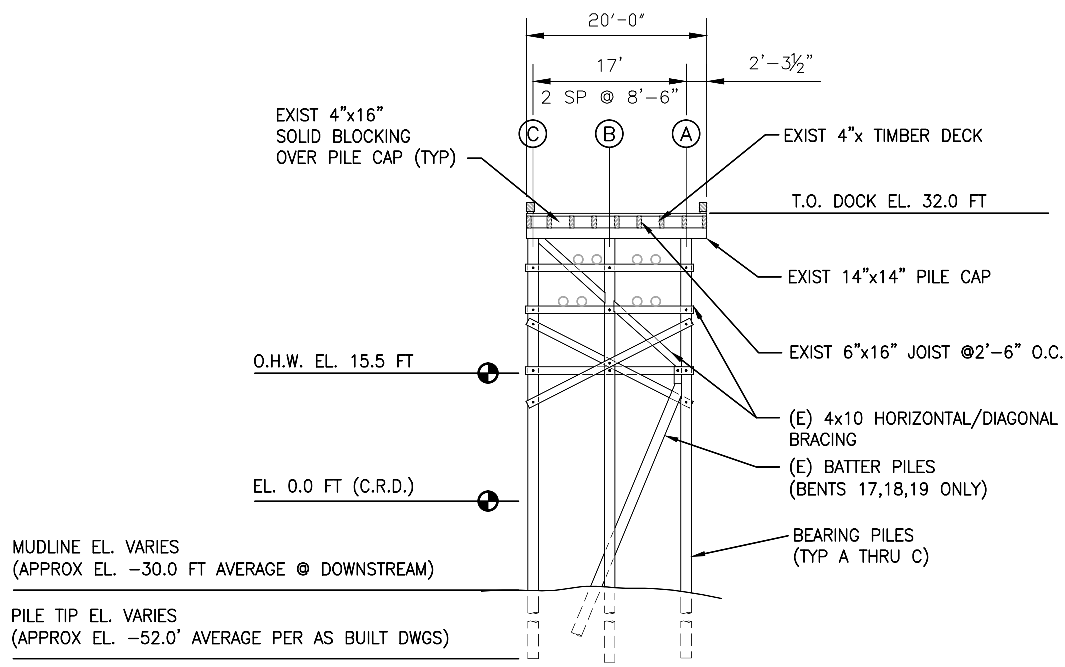
NO.	REVISION	BY / CHKD	DATE / APP'D

PHILLIPS 66 PIPELINE LLC

FOR BIDS
FOR APPR
FOR CONST
DRAWN HVT
CHECKED
APP'D

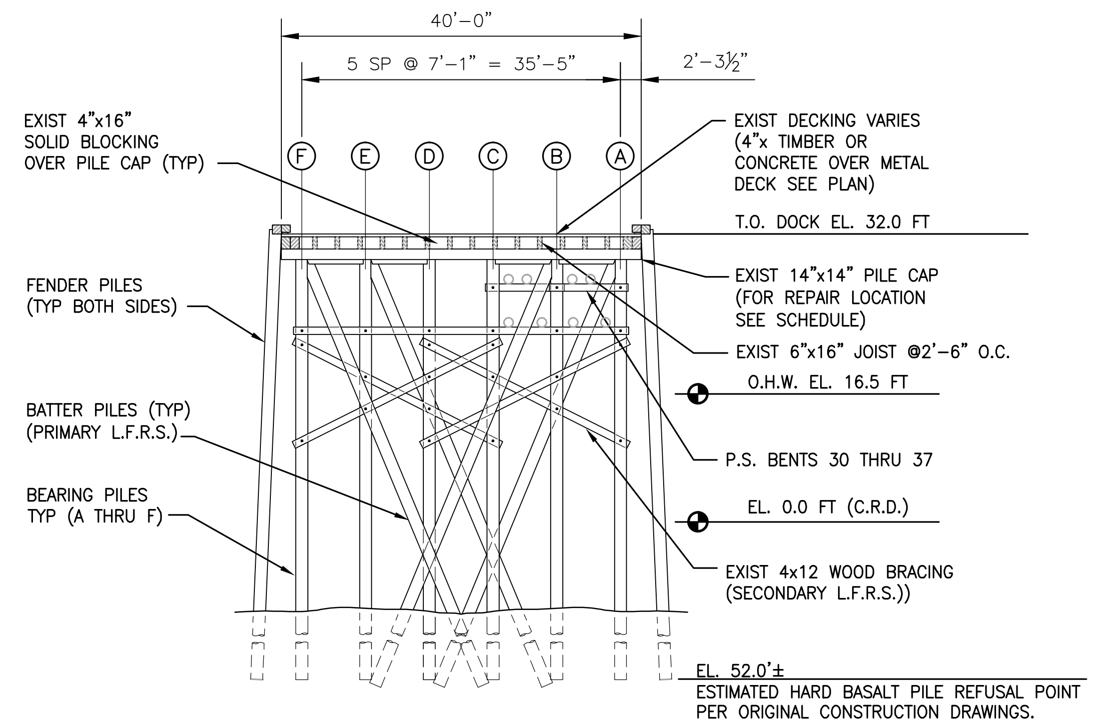
PORTLAND TERMINAL
DOCK
SITE PLAN

SCALE AS NOTED
PROJECT NO 25-124
FILE NAME (aka) DOCUMENT NUMBER
FIGURE 1



BENTS 3 THRU 22 (TYPICAL EXISTING)

SECTION A
3/32"=1'-0"
FIG 1



BENTS 23 THRU 58 (TYPICAL EXISTING)

SECTION B
3/32"=1'-0"
FIG 1

NOTES:

1. ALL ELEVATIONS NOTED IN COLUMBIA RIVER DATM (CRD)

5' 0' 10' 20'
SCALE: 3/32"=1'-0" (22x34)
SCALE: 3/64"=1'-0" (11x17)

ALPHA TECHNICAL GROUP INC.
2929 NW 29th AVE, PORTLAND, OR. 97210
Tel:(503)227-3317/Fax:(503)227-3244

ATGI PROJECT NUMBER 25-124

NO.	REVISION	BY / CHKD	DATE / APP'D

PHILLIPS 66 PIPELINE LLC

FOR BIDS
FOR APPR
FOR CONST
DRAWN HVT
CHECKED
APP'D

PORTLAND TERMINAL
DOCK
CROSS SECTIONS

SCALE AS NOTED
PROJECT NO 25-124
FILE NAME (aka) DOCUMENT NUMBER
FIGURE 2

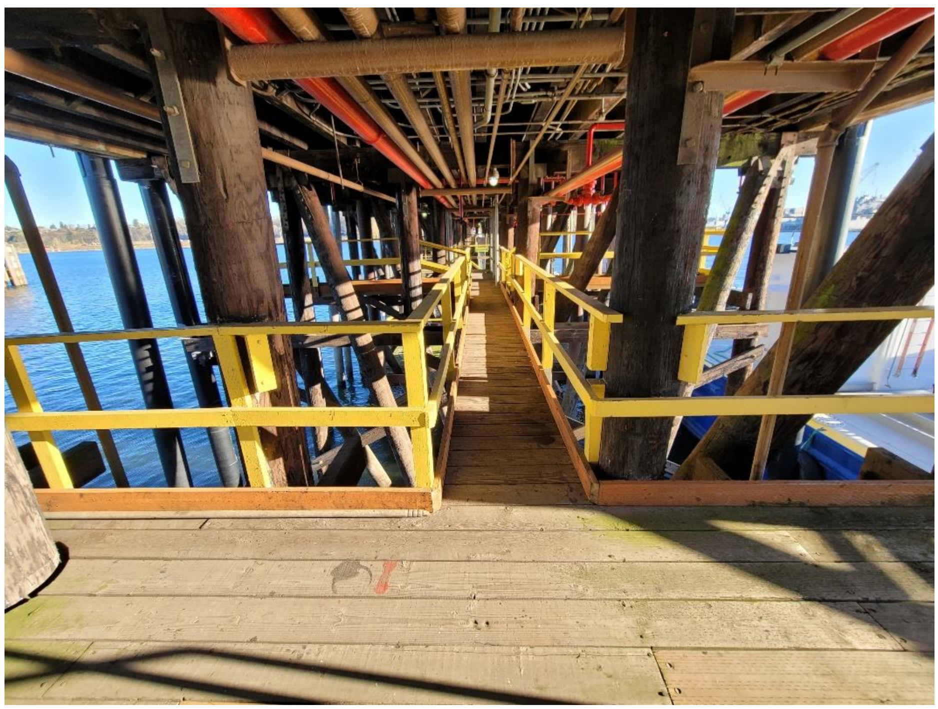
Appendix 2

- Dock Inspection Report and Drawings

Norwest Engineering Inc.
4110 NE 122ND Ave, Suite 207
Portland, Oregon 97230
Phone: (503) 254-0110
www.norwestengineering.com

Norwest Engineering
Consulting Engineers

WILLAMETTE RIVER TERMINAL: MARINE FACILITY STRUCTURAL CONDITION ASSESSMENT



NE Project #: RV1192
Client: Phillips 66
Location: 5528 NW Doane Ave., Portland, OR 97210

1192_REP301

Inspection Report

Revision	Date	Description	Engineer	Reviewed By	Approved By
A	12/31/2020	Issued for client review	VVG	ARM	JES
B	10/04/2021	Issued for client use	VVG	ARM	JES

WILLAMETTE RIVER TERMINAL: MARINE FACILITY STRUCTURAL CONDITION ASSESSMENT

NORWEST ENGINEERING, INC.

Project Manager: James Stone
Project Engineer: Valeriy Gubchak, PE
Inspection Team Leader: Valeriy Gubchak, PE
Field Team: Valeriy Gubchak
Ken Walrod
James Stone

REGISTERED PROFESSIONAL ENGINEER
83999PE
OREGON
DECEMBER 17, 2014
VALERIY GUBCHAK
RENEWAL DATE: 06/30/23

10/04/21

Underwater Inspection Team:

Subsea Global Solutions (SGS)
Port Angeles, WA
Dive Operations Manager: Mark Horness

Phillips 66

Project Manager: Nick Giotta, Maintenance Manager, Western Region

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EXECUTIVE SUMMARY

Norwest Engineering, Inc. (Norwest) was commissioned by Phillips 66 to provide a structural condition assessment for the Marine Facility at the Willamette River Terminal in Portland, Oregon, and develop an inspection report summarizing our methods and findings. Norwest performed an above water inspection from the water by boat, and from the dock and on land to cover all in and above-water elements. Norwest sub-contracted Subsea Global Solutions (SGS) to perform the below-water inspection. The results are summarized in the following report and Appendices.

Overall, the dock is rated in FAIR condition. There is advanced deterioration of the fender system on the pier head and both sides of the pier riverside of Grid 47. We recommend repairs there be carried out with moderate urgency. The floating deck and access to it should receive immediate attention to restore to serviceable condition.

Table 1: Overall ratings for the Marine Facility

Dock Grade Overall	**FAIR**
Area	**Grade**
Pier	FAIR
Approach	SATISFACTORY
Fender Piles	FAIR
Pier Head Protection	POOR
Walkways	SATISFACTORY
Floating Deck	POOR
Boathouse and Access	SATISFACTORY
Mooring Hardware	SATISFACTORY
Boathouse	GOOD
Boathouse Access	GOOD

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1 INTRODUCTION

Norwest Engineering, Inc. (Norwest) was commissioned by Phillips 66 to provide a structural condition assessment for the Marine Facility at the Willamette River Terminal in Portland, Oregon. This report provides a detailed description of the marine facility structures, their current condition, conclusions and recommendations for repair and future maintenance. Figures illustrating the location, marine facility structures, and piling plans are provided in Appendix A. Photographs are provided in Appendix B. ASCE Guidelines used for rating criteria and definitions are provided in Appendix C. The underwater inspection report is provided in Appendix D. Field inspection logs are provided in Appendix E, and level III test results are provided in Appendix F.

1.1 Project Personnel

Norwest's field team consisted of three engineers, including the licensed Engineer acting as team leader. The above-water inspection was performed over portions of 5 days spanning from October 8 thru December 2, 2020. Underwater inspection was carried out by Subsea Global Solutions (SGS) over 5 days from October 26 thru 30 under Norwest's direction and supervision.

1.2 Project Methodology

The inspection was carried out following guidance from ASCE 130, *Waterfront Facilities Inspection and Assessment*. The scope included visual inspection of 100% of the marine structures above and below water. The inspection focused on characterizing evidence of wear, damage, and corrosion of concrete and steel elements, and possible section loss, rot, abrasion, and/or overstressing of timber elements. Similar characterizations of fendering elements were conducted. Norwest inspected the in-water elements from deck level and from water level. Onshore elements were inspected on foot and from the boat when necessary, walking dry land when river levels were favorable. All inspection was conducted by and under the on-site supervision of the licensed Engineer.

This routine inspection followed a previous inspection performed in 2010. Based on findings in 2010 and subsequent maintenance reported, it was Norwest's judgement that a more rigorous scope including special aerial access was not warranted at this interval.

Visual inspection of timber elements was supplemented by hammer soundings. Soundings were performed on all piles at water level. Pile caps and stringers were inspected visually. Resistograph testing of timber piles was implemented as necessary where questionable elements were encountered and accessible by boat to further qualify their condition.

1.3 Limitations and Realistic Expectations

We are confident that our inspection routine, methodology, and professional approach to this project are consistent with the proposal and meet the standard of care. But we must caution

Phillips 66 that absolute certainty in locating decay and distress and assessment of load capacity is not possible and access restrictions for the inspectors also hindered the ability to expand the arms-length inspection zone.

Wood, the prevalent material in this facility and primary focus of the inspection and evaluation, is a natural biological building material with good strength and durability characteristics when properly maintained. But wood is the most variable of all the construction materials. Natural and environmental characteristics affect its strength and effectiveness of treatment. Specific gravity has the most effect but grain direction, knots, shake, reaction wood, and damage done by insects, and decay fungi also affect strength and ability to adsorb treatment. Hammer sounding, the primary inspection technique, is susceptible to misinterpretation because wet wood can sound like incipient and intermediate decay. The grading process for new materials, which uses trained inspectors, attempts to smooth out the variability. But grading is a human judgment subject to errors and materials of varying and substandard strength and characteristics do get incorporated into structures. These factors, combined with the size, scope, and complexity of the facility make it possible that some deficiencies could remain hidden.

2 FACILITY DESCRIPTION

The Willamette River Terminal is located across Front Avenue from the terminal at 5528 NW Doane Avenue in Portland, Oregon. Refer to Appendix A, Figures 1 and 2 for a site location map and aerial view. The marine facility is located on the Willamette River at river mile 7.9 upstream of the Columbia River. The marine facility serves the terminal fuel and lubricants businesses (loading and off-loading).

2.1 Marine Facility Overview

The marine facility consists of a pier and approach. There are no separate mooring or breasting dolphins. All bents on the pier beyond the approach include battered piles off-angle from the pile caps for multi-directional stability. The pier is lined by fender piles on both sides, as the facility continues to berth vessels on both sides. A separate floating boathouse structure is accessed by an aluminum gangway off the pier, and a floating platform between two pier bents is accessed by the substructure walkway. There is an extensive substructure walkway that provides mechanical access as well as multi-level vessel access. Piping from land to the service area is routed below the deck level.

The structure is primarily timber typical of 1960s and 1970s-era dock construction, with timber structural and fender piles, pile caps, stringers, bracing, and decking. Timber bracing (cross-bracing and struts) is thru-bolted to piles and pile caps. Some steel piles and framing are present, installed as replacement piles. Several pile bents were retrofit with steel piles and framing in a previous repair effort. Typically, replacement steel members are connected to each other by weld with and without gusset plates vs. bolting. Steel to timber connections are bolted.

The following sections describe the marine structures in some detail.

2.2 Area Descriptions

2.2.1 Pier and Approach

Refer to Figures 3 and 7 for a general dock and pile layout. Much of the approach is over dry land at normal water levels. The approach is narrower and supported by square-cut timber piles. The bents are braced transversely with timber bracing. The piles change to round cross section and batter piles are introduced where the first bollards are located. The main pier is wider, and each bent is supported longitudinally and transversely by off-angle batter piles. The pier head is protected from impact by multiple timber cluster piles and an arrangement of smaller diameter timber batter piles. Cross bracing is typically timber with some steel bracing and beams where original elements have been replaced.

Major loads supported by the pier include vessel breasting and mooring loads and piping. The deck is rated for vehicle access. Access to the vessels is via the substructure walkways, staircases/platforms, and retractable ladders. Access to the floating platform between bents 46 and 47 is from this walkway system. Access to the boathouse is off the southwest corner of the main pier via aluminum gangway.

2.2.1.1. Structural Piles

The approach and pier are supported by timber piles. There are square-cut timber piles supporting the approach from the abutment to Grid 11, each bent consisting of three plumb piles. From Grid 12-22, each bent consists of three round plumb piles, with bents 17-19 also having a single batter pile along the north side (Grid line A). Grid 23 thru 57 generally consist of six plumb and four batter piles at each bent. The pier head between Grids 57 and 58 has a more complex layout of smaller plumb and batter piles. Some bents include steel pipe piles where past pile replacements have occurred. Bents at Grids 39 and 41 consist of a steel pile and pile cap frame.

2.2.1.2. Decking, Bullrails, and Handrails

The decking consists of rough-cut timber planks laid transversely from Grids 1-16, and diagonally thereafter. Bullrails are present along both upstream and downstream sides of the dock and along the approach. They are elevated off the deck by 2-inch blocking at regular intervals to allow water to sheet off the deck. Handrails exist along the approach on the upstream side to the staircase to the boathouse, and along the downstream approach to Bollard 'B'. They are constructed of steel pipe and are painted. They are bolted to the top side of the bullrails. The secondary containment in the service area is concrete whose surface is sloped to closed system drainage points.

2.2.2 Fender Systems

The original design appears to have included two timber fender piles at each bent, on each side, from Grids 23-56. Some of the timber pile pairs remain. Other areas have only one of the two remaining. Still more areas have a single steel fender pile centered on the bent. It does not appear that fender piles have existed previously west of Grid 23, and there are none present today.

Timber cluster piles are arranged on either side of the tapered pier head beyond Grid 57. There are smaller diameter timber fender piles along the face of the pier head as well.

2.2.3 Substructure Walkways

There is an extensive walkway structure below-deck that accesses the below-deck piping, vessel/barge access points, and a floating deck. It is accessed by two main staircases that are accessed from deck openings at either end of the secondary containment. The main section extends from Grid 33.5 east to Grid 46, where a multi-level staircase and landing system accesses the floating deck between Grids 46 and 47. There is one major east-west corridor and numerous lateral walkways and platforms to fixed ladders, retractable gangways, and swing gates. The walkway system is generally of timber construction including framing, decking, stairs, and handrails.

The floating deck between Grids 46 and 47 is of timber construction, with timber decking and bullrails, timber framing, and floats. It is guided by collars around plumb piles 47A and 47F. It has four short metal pillars with flat, square tops that act as braces against the underside of the deck should the river level rise to unsafe levels for personnel. A fixed staircase is located on the shoreside of the deck, which accesses a short ladder back to the walkways and platforms.

A walkway system exists east (riverside) of Grid 47, accessed from a ladder at the floating deck, and through a trap door on the pier head. The walkway there is rarely used, access is limited, and needs upgrading for safety. It was not included in this evaluation.

2.2.4 Mooring Hardware

There are (4) double-bitt bollards arranged along the pier edge on the upstream side. There are (7) double-bitt bollards and (1) single-bitt bollard arranged along the pier edge on the downstream side. These are major mooring points for berthed vessels. One single-bitt bollard on Grid 25, and (1) small T-head bollard on Grid 42.5, are located in-board on the dock and likely are not used for vessel mooring. The bollards are labeled in the field, see Figures 7-10 for locations and naming convention. All bollards are bolted to the deck stringers, with nuts exposed on the deck side. Bollard load limits are not labeled.

2.2.5 Boathouse and Access

The boathouse is accessed by a timber staircase and platform and aluminum gangway off the main pier. The stairs and platform are timber framing and decking, supported by timber stringers between structural piles on Grid 23 and two dedicated steel piles and steel pile cap on Grid 22.5. The aluminum gangway extends parallel to the pier from the platform to a floating concrete deck. The gangway is fixed at the platform above and guided on rails and rollers on the floating dock. The stairs area self-leveling. The concrete deck supports the gangway landing and a small boat launch for a small aluminum service boat. The boathouse is a pole-barn type structure, with a roof and columns with no walls. The concrete deck is U-shaped, open on the riverside, and set on floats.

The two floating structures are independent but appear connected to remain within close proximity both laterally and vertically. They are guided by (5) composite piles. The three upstream piles are bare steel, while the three downstream piles are concrete and coated for abrasion resistance. The guides are constructed of galvanized steel with UHMW pads interfacing with the piles.

3 PREVIOUS INSPECTIONS

Norwest was provided one previous inspection document completed by Worley Parsons in 2010. The inspection included above water evaluations of structures. No underwater inspection was performed at that time. It is unclear when the previous underwater inspection was undertaken prior to 2020. The report was reviewed prior to completing the inspection process. A comparison of findings and recommendations from the prior and current inspections are provided in the Section 4.0 tables, and relevant discussions are provided in 5.0 in terms of Norwest's conclusions and recommendations.

3.1 2010 Above Water Inspection

Key findings and recommendations contained in the 2010 report are summarized as follows:

1. A significant number of timber stringers were found with significant section loss and decay and recommended for replacement.
2. A significant number of bullrails were at the end of their service life and recommended for replacement.
3. Multiple pile caps were found with significant section loss and recommended for replacement.
4. A total of 66 timber structural piles and 29 fender piles were recommended for replacement. The sections of wale and piles comprising the pier head protection system were recommended for replacement.
5. A fairly large section of decking was recommended for replacement.
6. Miscellaneous repairs to bracing, floating deck, and safety ladders were also recommended.
7. The substructure walkways, staircases, and access hardware were not mentioned in the report, except for fixed ladders on the pier faces.
8. The mooring hardware was not inspected.
9. The boathouse and access structures were not inspected.
10. No recommendation for inspection cycle was provided.

4 SUMMARY OF OBSERVED CONDITIONS AND RATINGS

Figures 3-13 in Appendix A illustrate elements and areas with ratings of Moderate damage or greater. A collection of photographs forming a visual record of the dock condition at present are provided in Appendix B. The observations summary and grading are provided by section below. A table with the overall rating and primary element ratings are provided in each section. Guidelines for general assessment ratings and visual reference for individual elements is provided in Appendix C. The underwater inspection report is provided in Appendix D. Inspection logs and Resistograph testing results are found in Appendices E and F.

The major deficiencies noted are associated with the fender system. Several fender piles are broken or missing. The pier head protection system is in a major state of disrepair and is not adequately protecting the dock from vessel strikes as intended. One pile cap needs to be replaced. A staircase section of the substructure walkways and the floating deck there will need immediate attention; access was barred by the Phillips 66 safety inspector after the inspection was performed. Two large sections of deck timbers are exhibiting significant wear and decay and need to be addressed.

4.1 Pier and Approach

Table 4-1: Pier and Approach Ratings

Area	Overall Rating	Element Damage Rating	Previous Issues	Change In Status From 2010
Pier	Fair			
Structural Piles		MN	Yes	Improved
Pile Caps and Stringers		MN	Yes	Improved
Bracing		MN	Minor	No Change
Decking and Bullrails		**MD-MJ**	**Yes**	**Worsening**
Concrete Secondary Containment		ND	No	No Change
Approach	Satisfactory			
Structural Piles		MN	No	No Change
Pile Caps and Stringers		MN	Yes	Improved
Bracing		MN	No	No Change
Decking and Bullrails		ND-MN	Yes	Improved
Handrails		ND	--	--

The pier structure is in FAIR condition overall. Some significant sections of decking are in poor condition and should be addressed in the near-term. One pile cap at Grid 47 is showing signs of overstress (see Appendix B, Picture 31). A few bullrails should be replaced in the near-term. All timber piles were graded with minor damage in the form of checking, vertical cracking, or minor to moderate hollow sounds. All steel piles were found to have minor surface corrosion. None are recommended for replacement at this time. Several of the findings in the 2010 report have been addressed, such as replacement structural piles, stringers, pile caps, and bullrails ("guardrails" in the 2010 report). We discovered fewer deficiencies overall than was reported in 2010, thus the improvement in condition noted above.

The approach structure is in SATISFACTORY condition overall. One stringer is recommended for replacement. A few piles were found to be in deteriorating condition and should be monitored regularly. None are recommended for replacement at this time. The bullrails and handrails are in good condition. The decking on the approach is in better condition than the pier.

4.2 Fender Systems

Table 4-2: Fender Piles and Pier Head Protection

Area	Overall Rating	Element Damage Rating	Previous Issues	Change In Status
Fender Piles	Fair			
Upstream Side		**MJ**	**Yes**	**Some replaced, additional in need**
Downstream Side		**MD**	**Yes**	**Some replaced, additional in need**
Pier Head Protection	POOR			
Cluster Piles		**SV**	**Yes**	**Worsening**
Batter and Plumb Piles		**MJ**	**Yes**	**Worsening**

The fender piles on both sides of the dock are in FAIR condition. Several piles are broken but standing, broken with half the pile still attached to the dock and the bottom half missing or below-water, or the upper half is missing where the bottom half is left in place. Others are missing altogether.

The pier head protection system is in SERIOUS condition overall. The cluster piles on the corners and single fender piles on the pier head face beyond Grid 58 are all broken at the water line. Several of the small-diameter batter piles are deteriorating.

4.3 Substructure Walkways

TABLE 4-3: SUBSTRUCTURE WALKWAY RATINGS

AREA	OVERALL RATING	ELEMENT DAMAGE RATING	PREVIOUS ISSUES	CHANGE IN STATUS
WALKWAYS (GRID 33.5-46)	SATISFACTORY			
STAIRCASES		ND	--	--
WALKWAYS AND PLATFORMS		**MN***	--	--
HANDRAILS		**MN***	--	--
LADDERS, GANGWAYS, GATES		MN	YES	NO CHANGE
FLOATING DECK	POOR			
TIMBER DECK AND FRAMING		MN	NO	NO CHANGE
FLOATS AND GUIDES		**MD**	**NO**	**WORSENING**
STAIR ACCESS		**SV**	**NO**	**WORSENING**

*Exceptions to the general rating are described below.

The substructure walkways are in SATISFACTORY condition overall. The timber framing, decking, staircases, and handrails show minor wear. A few deck boards at the end of a platform have significant decay and should be replaced, but they are relatively few. The paint on handrails and other points is in good condition. The retractable gangways were not inspected for function but exhibited no obvious damage, with one exception. Phillips 66 has flagged one for replacement. The fixed ladder on the upstream pier face is deformed slightly from vessel impact but is still functional.

The staircases and platforms on the east end, accessing the floating deck, are in POOR condition with moderate to major damage to structural framing from decay. Some signs of overstressing are evident. The handrail near the bottom of the staircase does not meet OSHA requirements for height, nor do they match from one side of the staircase to the other. The access to the floating deck is a fixed ladder to a fixed staircase on the deck that does not meet Phillips 66 standards for safety and access. This area was flagged by the Phillips 66 safety inspector and access to the deck via this system was barred from further use. It will need to be repaired or replaced immediately to restore access.

The floating deck structure is in POOR condition overall. The timber guides along the nearby piles are decaying and should be replaced. The deck itself is not level and lists shoreward, indicating the floats are deteriorating. The access issue described above will also need to be addressed.

4.4 Mooring Hardware

TABLE 4-4: MOORING HARDWARE RATINGS

AREA	OVERALL RATING	ELEMENT DAMAGE RATING	PREVIOUS ISSUES	CHANGE IN STATUS
BOLLARDS	SATISFACTORY			
UPSTREAM SIDE		MN	--	--
DOWNSTREAM SIDE		MN	--	--

The bollards are in SATISFACTORY condition overall. Some scale is building up on a few of them. One bollard has a moderate amount of scale and corrosion. Some have no wear at all. Some of the bolts are suspected to be loose, and we understand that has been a problem addressed in the past. Tightening all the bolts is a simple maintenance intervention. No damage or deterioration to the supporting structure beneath was noted at the bollard locations.

4.5 Boathouse and Access

TABLE 4-5: BOATHOUSE AND ACCESS RATINGS

AREA	OVERALL RATING	ELEMENT DAMAGE RATING	PREVIOUS ISSUES	CHANGE IN STATUS
BOATHOUSE STRUCTURES	GOOD			
BOATHOUSE		ND	--	--
LANDING		ND	--	--
GUIDE PILES		MN	--	--
ACCESS	GOOD			
STAIRCASE AND PLATFORM		ND	--	--
SUPPORT PILES		ND	--	--
GANGWAY		ND	--	--

The boathouse structures are in GOOD condition overall. The boathouse concrete deck, columns, and roof exhibit no damage. The landing and floating deck adjacent were recently replaced. The guide piles have minor damage from the guides rubbing them.

The access is in GOOD condition overall. No damage was noted on any of the elements. The gangway threshold plate and rollers appear in good working condition. The ramp steps and handrails have no apparent damage. The staircase and landing exhibit no apparent damage. All appear in working order.

4.6 Underwater Inspection

The full underwater inspection report is attached in Appendix D. Norwest supervised the inspection findings so that questions could be asked at the time divers were in the water. The inspection was carried out over four days in late October.

Please note that the grading of findings differs from the nomenclature used elsewhere in this report. In general, pile conditions were good for both steel and timber piles below the surface, except for the damaged or decaying piles that were also noted above water and marked in Norwest's inspection logs. No major discoveries were made during the inspection.

5 CONCLUSIONS AND RECOMMENDATIONS

Overall, the dock is rated in FAIR condition. There is advanced deterioration of the fender system on the pier head and both sides of the pier riverside of Grid 47. The load-bearing piles were found to be in fair condition, and many of the flagged piles from the 2010 report have been replaced with steel piles. However, the structure is at-risk of impact damage and overstressing while the fender system is in this condition. We recommend repairs be carried out with moderate urgency. A full discussion is found in Sections 5.1.2. The recommendations presented in Section 5.1 are illustrated in Figures 3 thru 13 in Appendix A.

5.1 Repair Recommendations by Area

Table 5-1: Summary of Recommendations

Area/Issues	Timeframe to Address	Recommendations
Pier and Approach		
Decking, Bullrails	2-5 yrs	Replace deck planks, either individually or whole areas; replace marked bullrails
Approach Stringer, Grid 6C-7C	2-5 yrs	Replace stringer
Approach Structural Piles	5-7 yrs	Monitor piles quarterly for further deterioration and expect to replace within 5-7 years
Pile Cap, Grid 47	1-2 yrs	Replace pile cap, add a plumb pile at mid-span
Fender Systems		
Upstream Side	1-2 yrs	Remove broken timber piles, install replacements
Downstream Side	1-2 yrs	Remove broken timber piles, install replacements
Pier Head	1-2 yrs	Either replace in-kind or design and install new protection system
Substructure Walkways		
Staircase/Platform to Floating Deck	3-6 months	Replace access to floating dock
Floating Deck	1-2 yrs	Replace pile guides, address deck listing issue

TABLE 5-1: SUMMARY OF RECOMMENDATIONS (CONT'D)

AREA/ISSUES	TIMEFRAME TO ADDRESS	RECOMMENDATIONS
MOORING HARDWARE		
ALL BOLLARDS	6 MONTHS	Tighten all bolt connections, replace bolts as necessary
BOLLARD 'G'	2-5 YRS	Remove rust and scale, recoat

5.1.1 Pier and Approach

The deck planks in both areas marked in the figures are in advancing states of repair. In both areas, multiple boards have obvious advanced decay and retain little bearing capacity. All boards in both areas show signs of ultraviolet damage and fungal decay, and the planks are visibly swollen. A contractor could evaluate the planks for individual replacement or Phillips 66 could elect to replace all planks in these areas to avoid regular replacement projects. The bullrails marked for replacement are at the end of their service life. They should be also replaced.

The pile cap at Grid 47 appears affected by previous changes to the structure. Plumb piles were cut near mid span and a steel beam placed beneath them. This system has proved inadequate and the beam is deformed. The pier bent should be evaluated to determine the best repair approach.

One stringer has a longitudinal crack that affects its load capacity. It should be replaced or repaired. A specialty contractor could be engaged to evaluate the possibility of gluing the cracks, given the appearance of the failure and the condition of timber, as an alternative to replacement. Four structural piles on the approach have large vertical checks. These piles should be monitored quarterly for further deterioration, more frequently if vehicle traffic increases.

5.1.2 Fender Systems

The pier head protection system exhibits serious damage. All the timber fender piles are broken at the water line or missing. The small-diameter structural piles between Grids 57 and 58 are showing advancing deterioration. This system may have reached its end of service life. Vessel impacts obviously have happened and should be expected to continue. Phillips 66 may consider in-kind replacement or engineer a new system with modern construction. In either case, action should be given at least moderate urgency.

Fender piles serve two functions: Absorb impact energy from vessels, and protect the structural piles supporting the pier. A gap exists between bents that is expected to be sufficiently narrow to protect the substructure. Larger gaps exist where fender piles are missing or broken. There are fewer missing piles riverside of Grid 47. The fender piles should be replaced where either missing or broken. Nine bents are marked with severely damaged or missing fender piles between Grids 23 and 47. Three-quarters of the bents riverside of Grid 47 have either broken or missing fender piles.

5.1.3 Substructure Walkways

The floating deck access is structurally deficient, and the transition to the deck has been flagged by Phillips 66 safety. Norwest recommends action is taken immediately if access is required for operations.

The supporting beams are in poor shape and need to be replaced. One handrail is inadequately built and should be replaced. The stairs and platform decking are not in bad shape, though the new access to the floating deck may render much of the remaining structure unsuitable to the new design. The uppermost staircase may be suitable to remain, though constructing the new structure may be easier replacing the staircase/platform system up to the main walkway. Norwest recommends the staircase be engineered to consider the potential floating deck access options and ensure the system is built to meet OSHA requirements.

The floating deck guides should be replaced. They were flagged in the 2010 report and are continuing to deteriorate. The floats should also be addressed to arrest to re-level the deck.

5.1.4 Mooring Hardware

Phillips 66 personnel noted that bollard bolts have been found loose and re-tightened in the past. Norwest did not note any obviously loose bolts, but several were questionable without testing with a wrench. Norwest recommends maintenance tighten all bolts and check at least annually.

All bollards are in structurally sound condition. The coatings are in various states of wear, the worst being ‘G’. The scale buildup will require this bollard to be removed and recoated soon.

5.2 Maintenance Schedule Recommendations

Phillips 66 intends to include Norwest’s maintenance recommendations over a five-year period, with the stipulation that the annual spend be kept as equal as possible. Norwest made judgements based on previous construction projects to provide our recommendations for the division of work summarized in Table 5-2 below. 2. Norwest has recommended a front-end engineering design (FEED) study in Year 3 to determine the path forward for the pier head. Concepts could include repair or replacement, and we recommend future vessel capacity be considered. The head replacement could include new larger capacity mooring dolphins as part of the structure, such as the neighboring pier is utilizing.

It is difficult to break up the fender pile removal/replacement tasks. It could all be completed easily in one mobilization. However, the spend will not be very even without dividing this task in some way. Norwest recommends doing the following:

1. Replace piles in the main service areas that come into contact with vessels most frequently. Split up half of the work to accomplish this in Year 1.
2. Replace the remainder in Year 3, most of them near the pier head. Though these areas are vulnerable to structural damage from a vessel strike, there has been no strikes to speak of and no damage to the exposed plumb piles while we anticipate the fender piles have been nonexistent for some time.

We understand that Phillips 66 has not yet determined whether the floating deck between bents 46 and 47 is needed. As such, repair or replacement of the deck and the staircase to it are still under consideration. Norwest recommends addressing them in the near term:

1. If the deck will be repaired, we recommend that happen in the near-term.
2. If the deck is deemed of no further use, we recommend it be removed in a year where a contractor with a barge and the right equipment can remove it.
3. If the deck remains in use, then we recommend the staircase be repaired as soon as possible.
4. If the staircase is deemed non-essential, then we recommend access to it be blocked in a permanent way.

TABLE 5-2: SUMMARY OF MAINTENANCE SCHEDULE RECOMMENDATIONS

YEAR	RECOMMENDATIONS
YEAR 1	**1.** Tighten bollards connections.
	2. Service areas, both berths: Remove all broken fender piles hanging from the structure for safety purposes. Remove deadheads in the water where new piles will be installed. Drive new steel fender piles, ~half of total fender piles called out for replacement.
	3. Repair or replace the floating deck, grids 46-47.
	4. Repair staircase to floating deck.
YEAR 2	**1.** Replace decking and bullrails on the shore side of the service area.
YEAR 3	**1.** Remainder of pier, both berths: Remove remaining deadheads in the water. Drive new steel fender piles primarily near the pier head, ~half of total fender piles called out for replacement.
	2. Complete FEED study for the pier head.
YEAR 4	**1.** Repair or replace pile cap at grid 47. Norwest suggests an engineered solution.
	2. Repair or replace the stringer on grid 6C-7C on the pier approach.
	3. Sandblast and recoat bollards as necessary.
	4. Complete detailed engineering for repair or replacement of the pier head.
YEAR 5	**1.** Repair or replace the pier head.
	2. Replace decking and bullrails on the river side of the service area (pier head).

5.3 Inspection Cycle

The next routine inspection above water should occur within no more than five years from present, based on the material, environment, and current facility condition. An underwater investigation should occur within five to six years from present.

APPENDIX A:

FIGURES

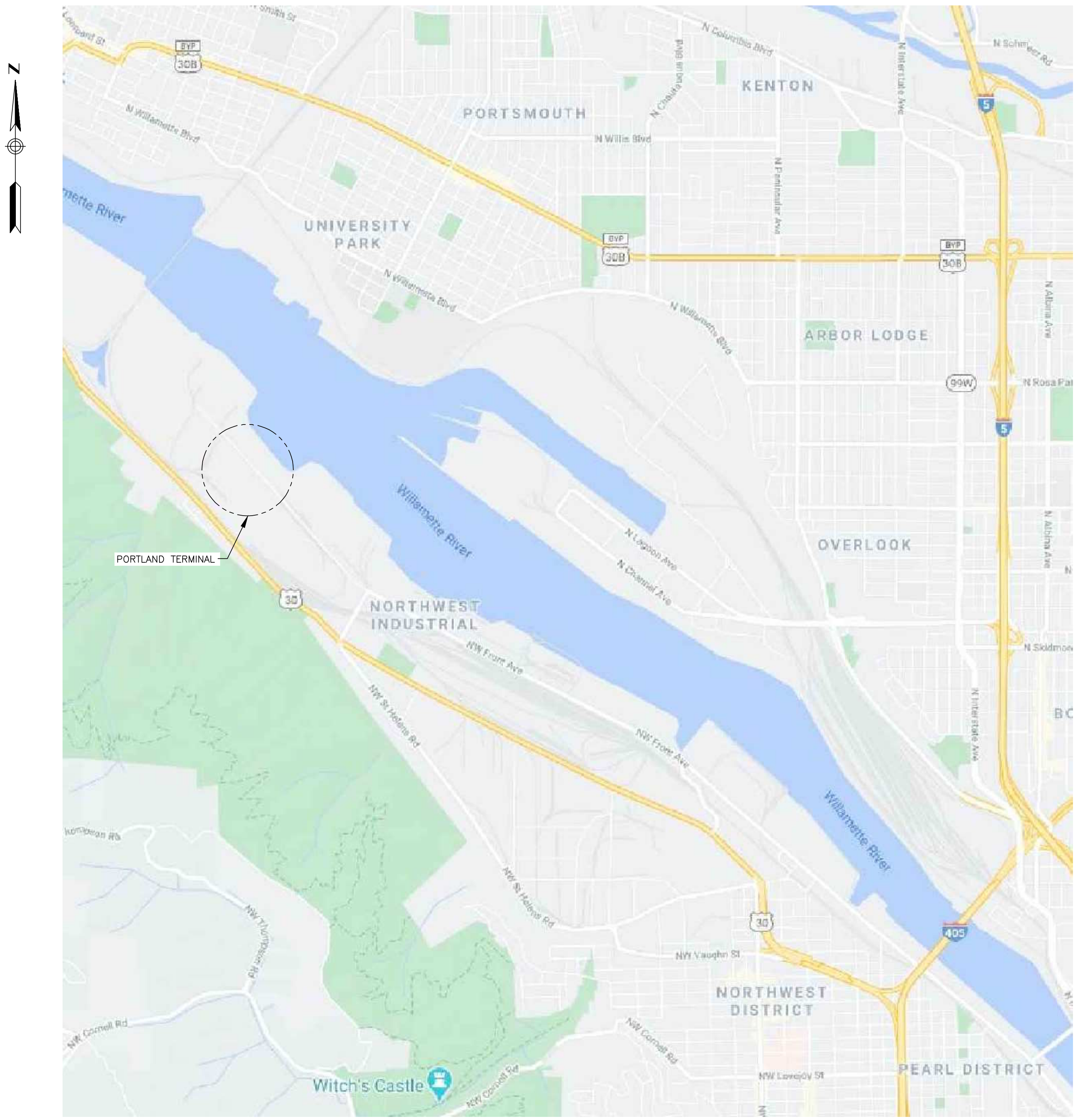

FIGURE 1
LOCATION MAP
SCALE: N.T.S.

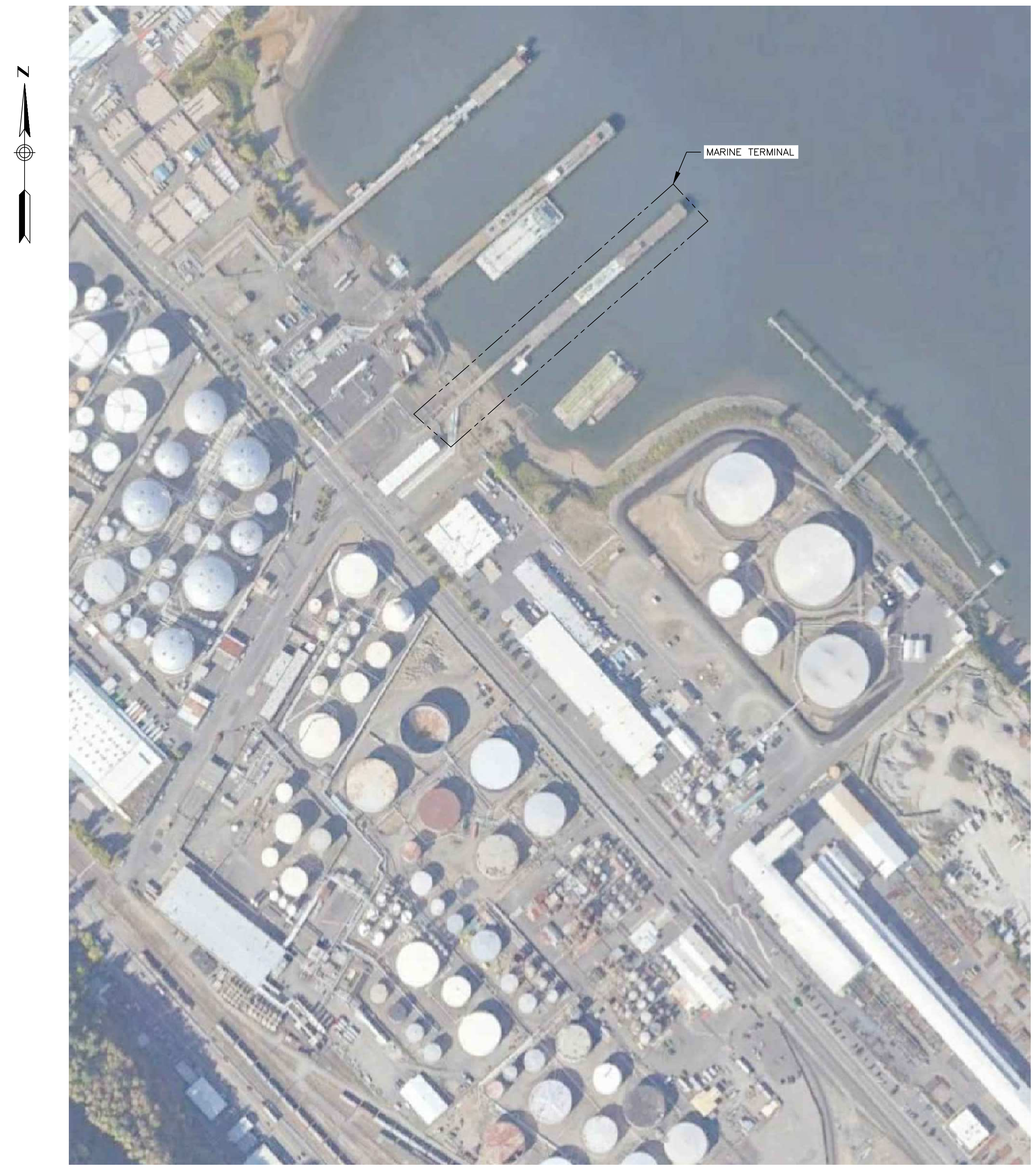

FIGURE 2
AERIAL VIEW
SCALE: N.T.S.

NO.	DESCRIPTION	BY	APPR.	DATE
A	ISSUED FOR REVIEW	RJV	VVG	12/31/20

REVISIONS

PHILLIPS 66

PORTLAND OREGON

MARINE FACILITY STRUCTURAL CONDITION ASSESSMENT
FIGURES 1 & 2 – LOCATION MAP & AERIAL VIEW
PORTLAND, OR

NORWEST ENGINEERING, INC.
CONSULTING ENGINEERS
PORTLAND, OREGON 503-254-0110
IRVINE, CALIFORNIA www.NorwestEngineering.com

N.E. # RV1192

CHKD. VVG	DRWN. RJV	PROJECT NO. RV1192
APPR. VVG	DATE 11/11/20	DWG. NO. D 1192-SKS001 REV. A
PLOT SCALE 1:1	SCALE NONE	

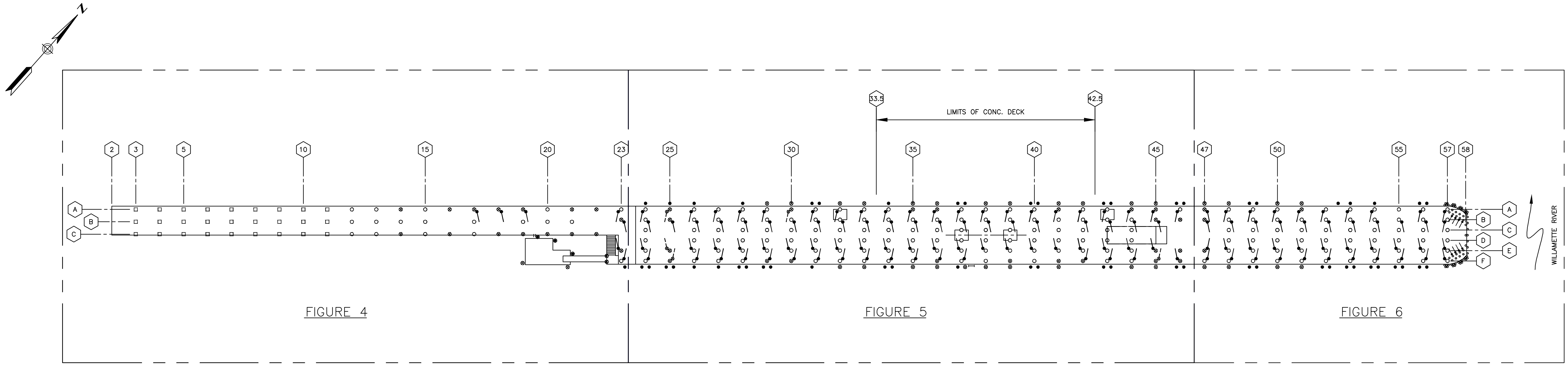

FIGURE 3: KEY PLAN

SCALE: 1/32"=1'-0"

NO.	DESCRIPTION	BY	APPR.	DATE
A	ISSUED FOR REVIEW	RJV	VVG	12/31/20

REVISIONS

PHILLIPS 66

PORTLAND OREGON

MARINE STRUCTURAL CONDITION ASSESSMENT
FIGURE 3: DOCK KEY PLAN

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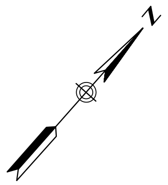
N.E. # RV1192

CHKD. VVG	DRWN. TKB	PROJECT NO. RV1192
APPR. VVG	DATE 10/06/20	DWG. NO. D 1192-SKS002
PLOT SCALE 1:1	SCALE AS NOTED	REV. A



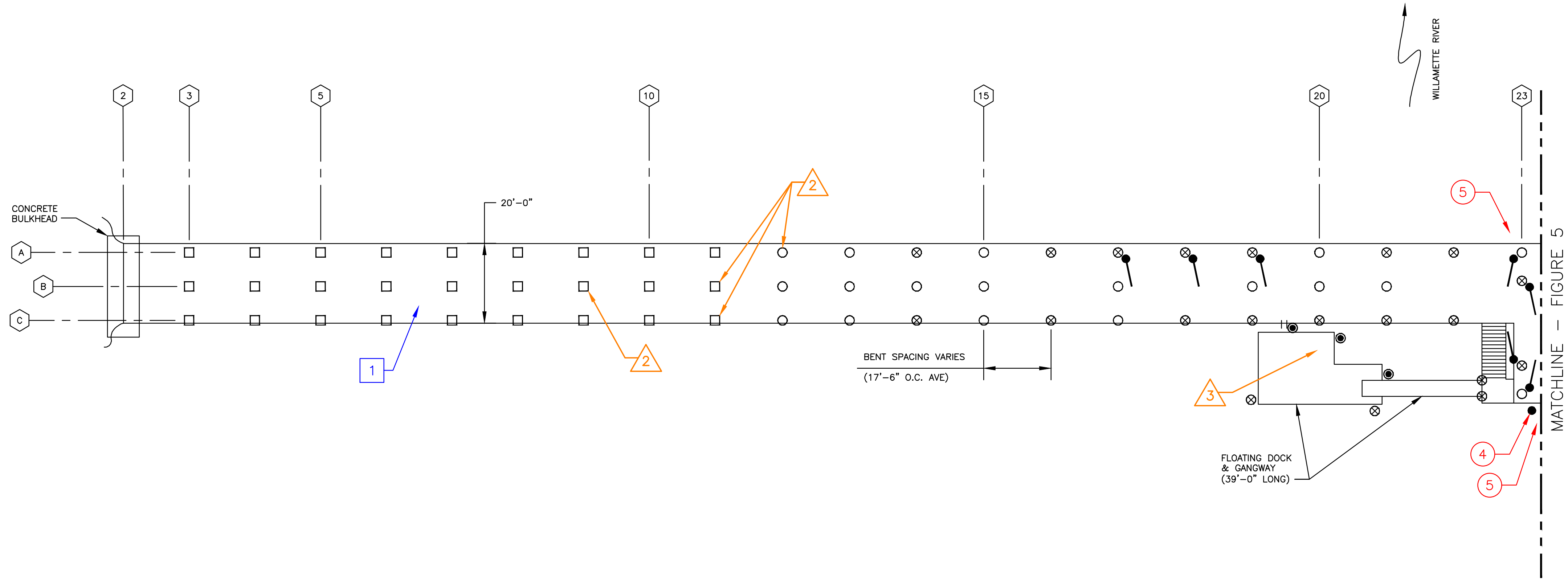


FIGURE 4: PLAN
SCALE: 1/16"=1'-0"

RATING ITEM #:

1. STRINGER (INNERMOST OF THE TWO EDGE STRINGERS) TO BE REPLACED.
2. FOUR TIMBER PILES WITH 1" SIGNIFICANT LONGITUDINAL SPLITS; MONITOR FOR DETERIORATING CONDITION.
3. REPAIR CONNECTIONS BETWEEN INDIVIDUAL CONCRETE SECTIONS OF GANGWAY LANDING TO PREVENT INDEPENDENT MOTION AND FURTHER LISTING.
4. REMOVE DEAD-HEAD PILE FROM DOCK, FROM WATER LINE IF PRESENT.
5. REPLACE MISSING FENDER PILE(S).

LEGEND:

- # – SV RATING
- # – MJ RATING
- # – MD RATING
- A 1 – PILE GRID CENTERLINE
- □ – SQUARE-CUT TIMBER PLUMB PILE
- ○ – TIMBER PLUMB PILE
- ● – TIMBER FENDER PILE
- ◉ – CONCRETE GUIDE PILE (COATED)
- ∘ – CLUSTERED TIMBER FENDER PILE
- ⊗ – STEEL BATTER PILE
- – TIMBER BATTER PILE
- – SMALL-DIAMETER TIMBER BATTER PILE
- ⊗ – STEEL PIPE PILE

NO.	DESCRIPTION	BY	APPR.	DATE
A	ISSUED FOR REVIEW	RJV	VWG	12/31/20

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MARINE STRUCTURAL CONDITION ASSESSMENT
FIGURE 4: FACILITY LAYOUT AND PILES

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N.E. # RV1192

CHKD. VWG	DRWN. TKB	PROJECT NO. RV1192
APPR. VWG	DATE 10/06/20	DWG. NO. D 1192-SKS003 / REV. A
PLOT SCALE 1:1	SCALE AS NOTED	

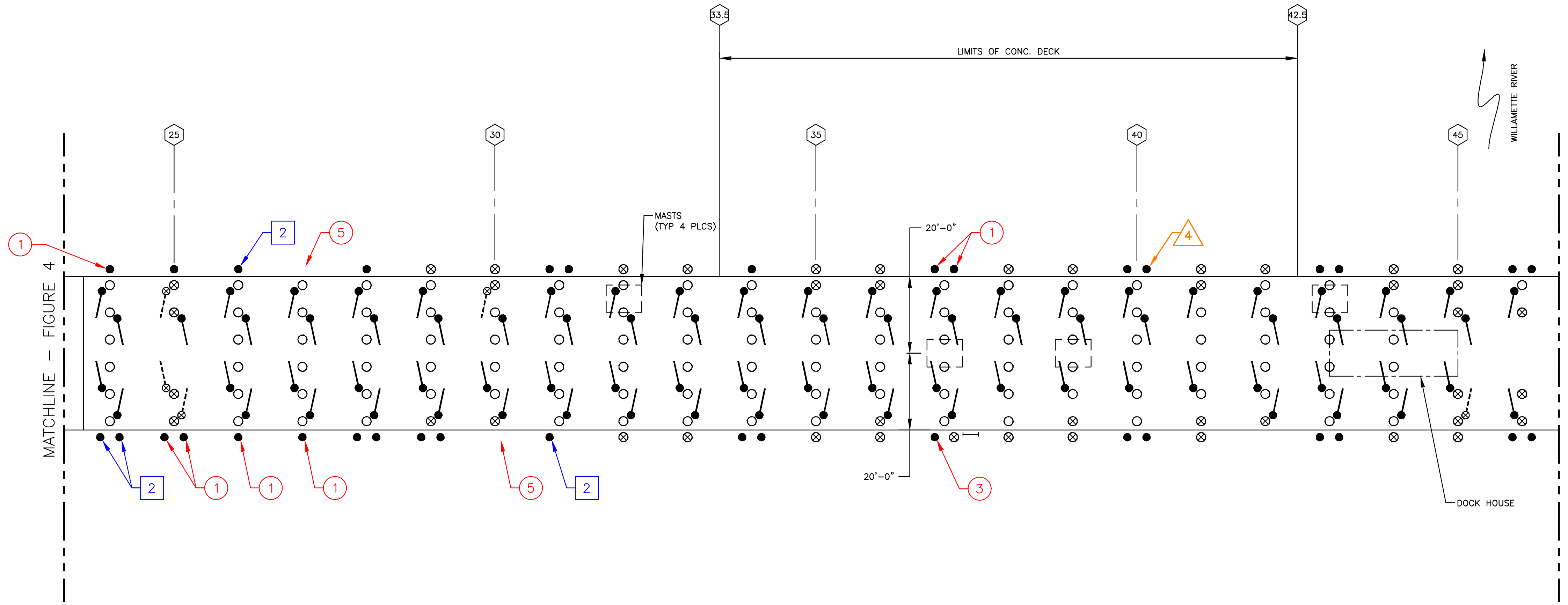

FIGURE 5: PLAN
SCALE: 1/16"=1'-0"

RATING ITEM #:

1. REMOVE BROKEN FENDER PILE(S) AND REPLACE PILE(S).
2. MONITOR FOR FURTHER DETERIORATION OR CONSIDER REPLACEMENT.
3. REMOVE BROKEN FENDER PILE.
4. REPLACE FENDER PILE.
5. REPLACE MISSING FENDER PILE(S).

LEGEND:

- # – SV RATING
- # – MJ RATING
- # – MD RATING
- A 1 – PILE GRID CENTERLINE
- □ – SQUARE–CUT TIMBER PLUMB PILE
- ○ – TIMBER PLUMB PILE
- ● – TIMBER FENDER PILE
- ∘ – CLUSTERED TIMBER FENDER PILE
- – STEEL BATTER PILE
- – TIMBER BATTER PILE
- – SMALL–DIAMETER TIMBER BATTER PILE
- ⊗ – STEEL PIPE PILE

NO.	DESCRIPTION	BY	APPR.	DATE
A	ISSUED FOR REVIEW	RJV	VWG	12/31/20

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MARINE STRUCTURAL CONDITION ASSESSMENT
FIGURE 5: FACILITY LAYOUT AND PILES

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CHKD. VWG	DRWN. TKB	PROJECT NO. RV1192	
APPR. VWG	DATE 10/06/20	DWG. NO. D 1192–SKS004	REV. A
PLOT SCALE 1:1	SCALE AS NOTED		

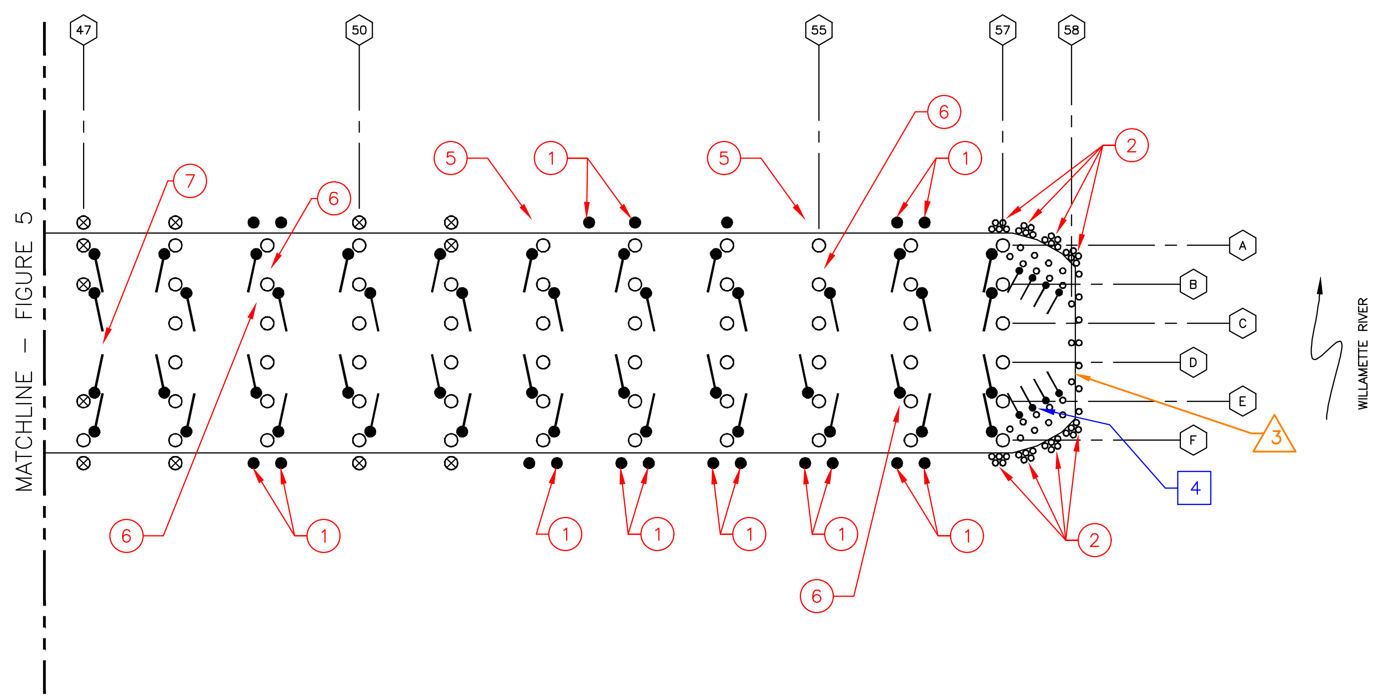

FIGURE 6: PLAN
SCALE: 1/16"=1'–0"

RATING ITEM #:

1. REMOVE BROKEN FENDER PILE(S) AND REPLACE PILE(S).
2. REMOVE BROKEN CLUSTER PILES AND REPLACE WITH NEW FENDERS, FENDER PILES, OR OTHER PIER HEAD PROTECTION SYSTEM.
3. REPLACE PILE CAP GRID 58.
4. 57E–58E SMALL DIAMETER PILES: CONSIDER AMENDING WITH NEW PIER HEAD PROTECTION SYSTEM.
5. REPLACE MISSING FENDER PILE(S).
6. BROKEN LATERAL BRACING MEMBERS.
7. REPLACE PILE CAP AT GRID 47. MAY BE NECESSARY TO ADD (1) PLUMB PILE FOR ADDITIONAL SUPPORT.

LEGEND:

- SV RATING
- MJ RATING
- MD RATING
- PILE GRID CENTERLINE
- SQUARE–CUT TIMBER PLUMB PILE
- TIMBER PLUMB PILE
- TIMBER FENDER PILE
- CLUSTERED TIMBER FENDER PILE
- STEEL BATTER PILE
- TIMBER BATTER PILE
- SMALL–DIAMETER TIMBER BATTER PILE
- STEEL PIPE PILE

NO.	DESCRIPTION	BY	APPR.	DATE
A	ISSUED FOR REVIEW	RJV	VVG	12/31/20

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MARINE STRUCTURAL CONDITION ASSESSMENT
FIGURE 6: FACILITY LAYOUT AND PILES

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N.E. # RV1192

CHKD. VVG	DRWN. TKB	PROJECT NO. RV1192
APPR. VVG	DATE 10/06/20	DWG. NO. D 1192–SKS005
PLOT SCALE 1:1	SCALE AS NOTED	REV. A

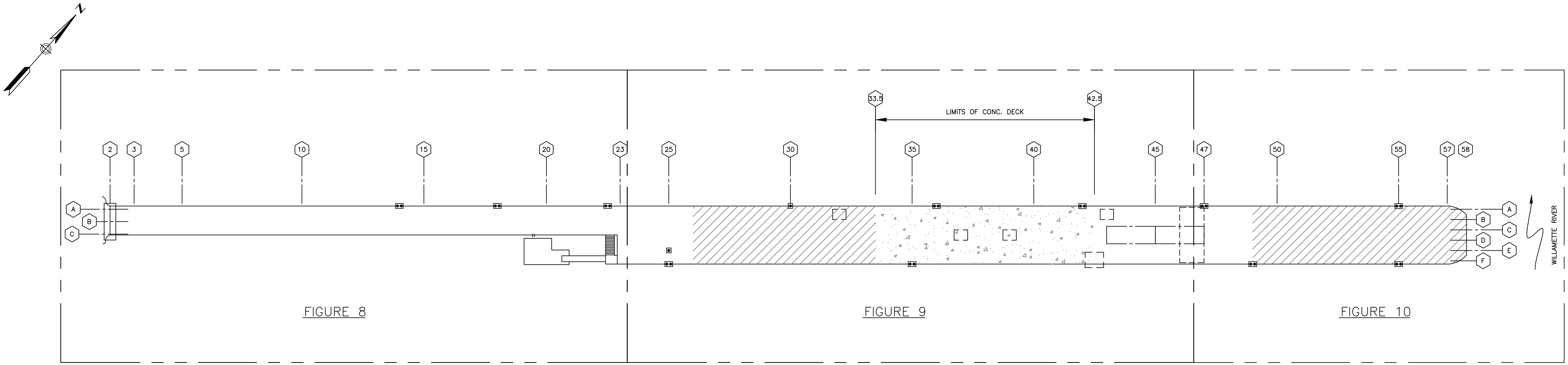

FIGURE 7: KEY PLAN
SCALE: 1/32"=1'-0"

NO.	DESCRIPTION	BY	APPR.	DATE
A	ISSUED FOR REVIEW	RJV	VVG	12/31/20

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MARINE STRUCTURAL CONDITION ASSESSMENT
FIGURE 7: DECK KEY PLAN

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N.E. #	CHKD. VVG	DRWN. TKB	PROJECT NO. RV1192	
RV1192	APPR. VVG	DATE 10/06/20	DWG. NO. D 1192-SKS010	REV. A
	PLOT SCALE 1:1	SCALE AS NOTED		

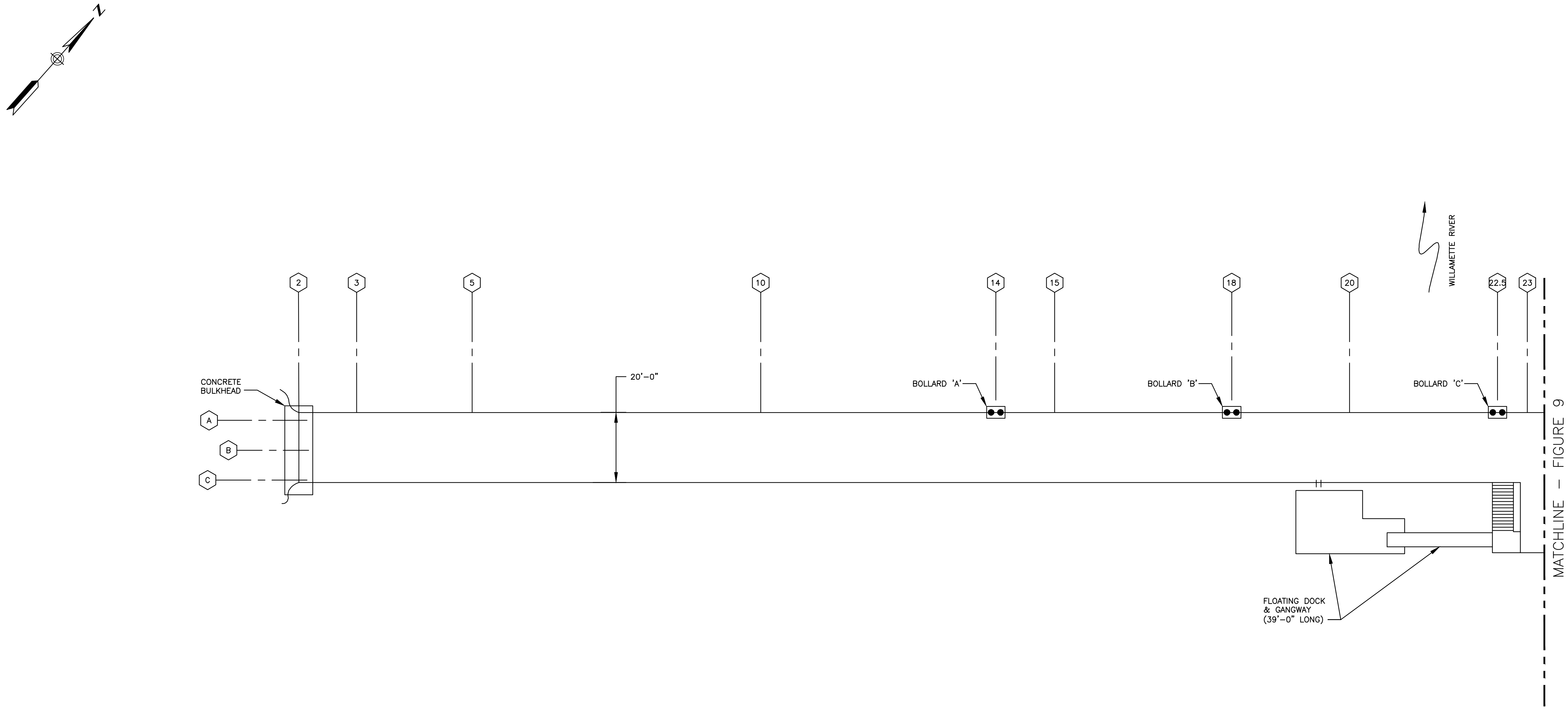

FIGURE 8: PLAN
SCALE: 1/16"=1'-0"

LEGEND:

\# – SV RATING

\# – MJ RATING

\# – MD RATING

A 1 – PILE GRID CENTERLINE

NO.	DESCRIPTION	BY	APPR.	DATE
A	ISSUED FOR REVIEW	RJV	VVG	12/31/20

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MARINE STRUCTURAL CONDITION ASSESSMENT
FIGURE 8: FACILITY LAYOUT – DECK

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N.E. # RV1192	CHKD. VVG	DRWN. TKB	PROJECT NO. RV1192
	APPR. VVG	DATE 10/06/20	DWG. NO. D 1192-SKS011 REV. A
	PLOT SCALE 1:1	SCALE AS NOTED	

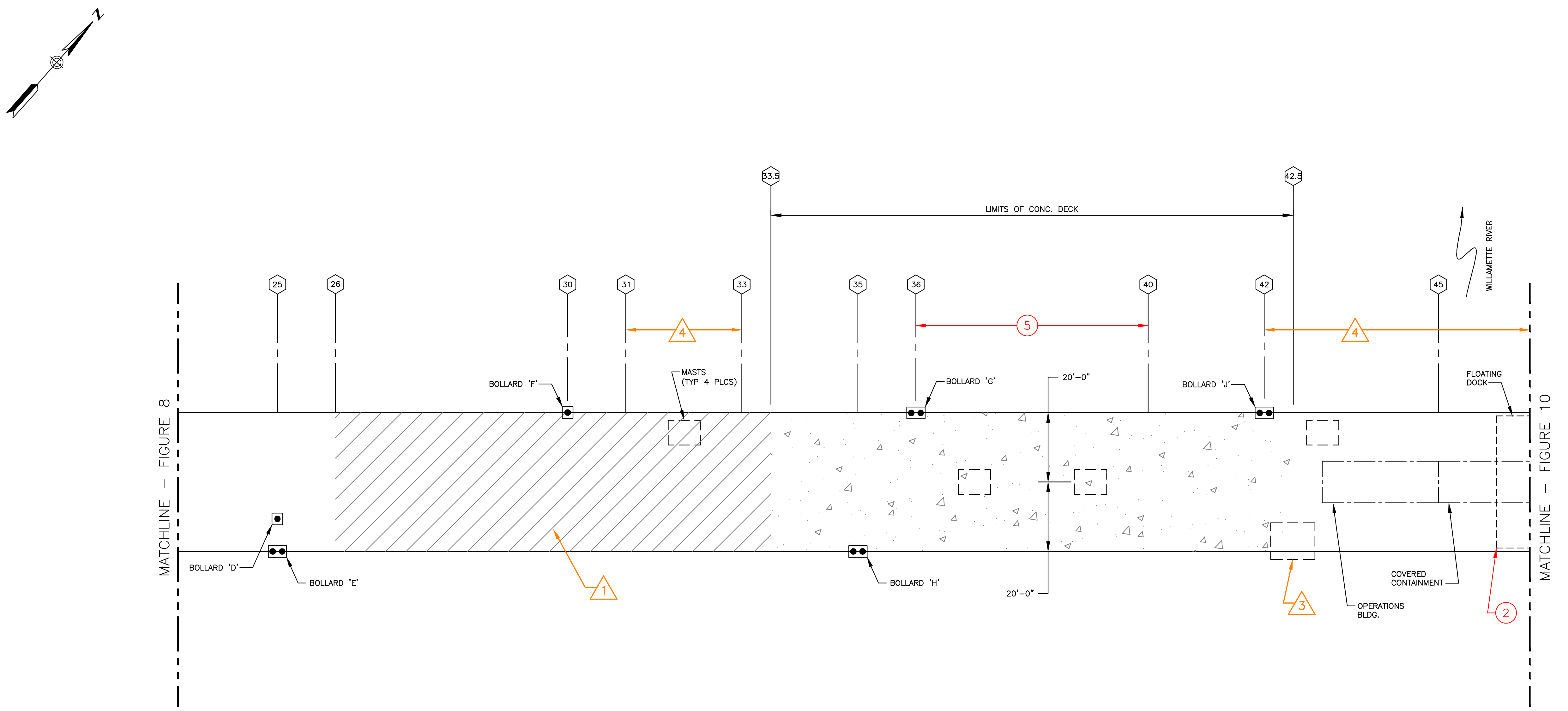

FIGURE 9: PLAN
SCALE: 1/16"=1'–0"

RATING ITEM #:

1. MULTIPLE INDIVIDUAL DECK BOARDS SHOW SIGNIFICANT ROT AND LOSS OF SECTION, MOLD INFESTATION, SWELLING. THE AREA SHOULD BE SURVEYED FOR BOARD REPLACEMENT. THE REMAINDER SHOULD BE MONITORED FOR FUTURE DETERIORATION.
2. FLOATING DECK ACCESS BELOW DECK: REPLACE TIMBER STAIR STRINGERS, HANDRAILS, LADDER ON LOWER LEVELS THIS END.
3. REPLACE INDIVIDUAL DECK BOARDS ON SUBSTRUCTURE PLATFORM NEAR ACCESS SWING GATE.
4. BULLRAIL EXHIBITS SPLITTING DAMAGE. REPLACE.
5. BULLRAIL EXHIBITS CROSS GRAIN TENSION OR TORSIONAL SPLITTING. REPLACE.

LEGEND:

- # – SV RATING
- # – MJ RATING
- # – MD RATING
- A 1 – PILE GRID CENTERLINE

NO.	DESCRIPTION	BY	APPR.	DATE
A	ISSUED FOR REVIEW	RJV	VVG	12/31/20

REVISIONS

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MARINE STRUCTURAL CONDITION ASSESSMENT
FIGURE 9: FACILITY LAYOUT – DECK

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N.E. # RV1192

CHKD. VVG	DRWN. TKB	PROJECT NO. RV1192
APPR. VVG	DATE 10/06/20	DWG. NO. D 1192–SKS012 REV. A
PLOT SCALE 1:1	SCALE AS NOTED	

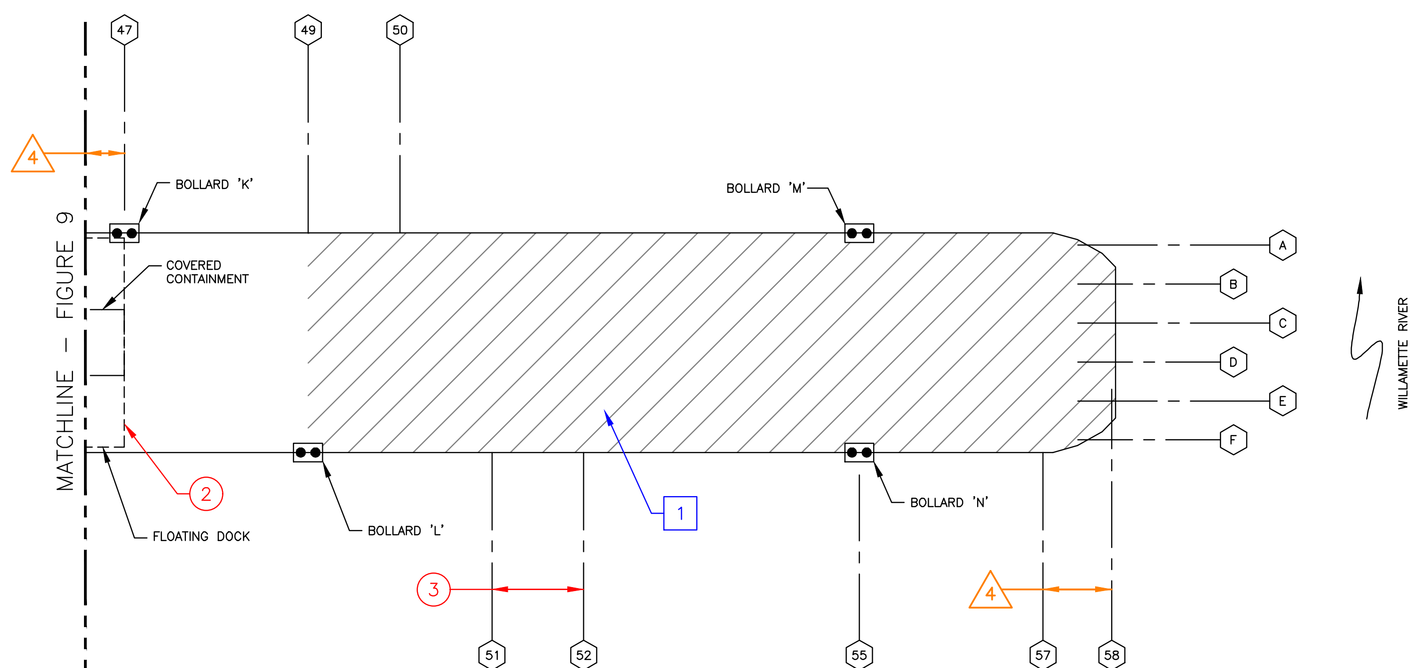

FIGURE 10: PLAN
SCALE: 1/16"=1'-0"

RATING ITEM #:

1. MULTIPLE INDIVIDUAL DECK BOARDS IN DISREPAIR AND SHOULD BE REPLACED. THE REMAINDER SHOULD BE MONITORED FOR FUTURE DETERIORATION.
2. FLOATING DECK ACCESS BELOW DECK: REPLACE TIMBER STAIR STRINGERS, HANDRAILS, LADDER ON LOWER LEVELS THIS END.
3. BULLRAIL WITH SEVERE SPLITTING DAMAGE AND DECAY. REPLACE.
4. BULLRAIL SHOWS SPLITTING DAMAGE. REPLACE.

LEGEND:

- # – SV RATING
- # – MJ RATING
- # – MD RATING
- A 1 – PILE GRID CENTERLINE

NO.	DESCRIPTION	BY	APPR.	DATE
A	ISSUED FOR REVIEW	RJV	VG	12/31/20

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MARINE STRUCTURAL CONDITION ASSESSMENT
FIGURE 10: FACILITY LAYOUT – DECK

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CHKD. VG	DRWN. TKB	PROJECT NO. RV1192
APPR. VG	DATE 10/06/20	DWG. NO. D 1192-SKS013 REV. A
PLOT SCALE 1:1	SCALE AS NOTED	

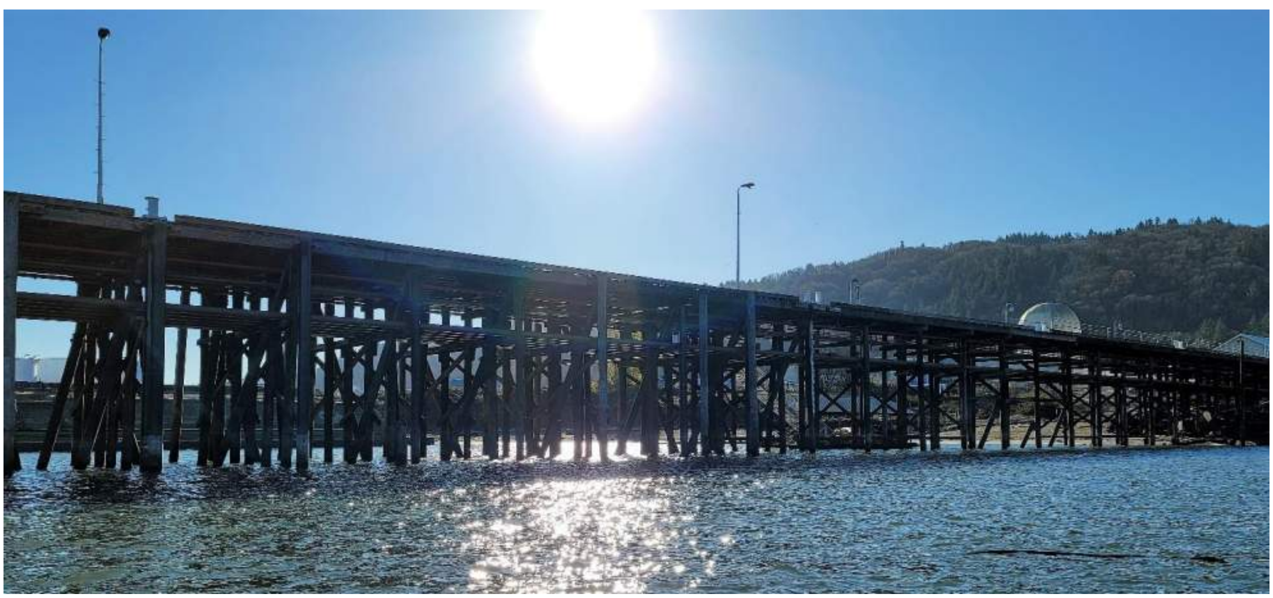
APPENDIX B:

SITE PHOTOGRAPHS

Visual record of structural conditions

circa 2020

Picture 1. Downstream berth and service area.



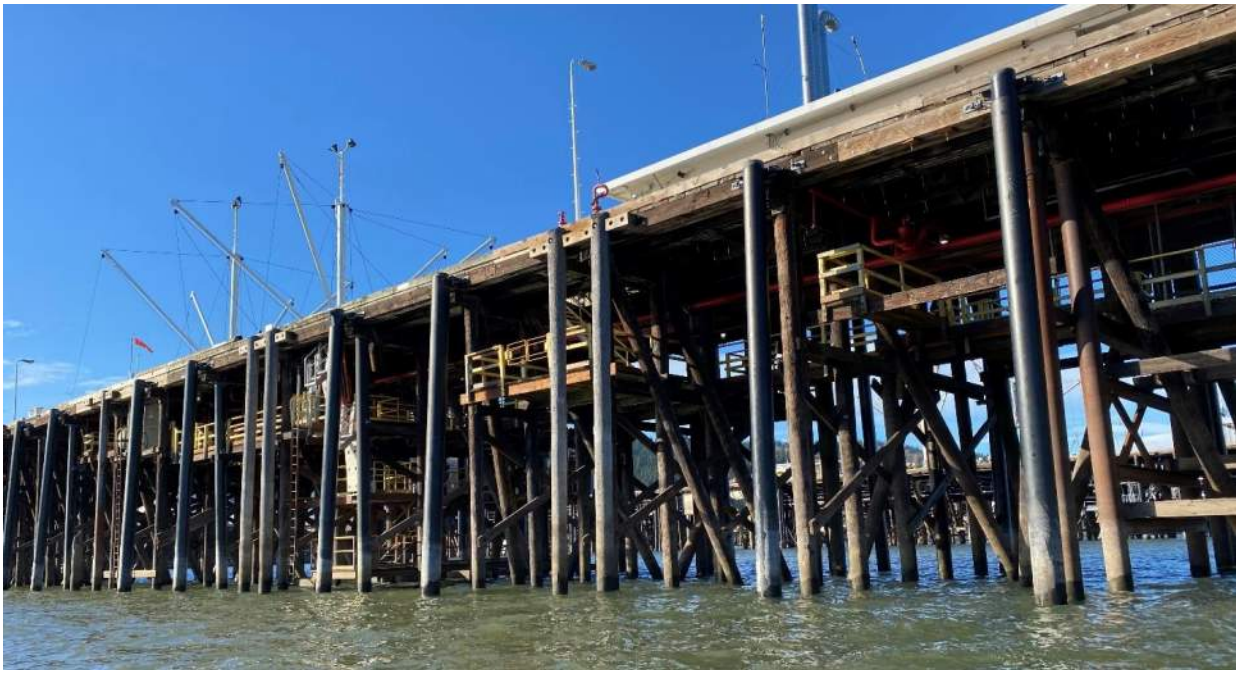
Picture 2. Downstream berth continued.

Picture 3. Pier approach from the downstream side. Note the combination of steel piles and bracing with the original timber elements.

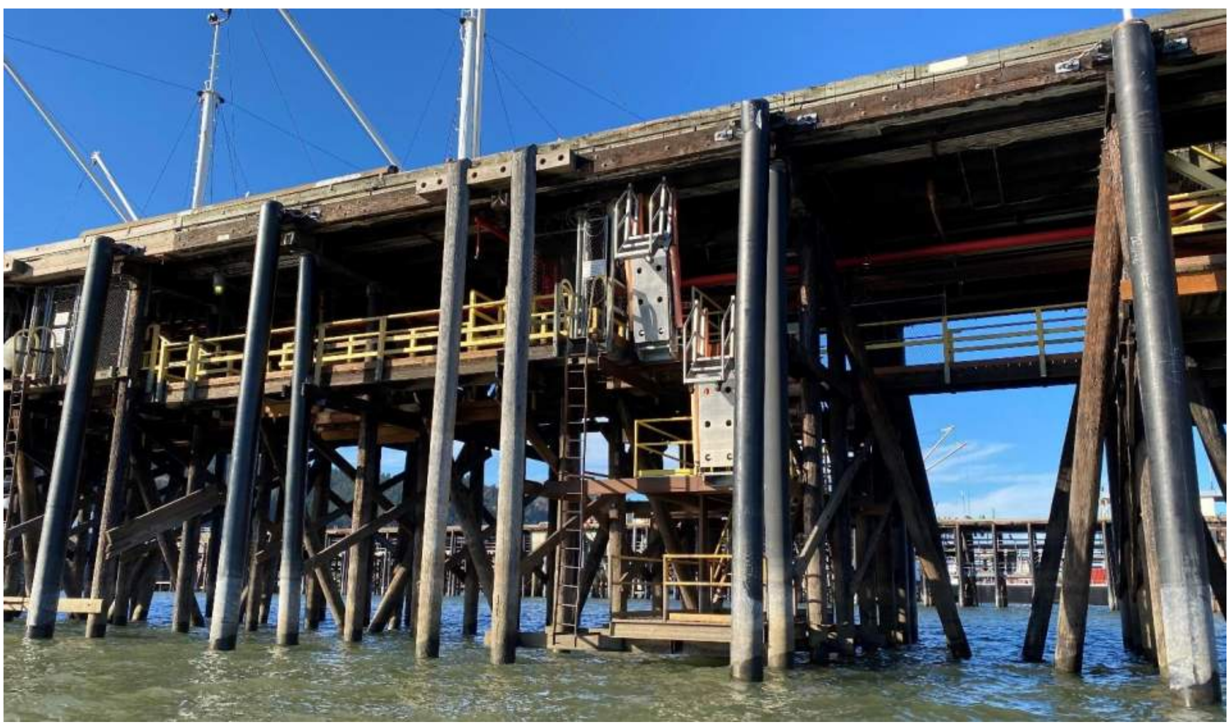
Picture 4. Approach to pier transition. Note the boathouse and access is the background.

Picture 5. Pier approach and boathouse, upstream side.

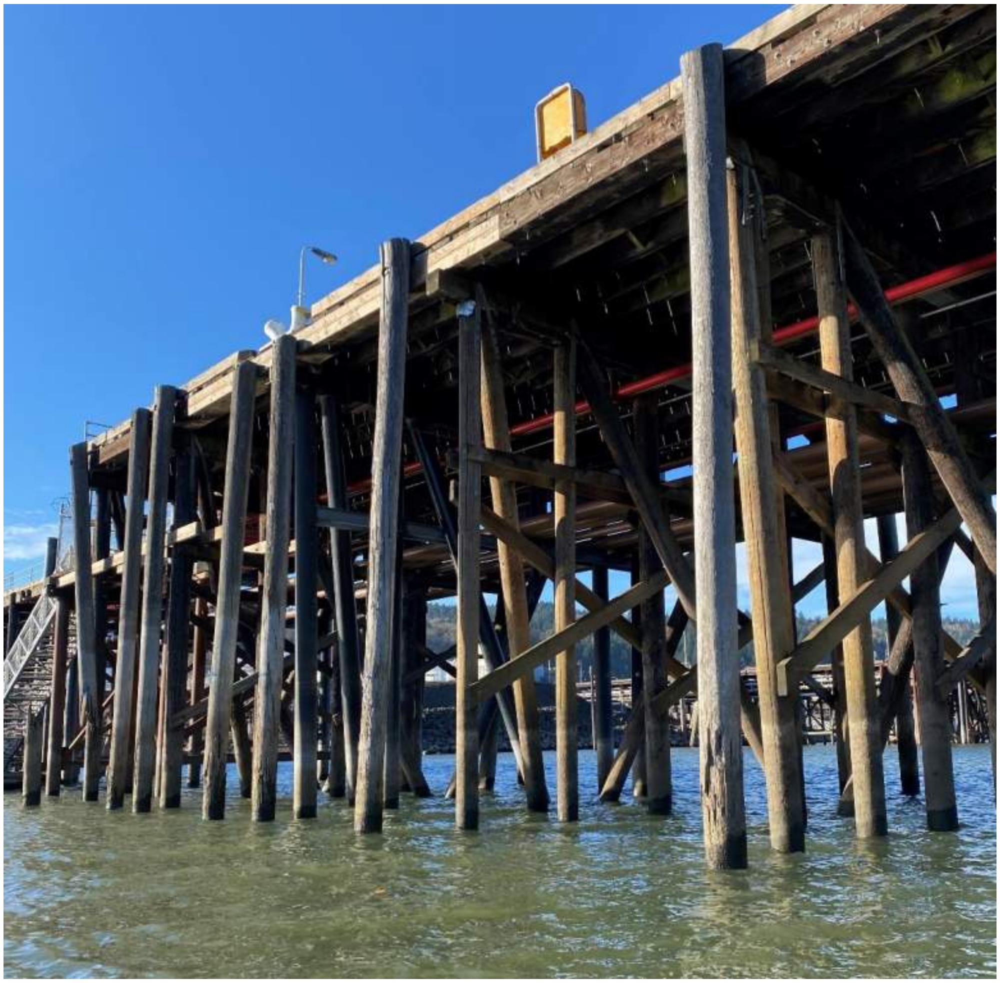
Picture 6. Pier approach at far west end, upstream side.



Picture 7. Upstream berth at the service area.



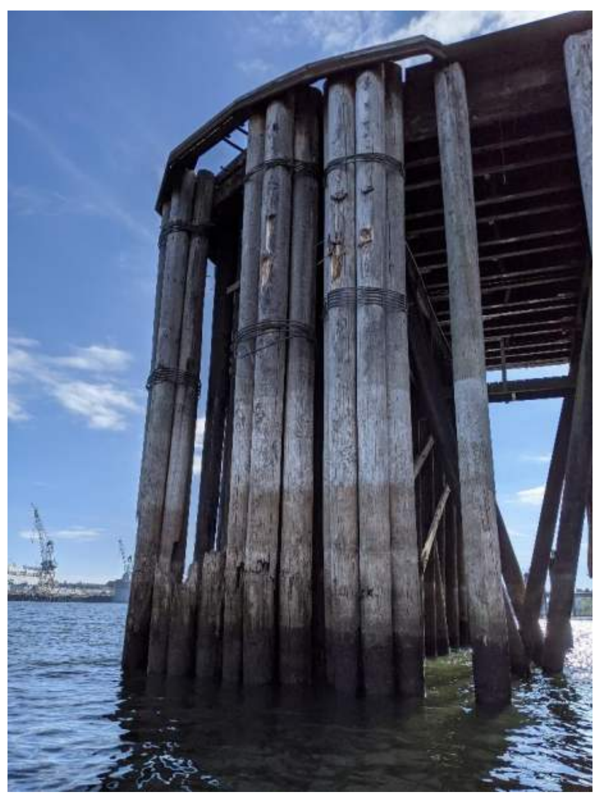
Picture 8. Upstream berth. Note the substructure walkways, multi-level platforms, ladder, and retractable gangways.



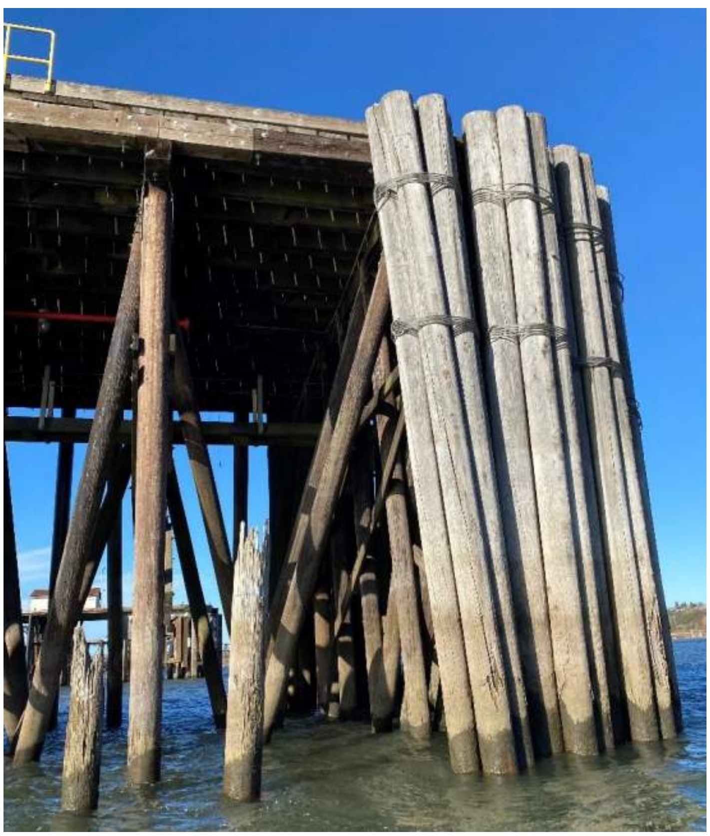
Picture 9. Upstream berth, shoreside. Note the fender piles broken at the water level., dead head.

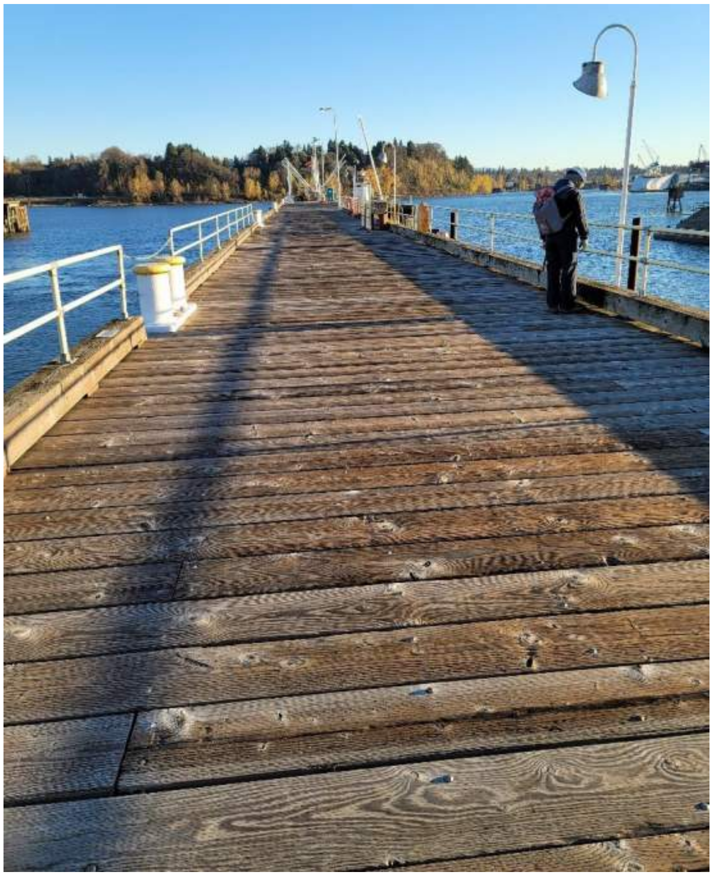
Picture 10. Boathouse and approach.

Pictures 11,12, 13, and 14. Pier head cluster piles. Note broken and missing piles, dead heads at either corner, broken and missing piles at head.



Pictures 15 and 16. Upstream berth, near head. Note broken/missing fender piles, exposed structure.

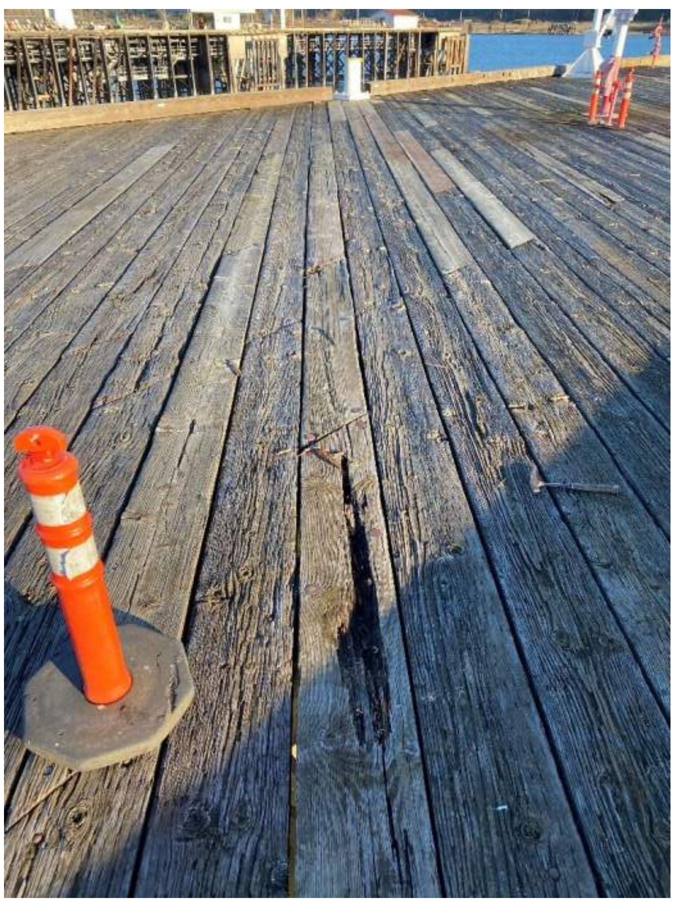




Picture 17. Deck and handrails along approach.

Picture 18. Bollard 'A', deck, handrails along approach. It does not appear to see regular use.

Picture 19. Bollard 'C'. Note wear to coating from line abrasion.

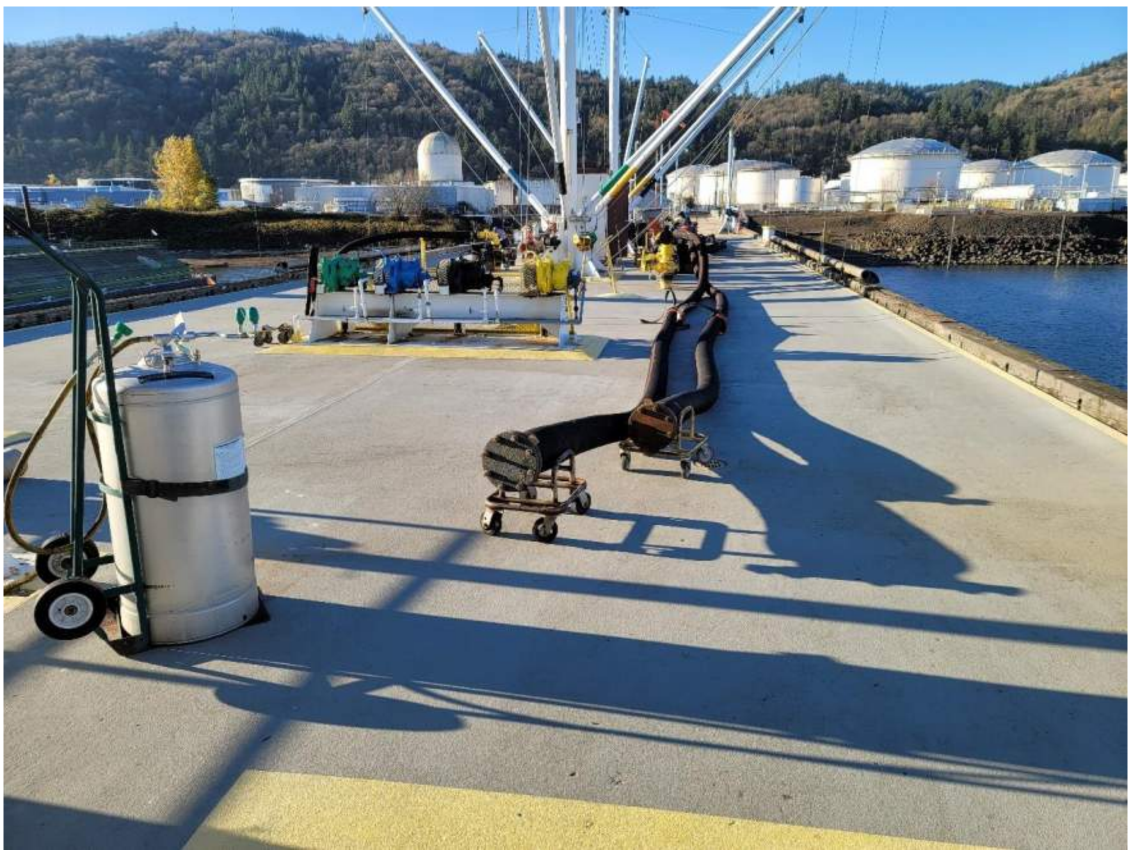


Picture 20. Evidence of significant decay to decking, shoreside of service area.

Picture 21. Decking riverside of the service area.

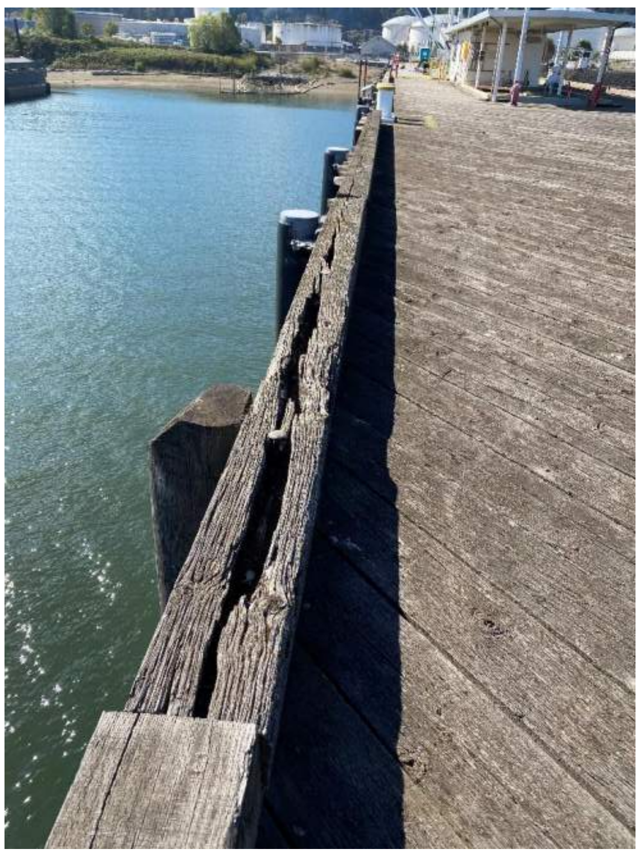
Picture 22. . Evidence of decay, riverside of service area.

Pictures 23: Service area and secondary containment, riverside of the operations shack.

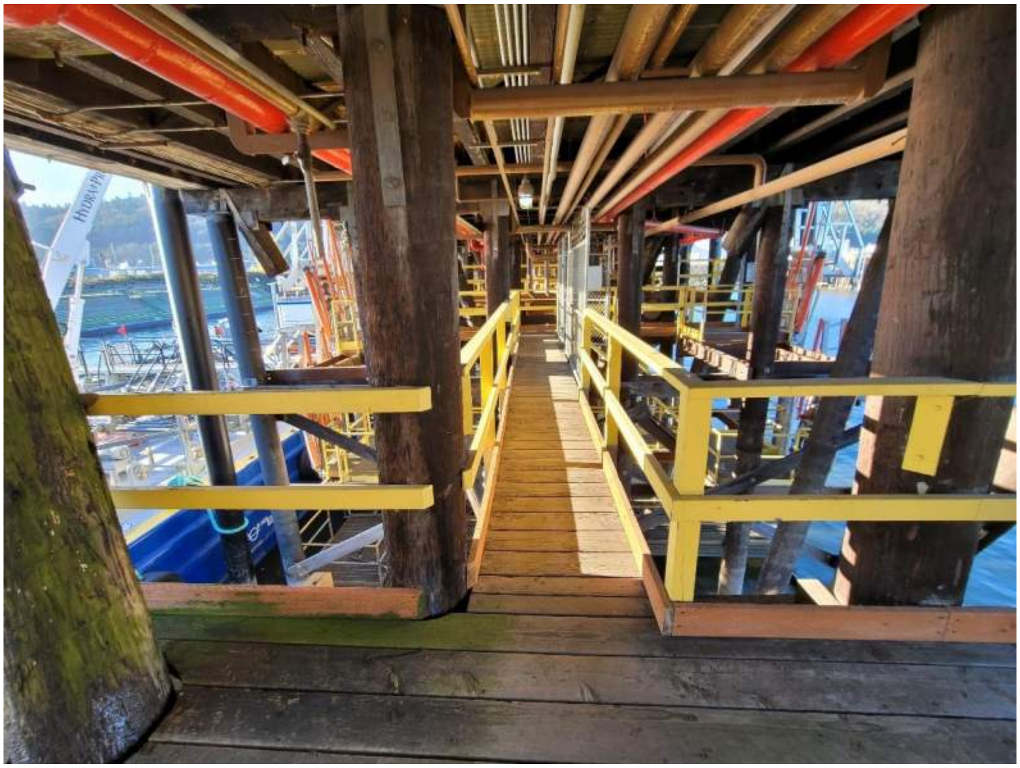


Picture 24. Service area. Note good condition of concrete surface.

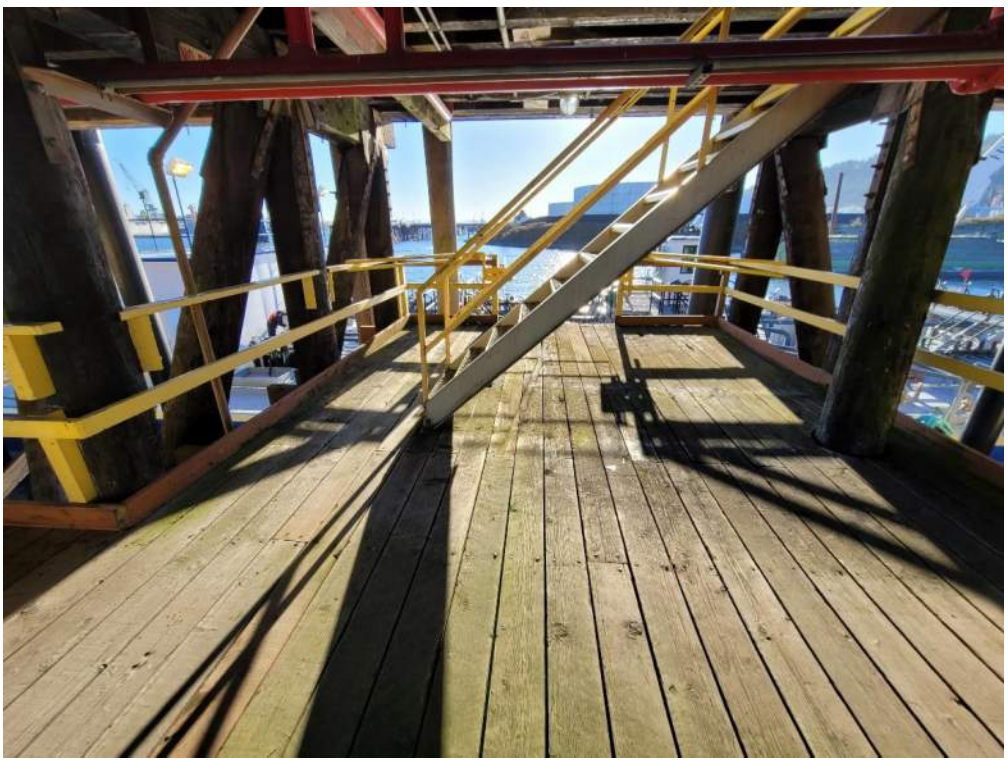
Picture 25. Split in bullrail, downstream berth.



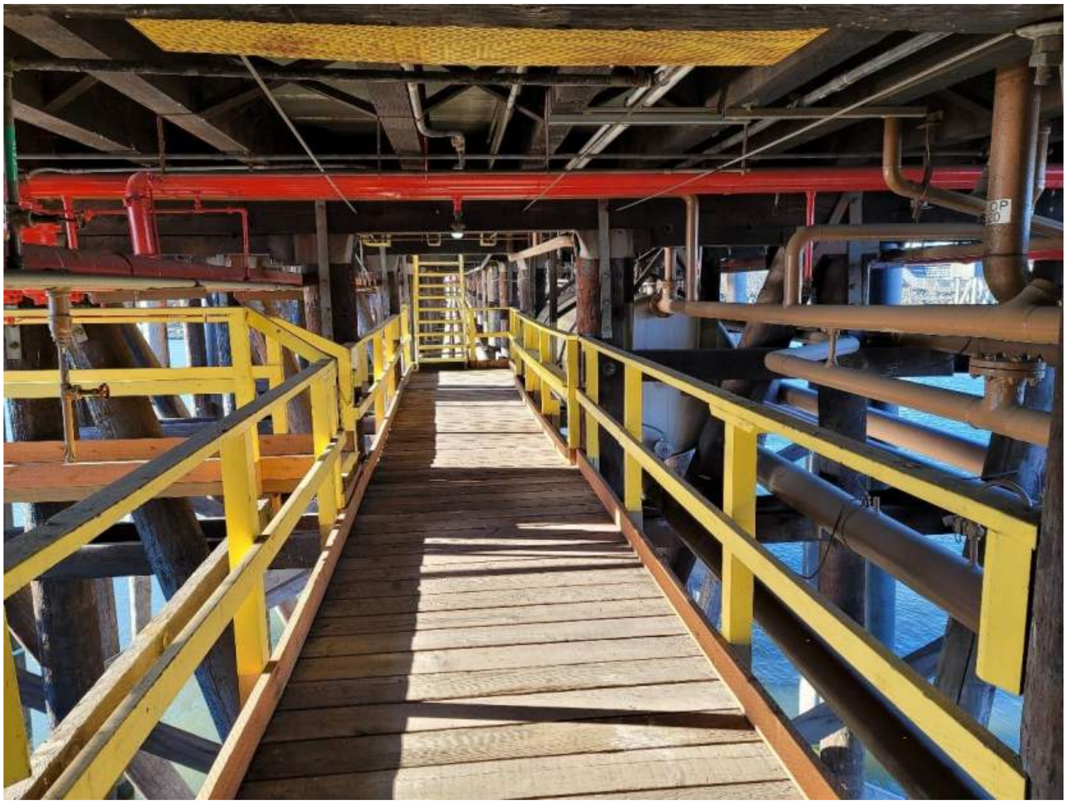
Picture 26. Severe fungal decay in bullrail, upstream berth.



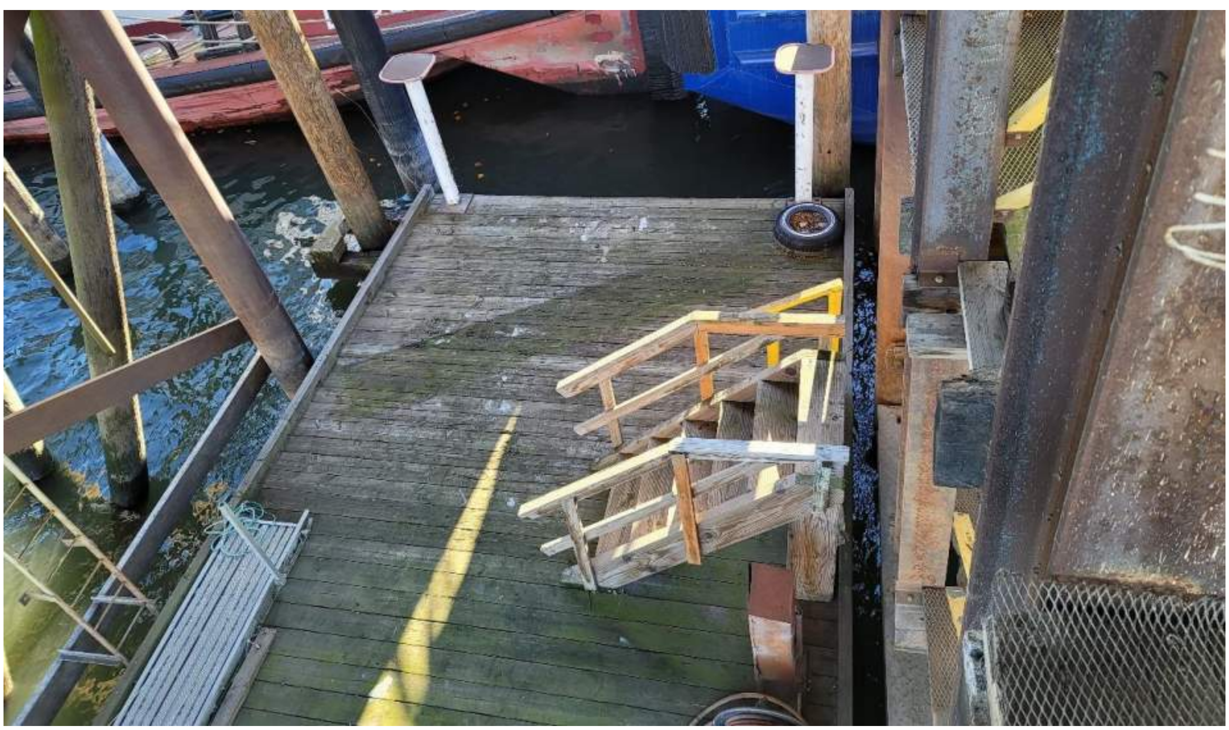
Picture 27. Substructure walkways and platforms.



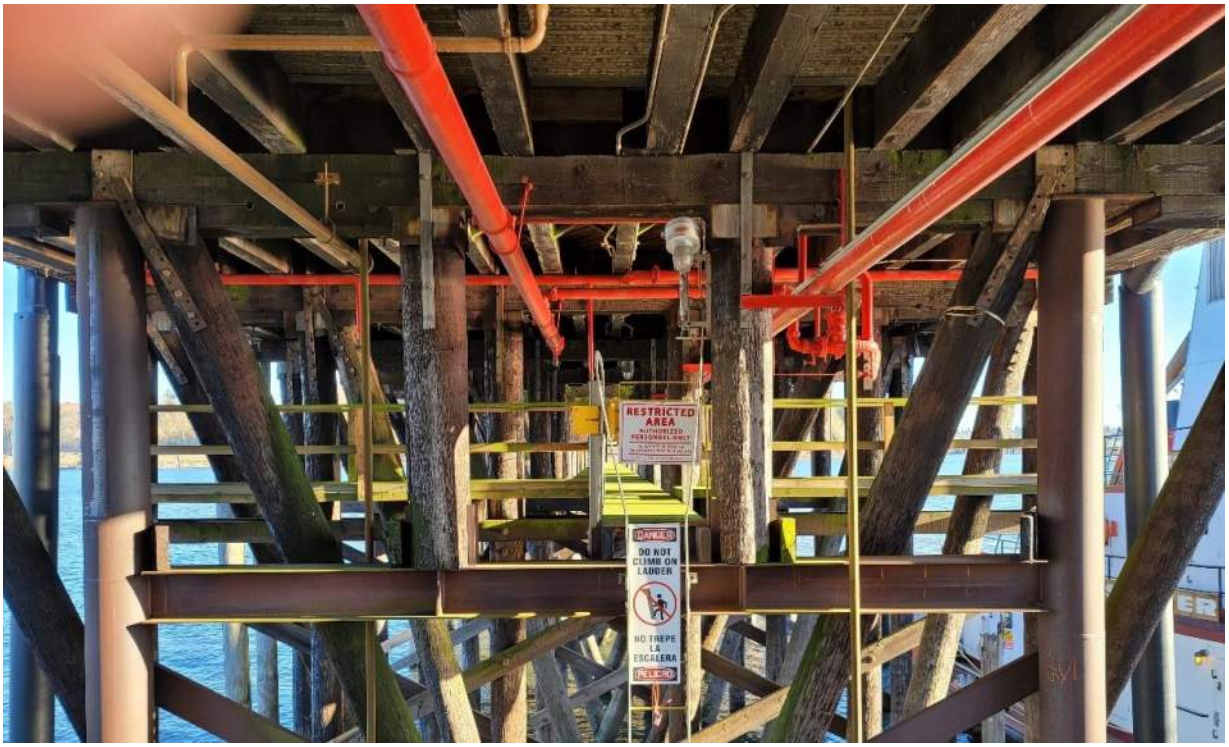
Picture 28. Substructure platform and staircase, shoreside of service area.



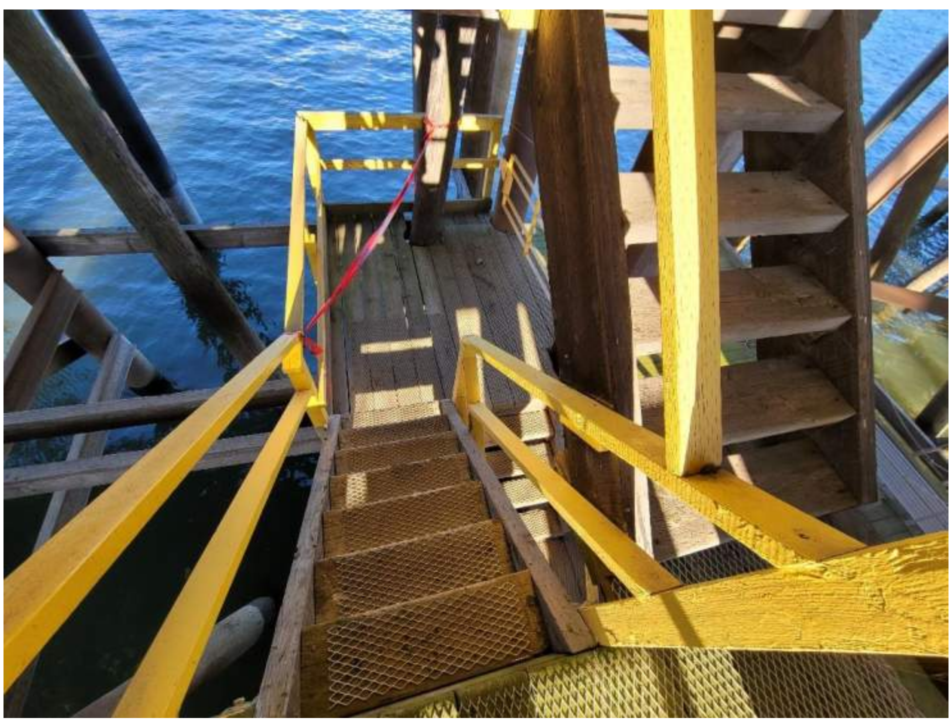
Picture 29. Substructure walkways, typical. Decking, handrails, paint in good shape.



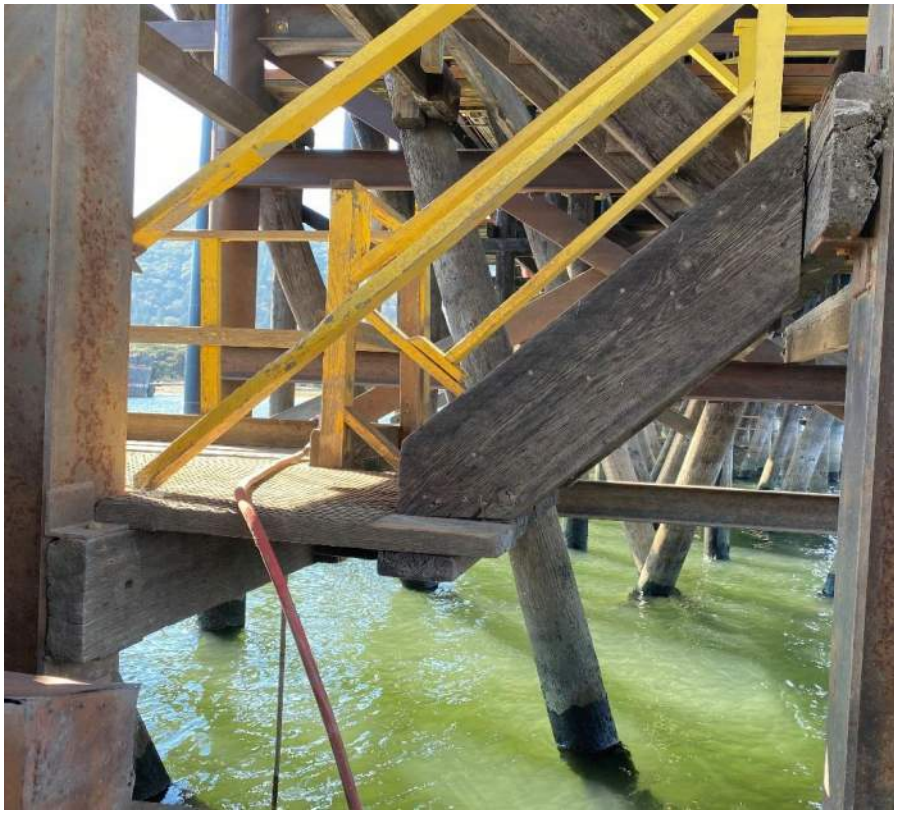
Picture 30. Floating deck, note stairs, timber guide (upper left). The deck is listing from left-to-right in the photo. The steel ladder to the left access the limited access walkway area.



Picture 31. Grid 47, looking east. Note significant sag (evidence of overstressing) in the pile cap and lack of mid-span support where the piles were removed below. Limited access walkway area is visible in the background.

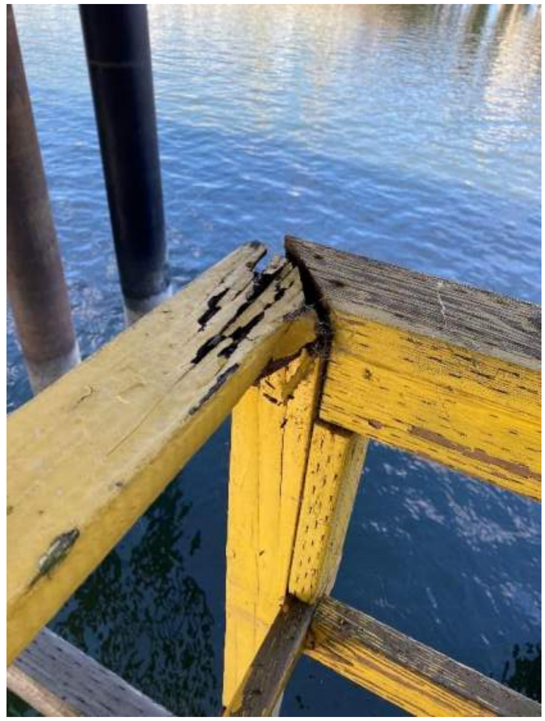


Picture 32. Stairs/platforms to the floating deck, Grid 46. The lower two levels were flagged for safety concerns.



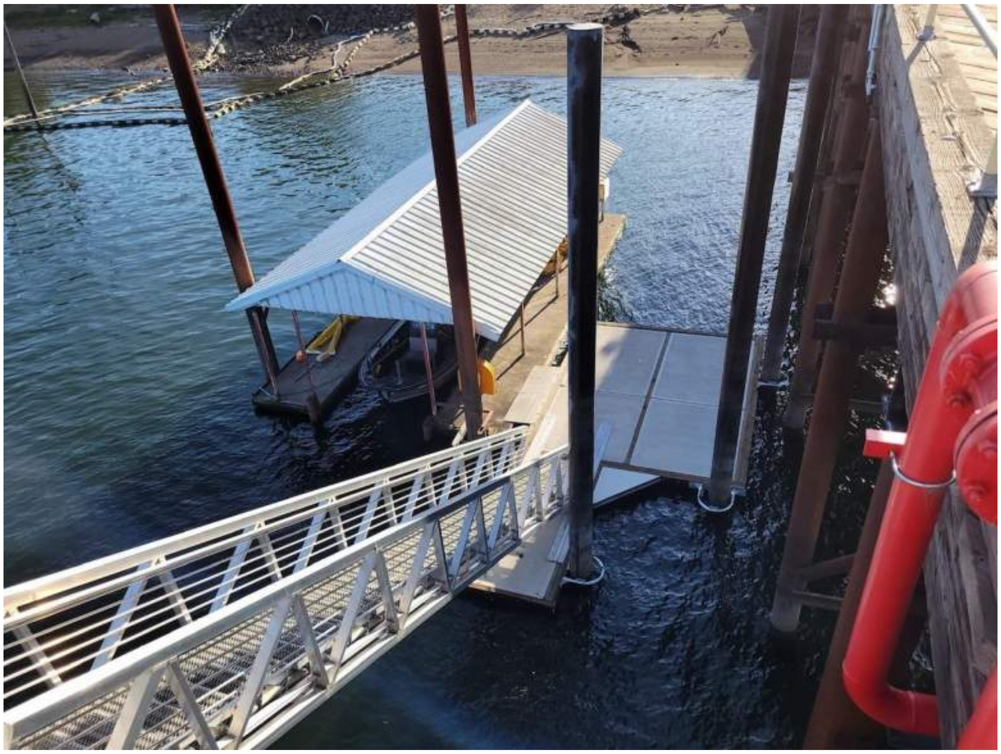
Picture 33. Grid 46, lower stair, taken from the floating deck. Note poor framing not catching base of stairs, evident decay in framing members, gaps forming in connections.

Picture 34. Grid 46, lower platform to floating deck, upstream side. Note significant framing decay and failing connection.



Pictures 35 and 36. Lower platform to floating deck in decay, Grid 46; handrail on third level broken at the corner connection.

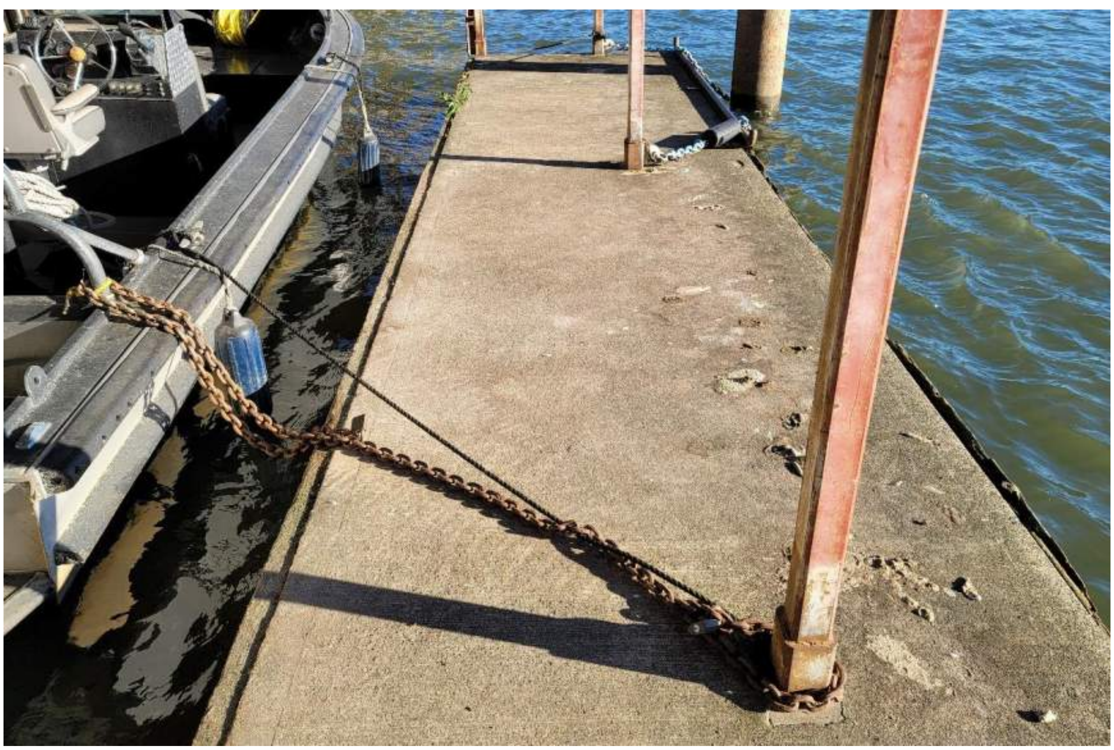
Picture 37. Platform near swing gate, upstream berth Grid 42. Note two deck boards in need of replacement, weathering and decay noted on ends of boards below gate in upper left corner.



Pictures 38. Boathouse, access gangway, and landing (2021).

Picture 39. Boathouse, gangway landing, and guide piles (2021).

Picture 40. Gangway landing and guides (2021).



Picture 41. Boathouse structure. Note good condition of concrete, roof columns.

Picture 42. Gangway landing (2021).

Picture 43. Gangway landing (2021). Typical guide system.

APPENDIX C:

ASCE REFERENCE: GUIDELINE FOR CONDITION AND DAMAGE RATINGS

REF: Heffron, et. al.. (2015). *Waterfront Facilities Inspection and Assessment, ASCE Manuals and Reports on Engineering Practice No. 130.* Reston, VA: American Society of Civil Engineers.

CONDITION ASSESSMENT RATINGS (GENERAL)

RATING	DESCRIPTION
GOOD	No visible damage or only minor damage noted. Structural elements may show very minor deterioration, but no overstressing observed. No repairs required.
SATISFACTORY	Limited minor to moderate defects or deterioration observed but no overstressing observed. No repairs are required.
FAIR	All primary structural elements are sound but minor to moderate defects or deterioration observed. Localized areas of moderate to advanced deterioration may be present but do not significantly reduce the load-bearing capacity of the structure. Repairs are recommended, but the priority of the recommended repairs is low.
POOR	Advanced deterioration or overstressing observed on widespread portions of the structure but does not significantly reduce the load-bearing capacity of the structure. Repairs may need to be carried out with moderate urgency.
SERIOUS	Advanced deterioration, overstressing, or breakage may have significantly affected the load-bearing capacity of primary structural components. Local failures are possible and loading restrictions may be necessary. Repairs may need to be carried out on a high-priority basis with urgency.
CRITICAL	Very Advanced deterioration, overstressing, or breakage has resulted in localized failure(s) of primary structural components. More widespread failures are possible or likely to occur, and load restrictions should be implemented as necessary. Repairs may need to be carried out on a very high-priority basis with strong urgency.

ELEMENT DAMAGE RATINGS*

RATING	DESCRIPTION
ND	No Damage
MN	Minor Damage
MD	Moderate Damage
MJ	Major Damage
SV	Severe Damage

*See damage ratings for differing elements and materials on the following pages.

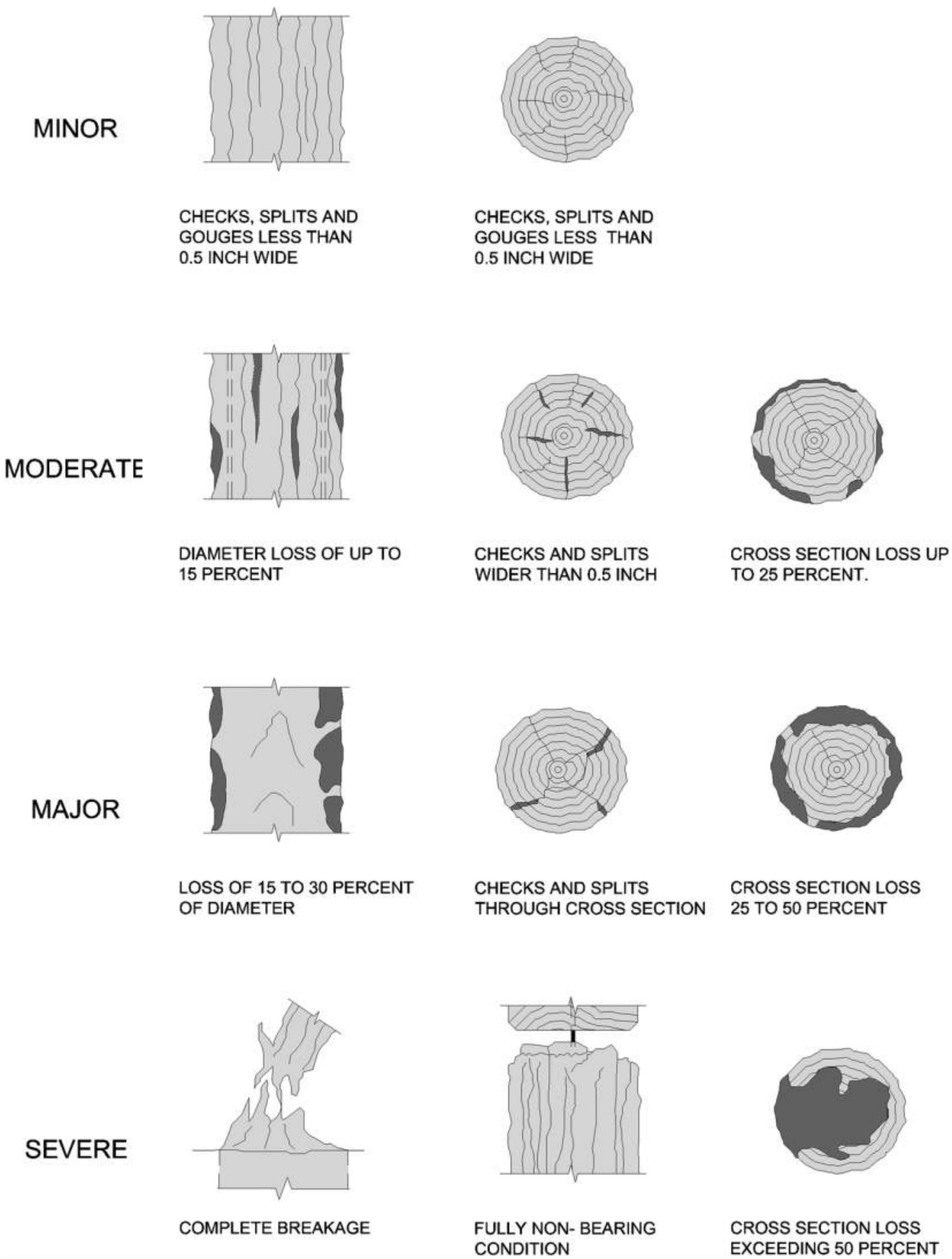


Figure 1: Condition Ratings for Timber Elements

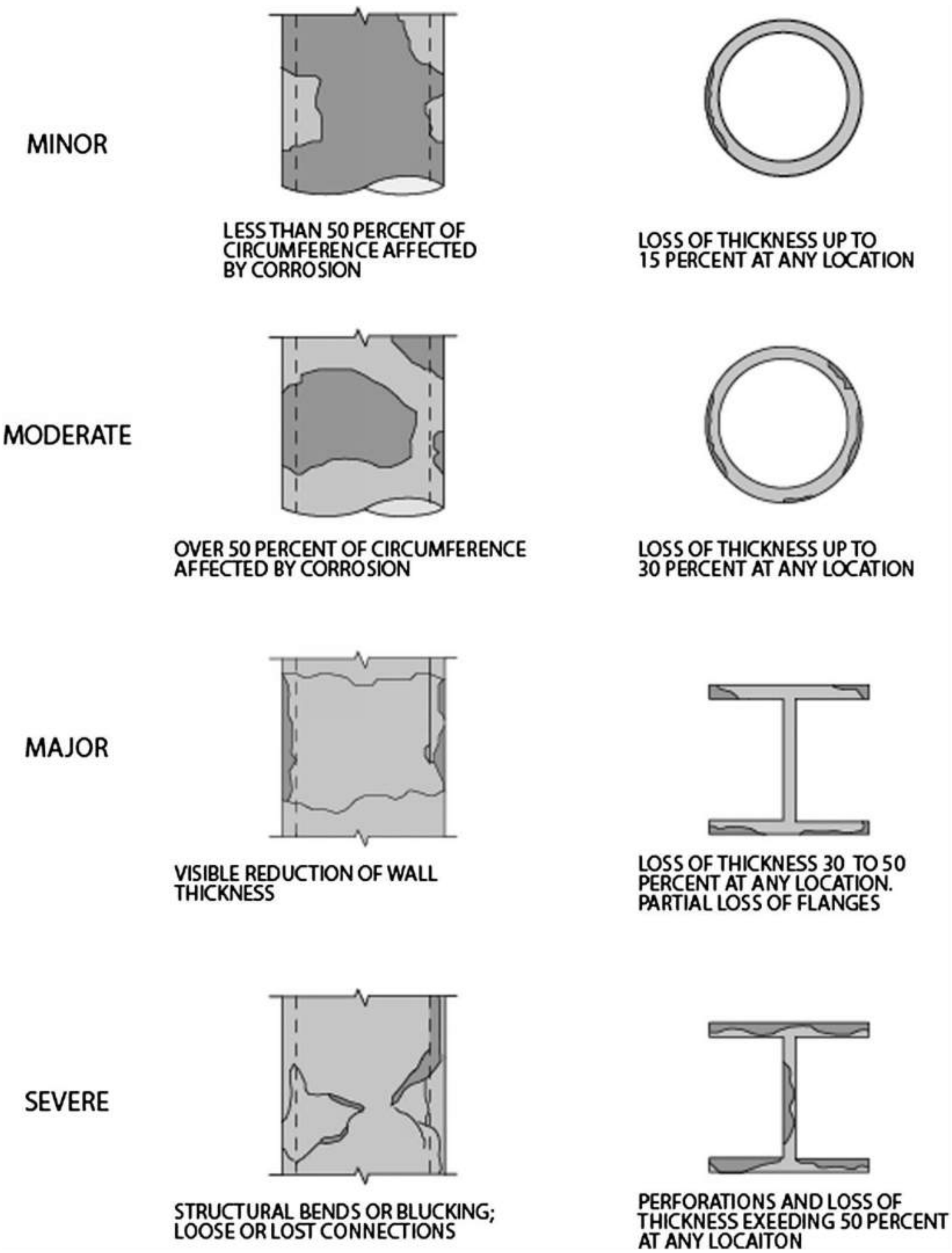


Figure 2: Damage Ratings for Steel Elements

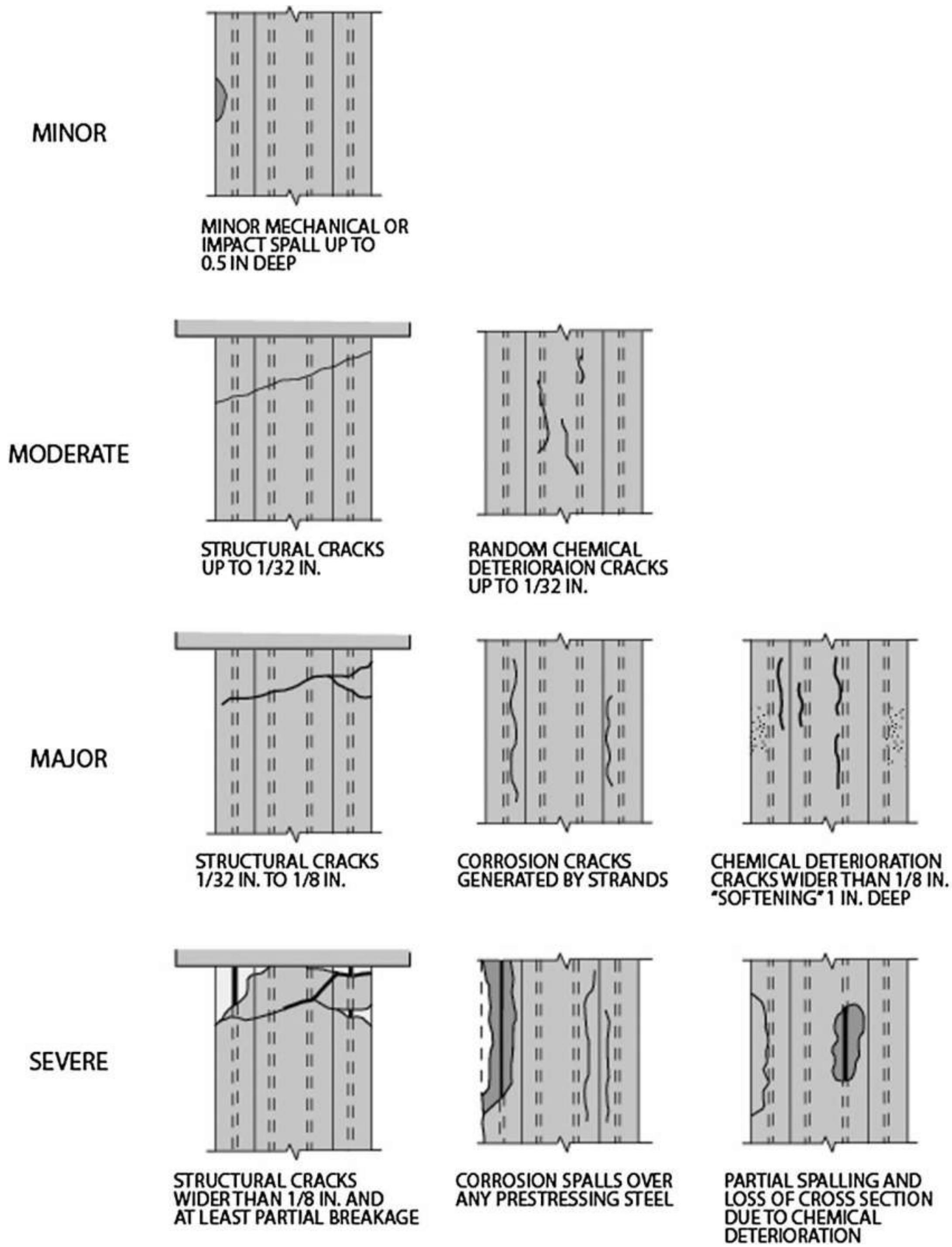


Figure 3: Damage Ratings for Pre-Stressed Concrete Elements

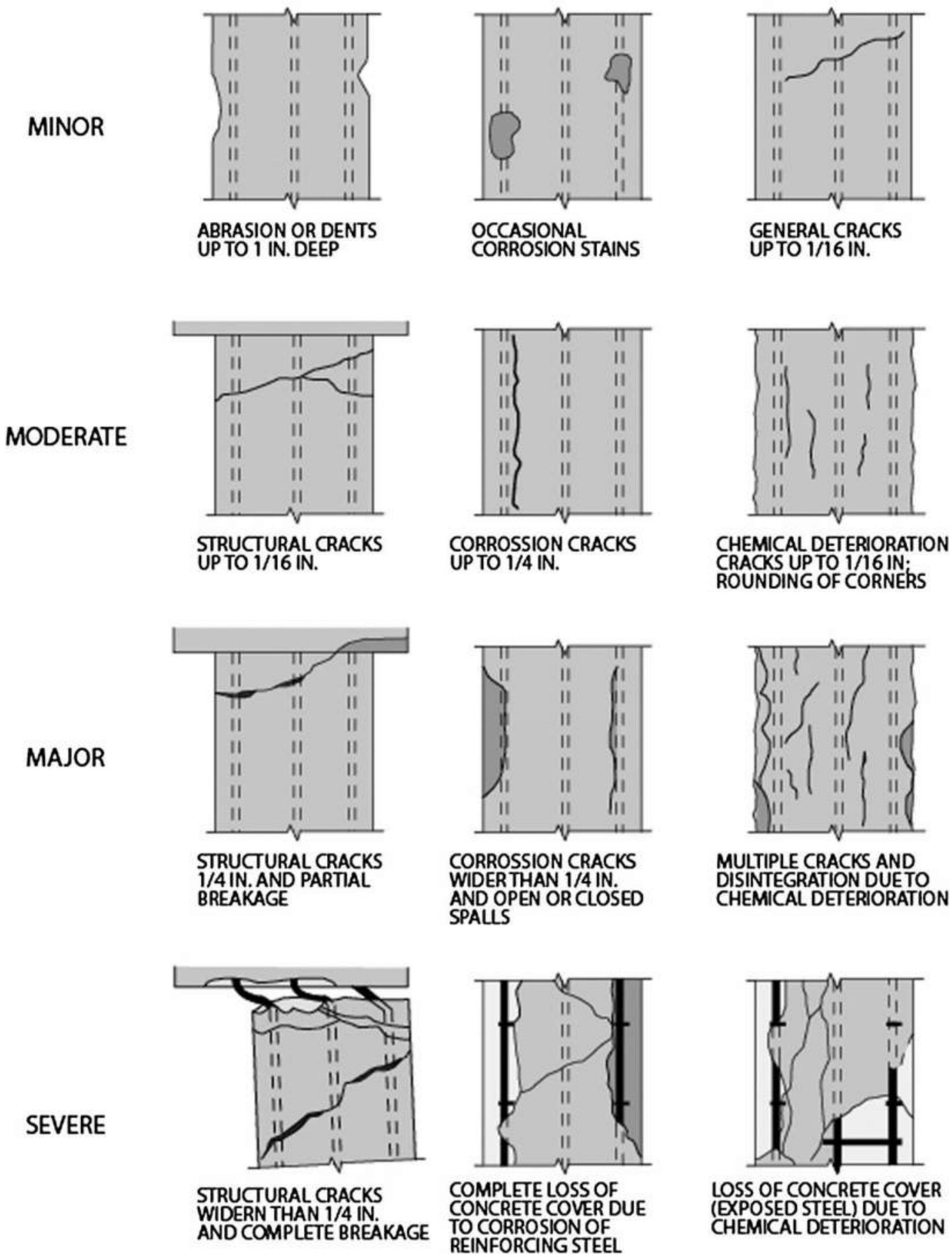


Figure 4: Damage Ratings for Reinforced Concrete Elements

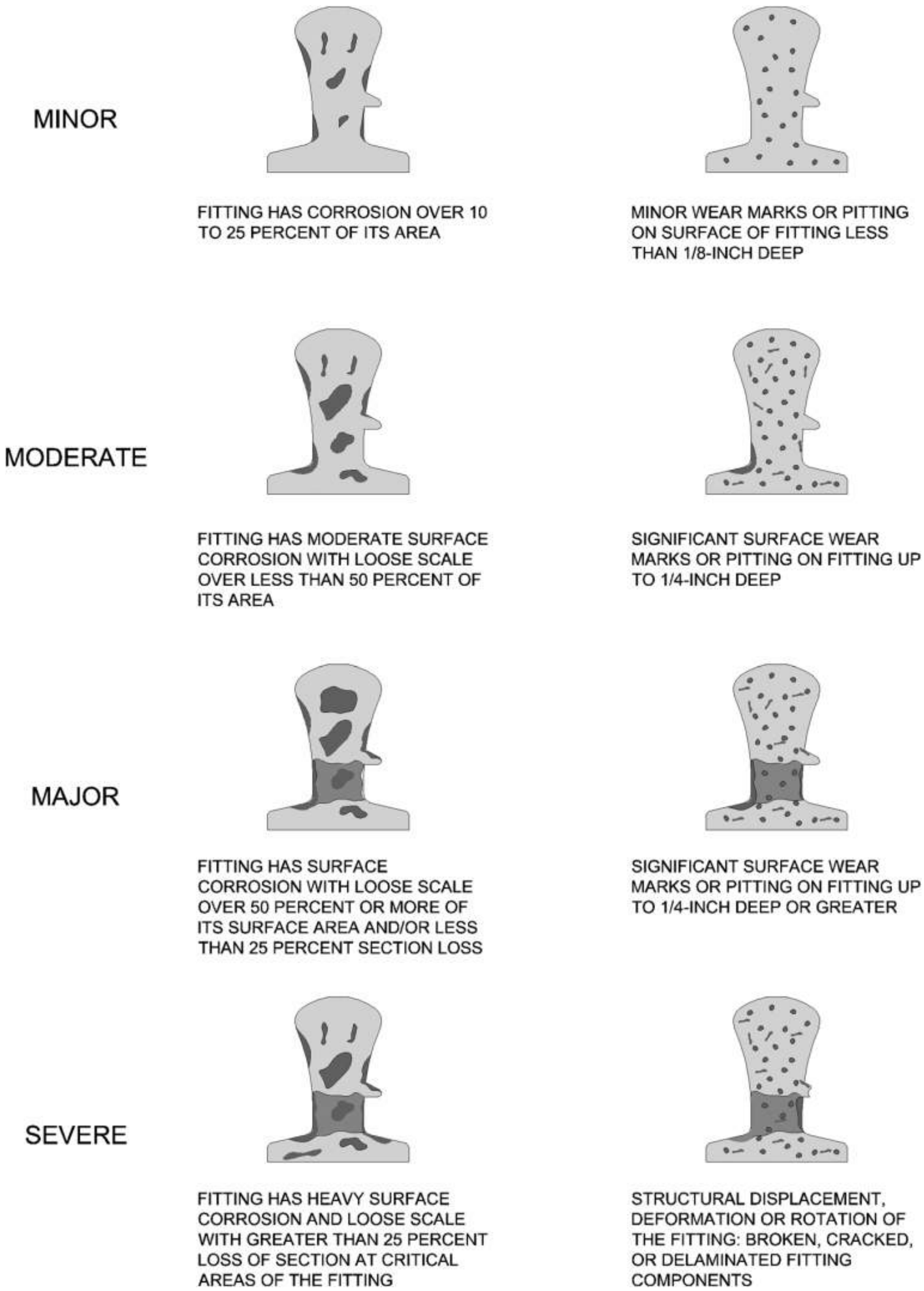


Figure 5: Damage Ratings for Mooring Hardware Elements (1)

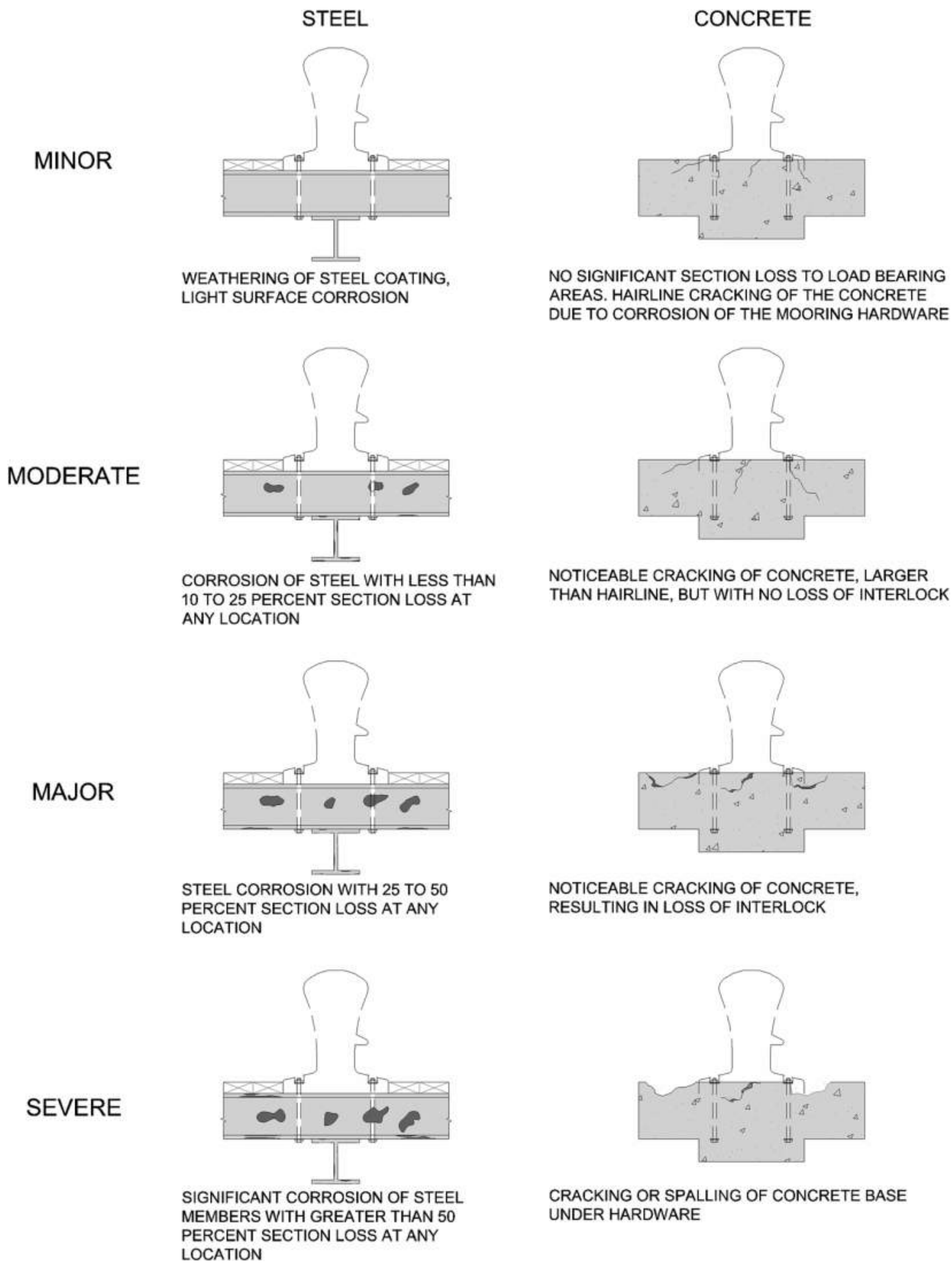


Figure 6: Damage Ratings for Mooring Hardware Elements (2)

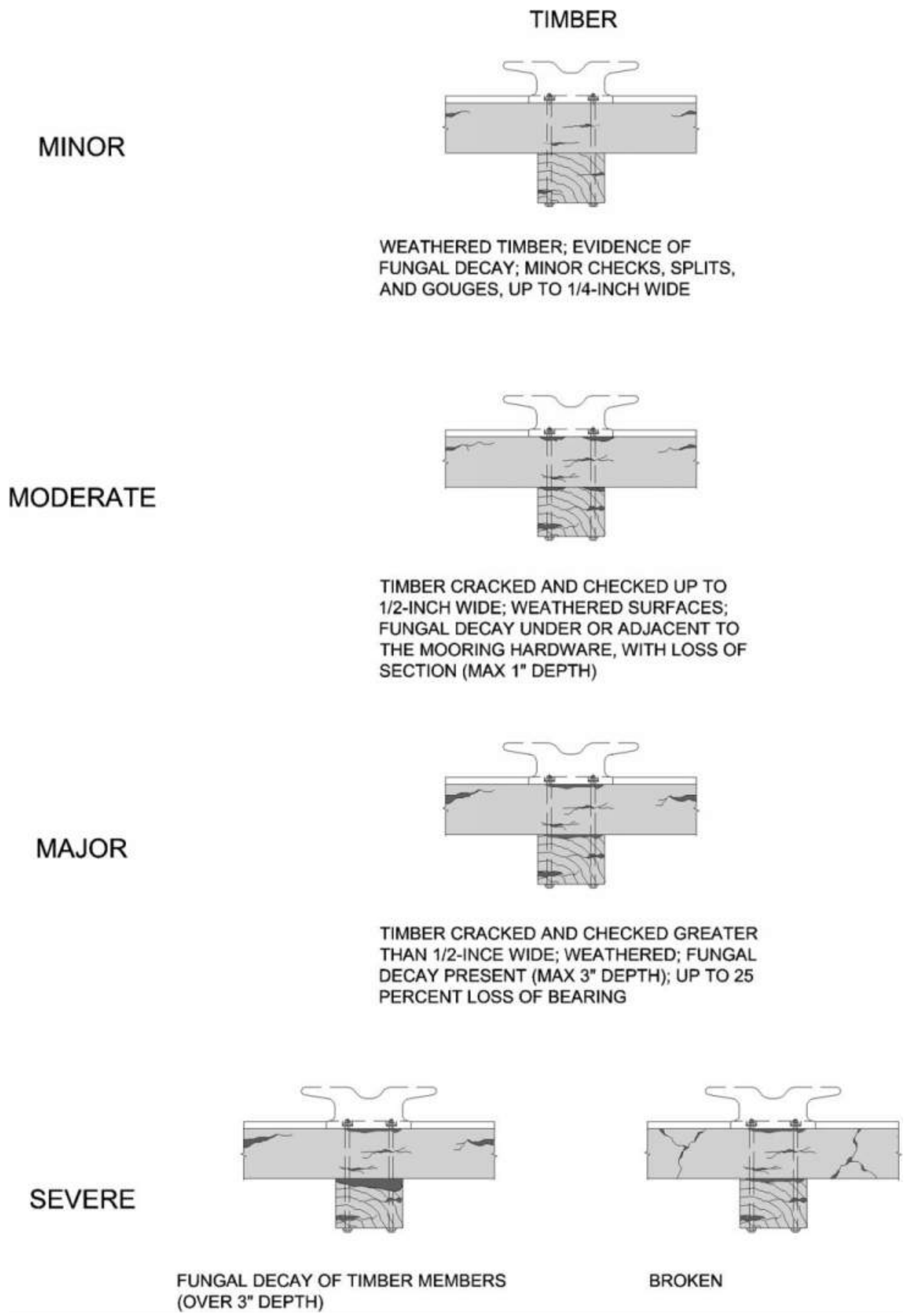


Figure 7: Damage Ratings for Timber Mooring Foundation Elements

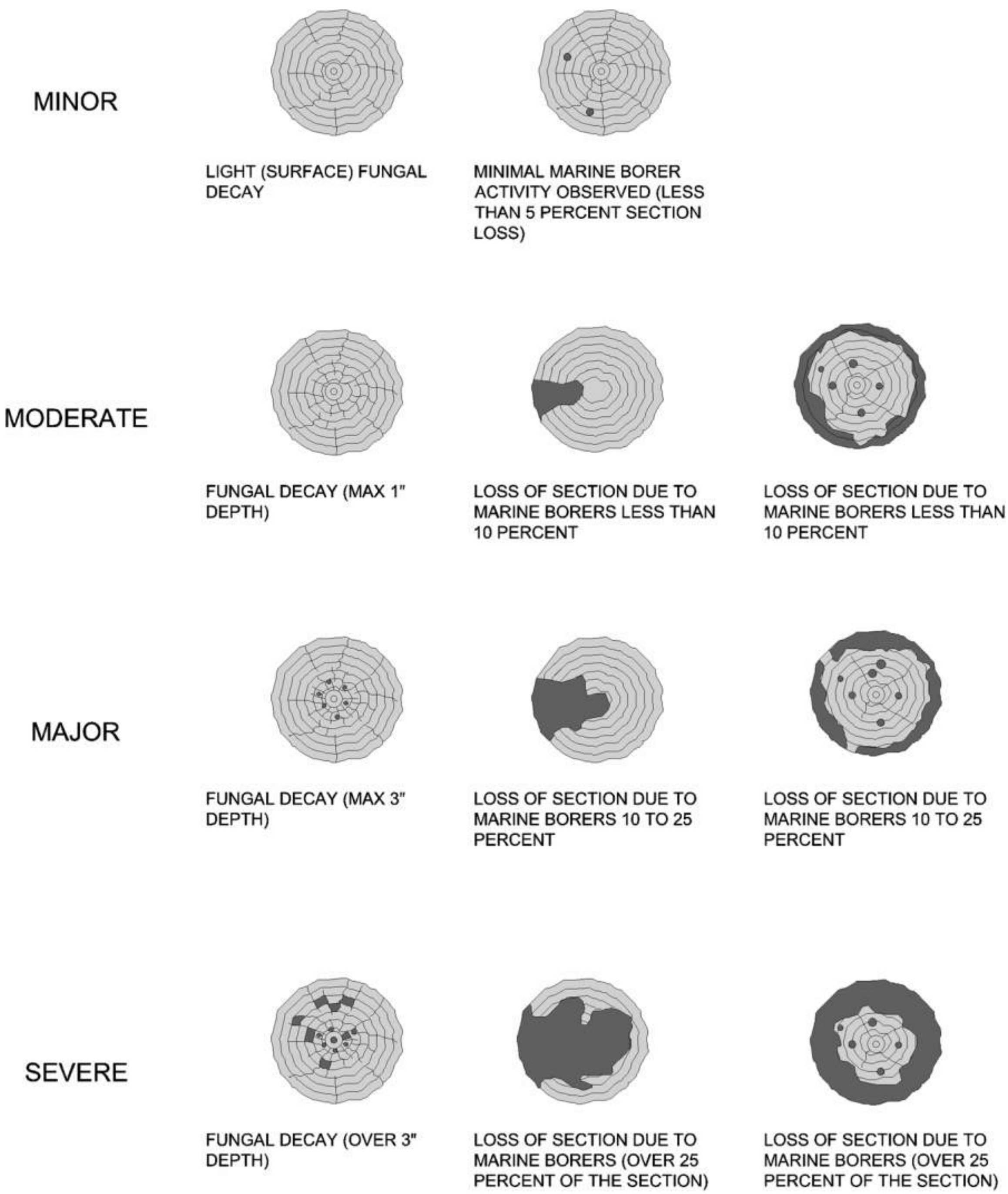


Figure 8: Damage Ratings for Timber Fender Pile Elements

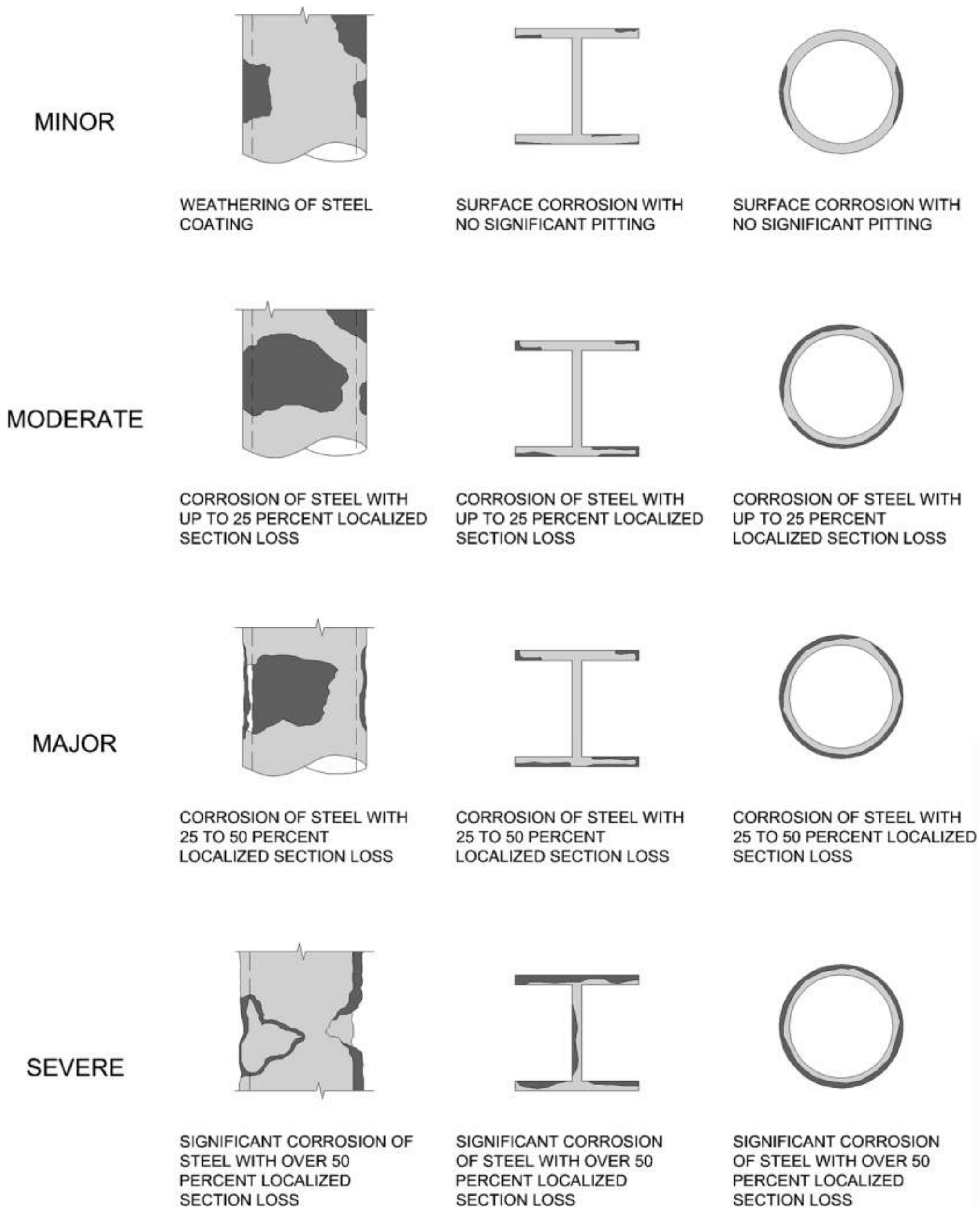


Figure 9: Damage Ratings for Steel Fender Pile Elements

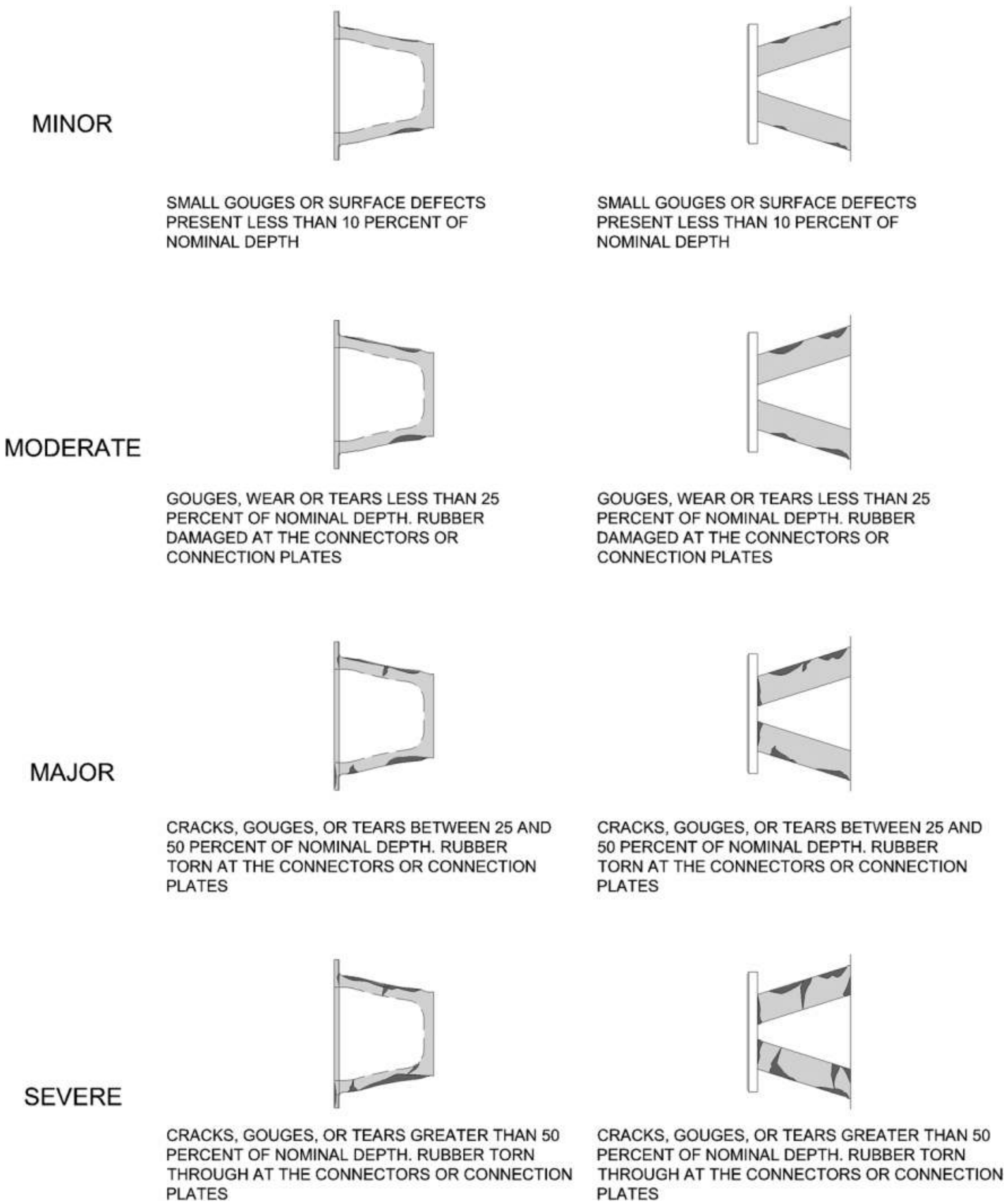


Figure 10: Damage Ratings for Rubber Fender Elements

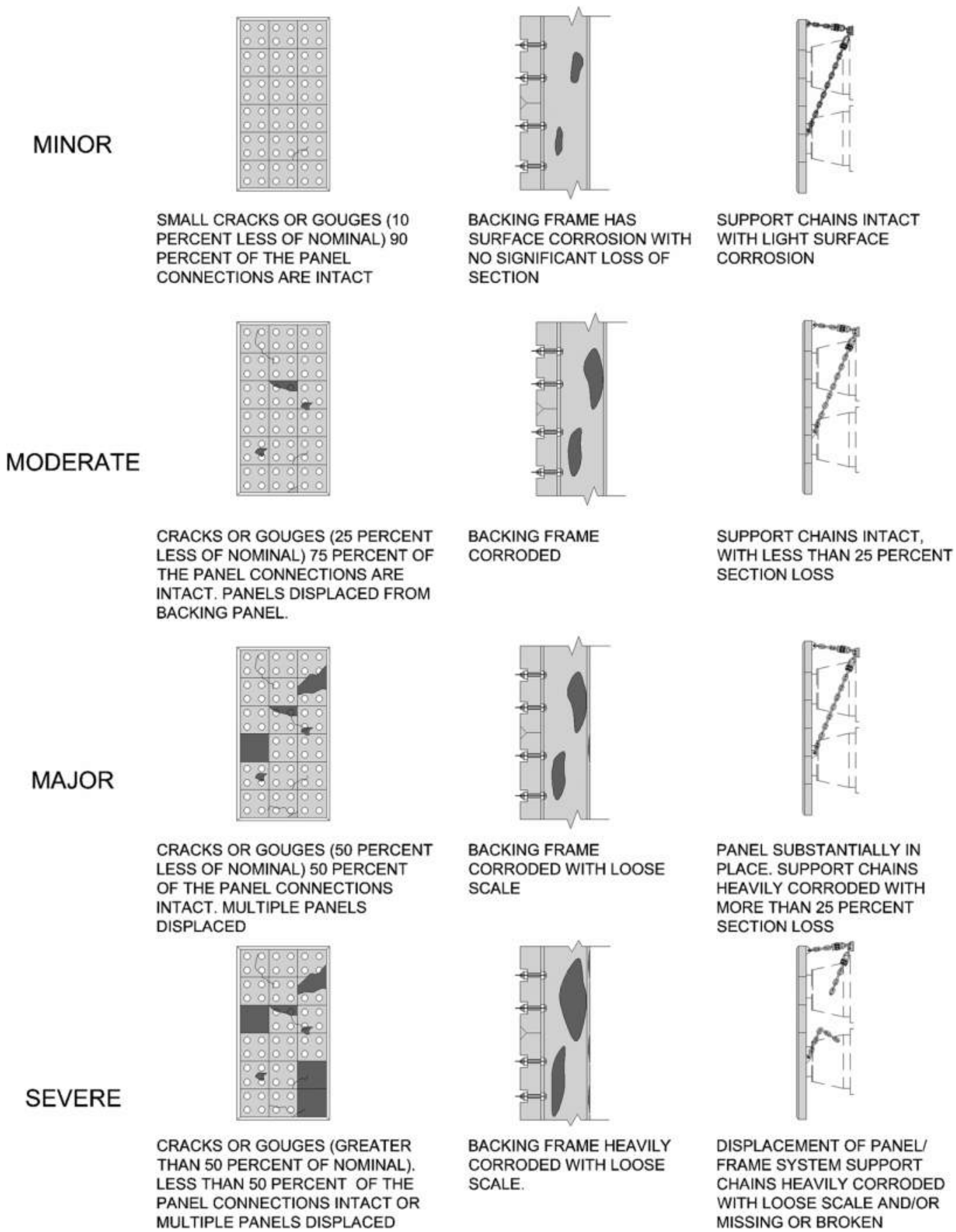


Figure 11: Damage Ratings for Fender Panel Elements

APPENDIX D:

UNDERWATER INSPECTION REPORT

SUBSEA GLOBAL SOLUTIONS

Pier 66

Pile Inspection (Underwater)

Oct 26-29, 2020
Portland, OR

Client:

Norwest Engineering
4110 Northeast 122nd Ave. Suite 207
Portland, OR 97230
USA

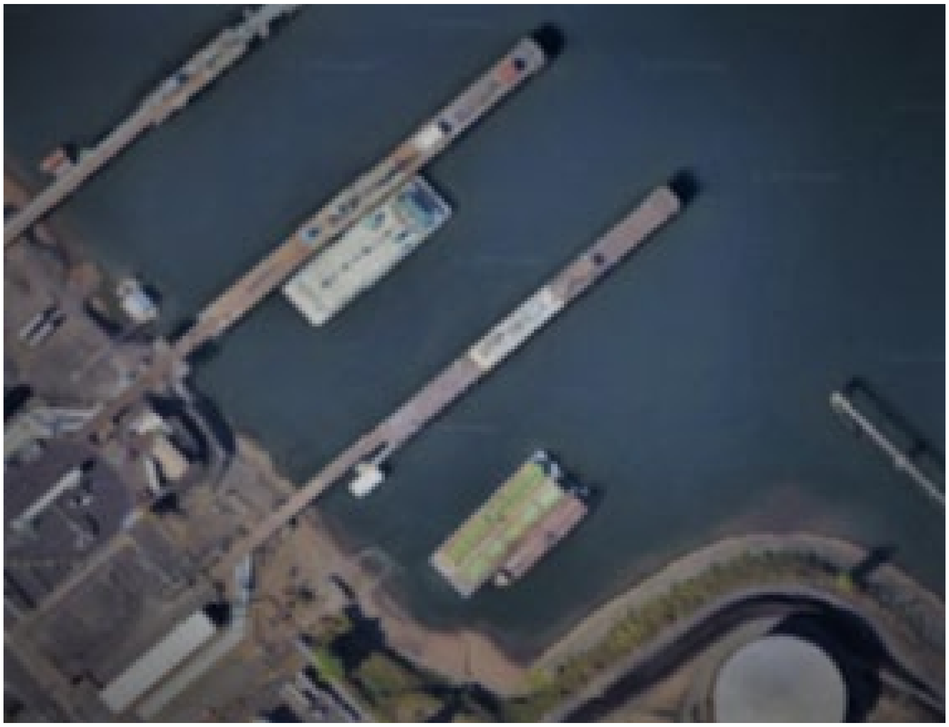
PO: RV1192-001

Job Number: 2425

SUBSEA GLOBAL SOLUTIONS
SGS PORT ANGELES DIVING, INC.

430 Marine Drive
Port Angeles, WA 98363
Tel: 360-457-9986
Fax: 360-457-9963
portangeles@sgsdiving.com

www.subseaglobalsolutions.com



Pier 66

Pile Inspection (Underwater)

REPORT CONTENTS

- **Job Summary / Observations**
- Recommendations
- Project Timeline
- Photo Log
- Technical Drawings

SGS Project Manager: Mark Horness

NW Engineering Representative: Valeriy Gubchak

Technical Report Author: Mark Horness

SEA STATE / WEATHER

Seas:	Calm
Current:	Moderate
Visibility:	2-3 Feet
Weather:	Sunny

JOB SUMMARY / OBSERVATIONS

SGS Port Angeles Diving was subcontracted to perform the underwater piling inspection of the P66 pier. Inspection started on Bent 58, moving shore side to Bent 13. Bent 12 and on had no submerged sections. Norwest Engineering had previously performed above water pile inspection. Pile that were visibly broken above water were not inspected, typically these were fender pile. For reference, pier was given upstream/downstream and channel/shore side to indicate pile without bent/row.

Floating dock and gangway had steel pile. All pile were found in good condition overall with surface corrosion observed near tidal zone. Securing chain found in good condition and free to move as designed. Adjacent small floating dock also had steel pile, these were found in good condition.

RECOMMENDATIONS FOR FUTURE ACTIONS

Following are recommended actions for the vessel based on the ship work performed for this job.
Some items are routine while others are specific and beneficial to the continual operations of the *Pier 66.*

- Address pile that have been reviewed by NW Engineering and need replacing.

Report prepared and written without prejudice. Underwater video and/or photography was utilized to document the findings/work. Please contact our office with any questions.

PILE CONDITION BREAKDOWN

Following shows pile condition per underwater inspection for *Pier 66*

Bent 58

- Fender –
- Row A – Timber, good.
- Batter A – Timber, good.
- Row B – Timber, good.
- Batter B – Timber, good.
- Row C – Timber, good.
- Batter C – Timber, good.
- Row D – Timber, good.
- Batter D – Timber, good.
- Row E – Timber, good.
- Batter E – Timber, good.
- Row F – Timber, good.
- Batter F – Timber, good.
- Fender –

Bent 57

- Fender –
- Row A – Timber, good.
- Batter A – Timber, good.
- Row B – Timber, good.
- Batter B – Timber, good.
- Row C – Timber, good.
- Batter C – Timber, good.
- Row D – Timber, good.
- Batter D – Timber, good.
- Row E – Timber, good.
- Batter E – Timber, good.
- Row F – Timber, good.
- Batter F – Timber, good.
- Fender –

Bent 56

- Fender –
- Row A – Timber, good.
- Batter A – Timber, good.
- Row B – Timber, good.
- Batter B – Timber, good.
- Row C – Timber, good.
- Batter C – Timber, good.
- Row D – Timber, good.
- Batter D – Timber, good.
- Row E – Timber, good.
- Batter E – Timber, good.
- Row F – Timber, good.
- Batter F – Timber, good.
- Fender –

Bent 55

- Fender –
- Row A – Timber, good.
- Batter A – Timber, good.
- Row B – Timber, good.
- Batter B – Timber, good.
- Row C – Timber, good.
- Batter C – Timber, good.
- Row D – Timber, good.
- Batter D – Timber, good.
- Row E – Timber, good.
- Batter E – Timber, good.
- Row F – Timber, good.
- Batter F – Timber, good.
- Fender –

Bent 54

- Fender –
- Row A – Timber, good.
- Batter A – Timber, good.
- Row B – Timber, good.
- Batter B – Timber, good.
- Row C – Timber, good.
- Batter C – Timber, good.
- Row D – Timber, good.
- Batter D – Timber, good.
- Row E – Timber, good.
- Batter E – Timber, good.
- Row F – Timber, good.
- Batter F – Timber, good.
- Fender –

Bent 53

- Fender –
- Row A – Timber, good.
- Batter A – Timber, good.
- Row B – Timber, good.
- Batter B – Timber, good.
- Row C – Timber, good.
- Batter C – Timber, good.
- Row D – Timber, good.
- Batter D – Timber, good.
- Row E – Timber, good.
- Batter E – Timber, good.
- Row F – Timber, good.
- Batter F – Timber, good.
- Fender –

Bent 52

- Fender –
- Row A – Timber, good.
- Batter A – Timber, good.
- Row B – Timber, good.
- Batter B – Timber, good.
- Row C – Timber, good.
- Batter C – Timber, good.
- Row D – Timber, good.
- Batter D – Timber, good.
- Row E – Timber, good.
- Batter E – Timber, good.
- Row F – Timber, good.
- Batter F – Timber, good.
- Fender –

Bent 51

- Fender –
- Row A – Timber, loose.
- Batter A – Timber, good.
- Row B – Timber, good.
- Batter B – Timber, good.
- Row C – Timber, damaged.
- Batter C – Timber, good.
- Row D – Timber, good.
- Batter D – Timber, good.
- Row E – Timber, good.
- Batter E – Timber, good.
- Row F – Timber, good.
- Batter F – Timber, good.
- Fender –

Bent 50

- Fender –
- Row A – Timber, good.
- Batter A – Timber, loose.
- Row B – Timber, damaged.
- Batter B – Timber, good.
- Row C – Timber, good.
- Batter C – Timber, good.
- Row D – Timber, good.
- Batter D – Timber, good.
- Row E – Timber, good.
- Batter E – Timber, good.
- Row F – Timber, good.
- Batter F – Timber, good.
- Fender –

Bent 49

- Fender – Steel, corrosion.
- Row A – Timber, good.
- Batter A – Timber, good.
- Row B – Timber, good.
- Batter B – Timber, good.
- Row C – Timber, good.
- Batter C – Timber, good.
- Row D – Timber, good.
- Batter D – Timber, good.
- Row E – Timber, good.
- Batter E – Timber, good.
- Row F – Timber, good.
- Batter F – Timber, good.
- Fender –

Bent 48

- Fender – Steel, corrosion.
- Row A – Timber, good.
- Batter A – Timber, good.
- Row B – Timber, good.
- Batter B – Timber, good.
- Row C – Timber, good.
- Batter C – Timber, good.
- Row D – Timber, good.
- Batter D – Timber, good.
- Row E – Timber, good.
- Batter E – Timber, good.
- Row F – Timber, good.
- Batter F – Timber, good.
- Fender –

Bent 47

- Fender – Steel, corrosion.
- Row A – Steel, corrosion.
- Batter A – Timber, good.
- Row B – Steel, corrosion.
- Batter B – Timber, good.
- Row C – Steel, good.
- Batter C – Timber, good.
- Row D – Steel, good.
- Batter D – Timber, good.
- Row E – Steel, corrosion.
- Batter E – Timber, good.
- Row F – Steel, good.
- Batter F – Timber, good.
- Fender –

Bent 46

- Fender –
- Row A – Timber, good.
- Batter A – Timber, good.
- Row B – Steel, corrosion.
- Batter B – Timber, good.
- Row C – Timber, good.
- Batter C – Timber, good.
- Row D – Timber, good.
- Batter D – Timber, good.
- Row E – Steel, corrosion.
- Batter E – Timber, good.
- Row F – Steel, corrosion.
- Batter F – Timber, good.
- Fender –

Bent 45

- Fender –
- Row A – Steel, good.
- Batter A – Steel, corrosion.
- Row B – Timber, good.
- Batter B – Timber, good.
- Row C – Timber, good.
- Batter C – Timber, good.
- Row D – Timber, good.
- Batter D – Timber, good.
- Row E – Timber, good.
- Batter E – Timber, good.
- Row F – Timber, good.
- Batter F – Timber, good.
- Fender –

Bent 44

- Fender –
- Row A – Timber, good.
- Batter A – Timber, good.
- Row B – Timber, good.
- Batter B – Timber, good.
- Row C – Timber, good.
- Batter C – Timber, good.
- Row D – Timber, good.
- Batter D – Timber, good.
- Row E – Timber, questionable.
- Batter E – Steel, corrosion.
- Row F – Timber, good.
- Batter F – Timber, good.
- Fender –

Bent 43

- Fender –
- Row A – Timber, good.
- Batter A – Steel, good.
- Row B – Timber, damaged.
- Batter B – Timber, good.
- Row C – Timber, good.
- Batter C – Timber, good.
- Row D – Timber, good.
- Batter D – Timber, good.
- Row E – Timber, damaged.
- Batter E – Timber, good.
- Row F – Timber, good.
- Batter F – Timber, good.
- Fender –

Bent 42

- Fender –
- Row A – Timber, good.
- Batter A – Timber, good.
- Row B – Timber, good.
- Batter B – Timber, good.
- Row C – Timber, good.
- Batter C – Timber, good.
- Row D – Timber, good.
- Batter D – Timber, good.
- Row E – Timber, good.
- Batter E – Timber, good.
- Row F – Timber, good.
- Batter F – Timber, good.
- Fender –

Bent 41

- Fender – Steel corrosion.
- Row A – Timber, good.
- Batter A – Timber, good.
- Row B – Timber, loose.
- Batter B – Timber, good.
- Row C – Timber, good.
- Batter C – Timber, good.
- Row D – Timber, good.
- Batter D – Timber, good.
- Row E – Timber, good.
- Batter E – Timber, good.
- Row F – Timber, good.
- Batter F – Timber, good.
- Fender – Steel, corrosion.

Bent 40

- Fender –
- Row A – Steel, good.
- Batter A – Timber, good.
- Row B – Timber, good.
- Batter B – Timber, good.
- Row C – Timber, good.
- Batter C – Timber, good.
- Row D – Timber, good.
- Batter D – Timber, good.
- Row E – Timber, good.
- Batter E – Timber, good.
- Row F – Steel, good.
- Batter F – Timber, good.
- Fender –

Bent 39

- Fender – Steel, corrosion.
- Row A – Timber, good.
- Batter A – Timber, good.
- Row B – Timber, good.
- Batter B – Timber, good.
- Row C – Timber, good.
- Batter C – Timber, good.
- Row D – Timber, good.
- Batter D – Timber, good.
- Row E – Steel, corrosion.
- Batter E – Timber, good.
- Large timber wedged between pile E & F. Timber on offshore side of pile F, and shore side of pile E. Timber was wedged firmly between pile, diver could not move. Timber was approx. 20’ L x 6’ in diameter.
- Row F – Steel, corrosion.
- Batter F – Timber, good.
- Fender –

Bent 38

- Fender –
- Row A – Timber, good.
- Batter A – Timber, good.
- Row B – Timber, good.
- Batter B – Timber, good.
- Row C – Timber, good.
- Batter C – Timber, good.
- Row D – Timber, good.
- Batter D – Timber, good.
- Row E – Timber, good.
- Batter E – Timber, good.
- Row F – Timber, good.
- Batter F – Timber, good.
- Fender – Steel, corrosion.

Bent 37

- Fender – Timber, deadhead.
- Row A – Timber, good.
- Batter A – Timber, good.
- Row B – Timber, good.
- Batter B – Timber, good.
- Row C – Timber, good.
- Batter C – Timber, good.
- Row D – Timber, good.
- Batter D – Timber, good.
- Row E – Timber, good.
- Batter E – Timber, good.
- Row F – Timber, good.
- Batter F – Timber, good.
- Fender – Steel, corrosion.

Bent 36

- Fender – Steel, corrosion.
- Row A – Steel, corrosion.
- Batter A – Timber, good.
- Row B – Timber, good.
- Batter B – Timber, good.
- Row C – Timber, good.
- Batter C – Timber, good.
- Row D – Timber, good.
- Batter D – Timber, good.
- Row E – Timber, good.
- Batter E – Timber, good.
- Row F – Steel, corrosion.
- Batter F – Timber, good.
- Fender – Steel, corrosion.

Bent 35

- Fender – Steel, corrosion.
- Row A – Timber, good.
- Batter A – Steel, corrosion.
- Row B – Timber, good.
- Batter B – Timber, good.
- Row C – Timber, good.
- Batter C – Timber, good.
- Row D – Timber, good.
- Batter D – Timber, good.
- Row E – Timber, good.
- Batter E – Timber, good.
- Row F – Timber, good.
- Batter F – Timber, good.
- Fender – Steel, corrosion.

Bent 34

- Fender – Timber, loose.
- Row A – Timber, good.
- Batter A – Timber, good.
- Row B – Timber, good.
- Batter B – Timber, good.
- Row C – Timber, good.
- Batter C – Timber, good.
- Row D – Timber, good.
- Batter D – Timber, good.
- Row E – Steel, good.
- Batter E – Timber, good.
- Row F – Timber, good.
- Batter F – Timber, good.
- Fender –

Bent 33

- Fender – Steel, corrosion.
- Row A – Timber, good.
- Batter A – Timber, good.
- Row B – Timber, good.
- Batter B – Timber, good.
- Row C – Timber, good.
- Batter C – Timber, good.
- Row D – Timber, good.
- Batter D – Timber, good.
- Row E – Timber, good.
- Batter E – Timber, good.
- Row F – Timber, good.
- Batter F – Timber, good.
- Fender – Steel, corrosion.

Bent 32

- Fender –
- Row A – Timber, good.
- Batter A – Timber, good.
- Row B – Timber, good.
- Batter B – Timber, good.
- Row C – Timber, good.
- Batter C – Timber, good.
- Row D – Timber, damaged.
- Batter D – Timber, good.
- Row E – Timber, good.
- Batter E – Timber, good.
- Row F – Timber, good.
- Batter F – Timber, good.
- Fender –

Bent 31

- Fender –
- Row A – Timber, good.
- Batter A – Timber, good.
- Row B – Timber, good.
- Batter B – Timber, good.
- Row C – Timber, good.
- Batter C – Timber, good.
- Row D – Timber, good.
- Batter D – Timber, good.
- Row E – Timber, good.
- Batter E – Timber, good.
- Row F – Timber, good.
- Batter F – Timber, good.
- Fender –

Bent 30

- Fender –
- Row A – Timber, good.
- Batter A – Timber, good.
- Row B – Timber, good.
- Batter B – Timber, good.
- Row C – Timber, good.
- Batter C – Timber, good.
- Row D – Timber, good.
- Batter D – Timber, good.
- Row E – Timber, good.
- Batter E – Timber, good.
- Row F – Timber, good.
- Batter F – Timber, good.
- Fender –

Bent 29

- Fender – Steel, corrosion.
- Row A – Timber, weathering at surface.
- Batter A – Timber, good.
- Row B – Timber, weathering at surface.
- Batter B – Timber, good.
- Row C – Timber, weathering at surface.
- Batter C – Timber, good.
- Row D – Timber, weathering at surface.
- Batter D – Timber, good.
- Row E – Timber, weathering at surface.
- Batter E – Timber, good.
- Row F – Steel, good.
- Batter F – Timber, good.
- Fender – Timber, weathering at surface.

Bent 28

- Fender – Timber, deadhead.
- Row A – Timber, good.
- Batter A – Timber, good.
- Row B – Timber, good.
- Batter B – Timber, good.
- Row C – Timber, good.
- Batter C – Timber, good.
- Row D – Timber, good.
- Batter D – Timber, good.
- Row E – Timber, good.
- Batter E – Timber, good.
- Row F – Timber, good.
- Batter F – Timber, good.
- Fender – Timber, weathering at surface.

Bent 27

- Fender – Timber, deadhead.
- Row A – Timber, good.
- Batter A – Timber, good.
- Row B – Timber, good.
- Batter B – Timber, good.
- Row C – Timber, good.
- Batter C – Timber, good.
- Row D – Timber, good.
- Batter D – Timber, good.
- Row E – Timber, good.
- Batter E – Timber, good.
- Row F – Timber, good.
- Batter F – Timber, good.
- Fender – Timber, weathering at surface.

Bent 26

- Fender – Timber, questionable.
- Row A – Timber, good.
- Batter A – Timber, good.
- Row B – Timber, good.
- Batter B – Timber, good.
- Row C – Timber, good.
- Batter C – Timber, good.
- Row D – Timber, good.
- Batter D – Timber, good.
- Row E – Timber, good.
- Batter E – Timber, good.
- Row F – Timber, good.
- Batter F – Timber, good.
- Fender – Timber, questionable.

Bent 25

- Fender – Timber, good.
- Row A – Steel, good.
- Batter A – Timber, good.
- Row B – Steel, good.
- Batter B – Steel, good.
- Row C – Nonexistent.
- Batter C – Timber, good.
- Row D – Nonexistent.
- Batter D – Timber, good.
- Row E – Timber, good.
- Batter E – Steel, good.
- Row F – Timber, good.
- Batter F – Timber, good.
- Fender – Timber, good.

Bent 24

- Fender – Timber, deadhead.
- Row A – Timber, good.
- Batter A – Timber, good.
- Row B – Timber, good.
- Batter B – Timber, good.
- Row C – Timber, good.
- Batter C – Timber, good.
- Row D – Timber, good.
- Batter D – Timber, good.
- Row E – Timber, good.
- Batter E – Timber, good.
- Row F – Timber, good.
- Batter F – Timber, good.
- Fender – Timber, good.

Bent 23

- Fender –
- Row A – Timber, good.
- Batter A – Timber, good.
- Row B – Steel, corrosion.
- Batter B – Timber, good.
- Row C – Nonexistent.
- Batter C – Timber, good.
- Row D – Nonexistent.
- Batter D – Timber, good.
- Row E – Steel, corrosion.
- Batter E – Timber, good.
- Row F – Timber, good.
- Batter F – Timber, good.
- Fender – Timber, deadhead.

Bent 22

- Row A – Steel, corrosion.
- Row C – Steel, corrosion.

Bent 21

- Row A – Steel, corrosion.
- Row B – Timber, good.
- Row C – Steel, corrosion.

Bent 20

- Row A – Timber, good.
- Row B – Timber, good.
- Row C – Steel, corrosion.

Bent 19

- Row A – Steel, corrosion.
- Batter A – Timber, good.
- Row B – Timber, good.
- Row C – Steel, corrosion.

Bent 18

- Row A – Steel, good.
- Batter A – Timber, good.
- Row C – Steel, corrosion.

Bent 17

- Row A – Steel, good.
- Batter A – Timber, good.
- Row C – Steel, good.

Bent 16

- Row A – Steel, good.
- Batter A – Timber, good.
- Row B – Timber, good.
- Row C – Timber, good.

Bent 15

- Row A – Steel, good.
- Row C – Steel, corrosion.

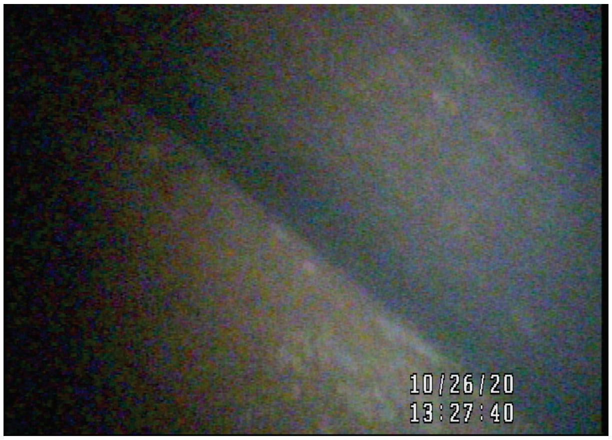
Bent 14

- Row A – Timber, good.
- Row B – Timber, good.
- Row C – Timber, good.

Bent 13

- Row A – Timber, good.
- Row B – Timber, good.
- Row C – Timber, good.

01



Typical Dolphin Cluster Pile Condition.

02

Typical Timber Pile Condition.

03

Pile 51 C - Above water.

04

Pile 51 C - Minor soft spot, approx. baseball size.

05

Pile 51 A - Loose at top.

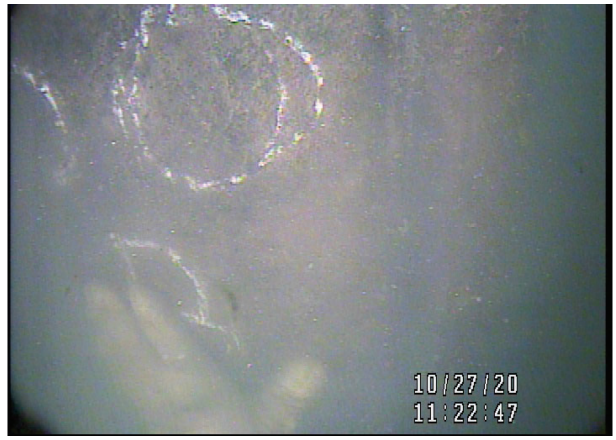
06

Pile 50 A - Loose at top.

07

Pile 50 B

08



Pile 50 B - Splitting observed.

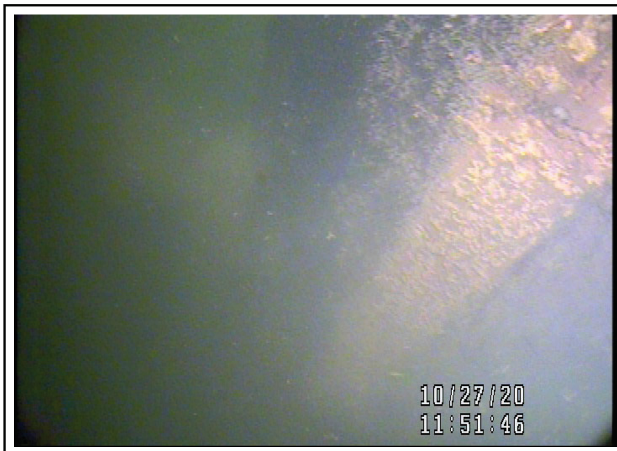
09

Pile 50 B - Splitting observed.

10

Pile 50 B - Splitting observed.

11



Pile 49 - Minor corrosion sighted.

12

Pile 49 - Minor corrosion sighted.

13

Fender Pile 48

14

Fender Pile 48 - Minor corrosion sighted.

15

Fender Pile 47

16

Fender Pile 47 - Minor corrosion sighted.

17

Pile 47 E

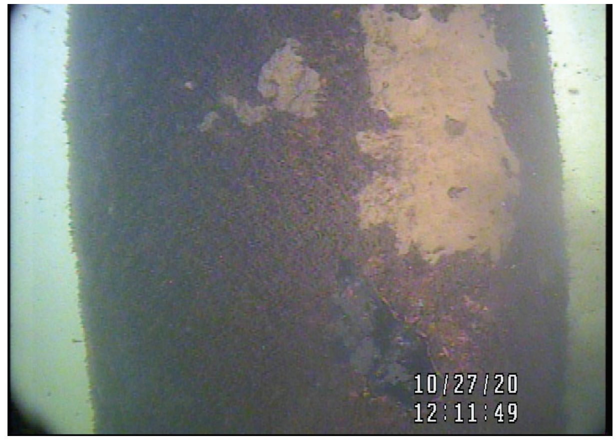
18

Pile 47 E - Minor corrosion sighted.

19

Pile 47 B

20



Pile 47 B - Minor corrosion sighted.

21

Pile 47 B - Minor corrosion sighted.

22

Pile 47 B - Minor corrosion sighted.

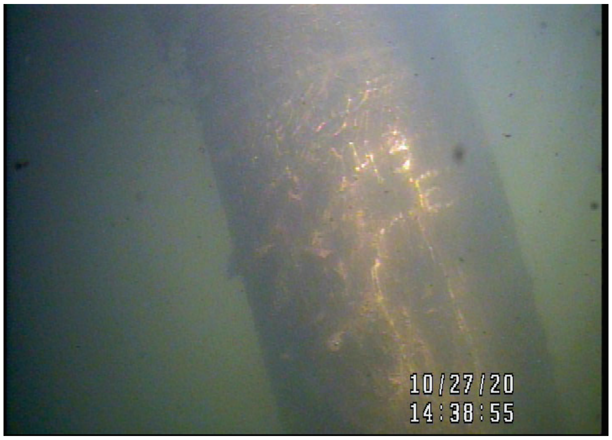
23

Pile 47 A

24

Pile 47 A - Minor corrosion sighted.

25



Pile 43 E - Timber compromise observed.

26

Pile 43 E - Timber compromise observed.

27

Fender Pile 41 - Minor corrosion sighted.

28

Fender Pile 41 - Minor corrosion sighted.

29

Fender Pile 41 Downstream side - Minor corrosion sighted.

30

Fender Pile 41 Downstream side - Minor corrosion sighted.

31

Pile 41 B - Loose at top.

32

Pile 41 B - Loose at top. No splitting observed at mudline.

33

Fender Pile 38 Upstream Side.

34

Fender Pile 38 - Minor corrosion sighted.

35

Fender Pile 33 Downstream side.

36

Fender Pile 33 - Minor corrosion sighted.

37

Pile 32 D

38

Pile 32 D - Splitting observed.

39

Pile 32 D - Weather checking observed.

40

Pile 32 D - Pocket of wood missing observed.

41

Pile 32 D - Pocket of wood missing observed.

42

Pile 32 D - Pocket of wood missing observed.

APPENDIX E:

INSPECTION LOGS

Norwest Engineering
Consulting Engineers

Phillips 66	CLIENT NAME
Portland, OR	TERMINAL NAME/LOCATION
Portland Dock Condition Assessment	PROJECT NAME
RV1192	PROJECT NUMBER

Initials: KW | Date: 10/8/2020 | Sheet 1 of 22

Pile Type: (Bearing / Batter, Fender) | Material: See Figures

LEVEL I/II INSPECTION FORM

LOCATION/GRID	L-II	L-III*	CONDITION RATING ND	MN	MD	MJ	SV	NOTES
3A				X				
3B				X				
3C				X				
4A				X				MAJOR FULL LENGTH SPLIT
4B				X				
4C				X				
5A				X				
5B				X				
5C				X				
6A				X				
6B				X				
6C				X				MINOR SPLIT FULL LENGTH (MID SECTION)
**6/7				X				MAJOR SPLIT THROUGH ENTIRE SECTION
7A				X				
7B				X				
7C				X				MAJOR SPLIT ENTIRE COLUMN (MID SECTION)

*Details on a separate form

General Notes: # = 1ST TAG * = RESISTOGRAPH ** = STRINGER LOCATED BETWEEN BENTS 6 AND 7, SECOND NEAREST TO UPSTREAM SIDE
(R) OFFSHORE SIDE OF BENT (S) ONSHORE SIDE OF BENT

1.. BROKEN BRACE BETWEEN A&B
2. LARGE CRACK IN BRACE BETWEEN B&C
3. LOWER CROSS BRACE SPLIT @ BOLT CONNECTION (BROKEN OFF)

Norwest Engineering Consulting Engineers	
	Phillips 66 CLIENT NAME
	Portland, OR TERMINAL NAME/LOCATION
	Portland Dock Condition Assessment PROJECT NAME
	RV1192 PROJECT NUMBER

Initials: KW	Date: 10/8/2020	Sheet 2	of 22
Pile Type: (Bearing / Batter, Fender)	Material See Figures	**LEVEL I/II INSPECTION FORM**	

LOCATION/GRID	L-II	L-III*	CONDITION RATING ND	MN	MD	MJ	SV	NOTES
8A				X				
8B				X				
8C				X				
9A				X				
9B					X			LARGE 1" SPLIT FULL LENGTH
9C				X				
10A				X				
10B				X				
10C				X				
11A				X				
11B					X			LARGE 1" SPLIT FULL LENGTH
11C					X			LARGE 1" SPLIT FULL LENGTH
12A					X			
*12B		X		X				MODERATE HOLLOW SOUNDINGS
12C				X				MINOR HOLLOW SOUNDINGS
13A				X				
13B				X				
13C				X				MINOR HOLLOW SOUNDINGS
14A				X				STEEL
14B				X				
14C				X				STEEL
#15A				X				
15B				X				MINOR CHECK
15C				X				
16A				X				STEEL
16B								REMOVED
16C				X				STEEL
17A				X				STEEL
17A BATTER				X				
*17B		X						MODERATE HOLLOW SOUNDINGS
17C				X				MINOR HOLLOW SOUNDINGS

*Details on a separate form

General Notes: # = 1ST TAG * = RESISTOGRAPH ** = STRINGER LOCATED BETWEEN BENTS 6 AND 7, SECOND NEAREST TO UPSTREAM SIDE

(R) OFFSHORE SIDE OF BENT (S) ONSHORE SIDE OF BENT

1.. BROKEN BRACE BETWEEN A&B

2. LARGE CRACK IN BRACE BETWEEN B&C

3. LOWER CROSS BRACE SPLIT @ BOLT CONNECTION (BROKEN OFF)

Norwest Engineering Consulting Engineers	Phillips 66 (CLIENT NAME)
	Portland, OR (TERMINAL NAME/LOCATION)
	Portland Dock Condition Assessment (PROJECT NAME)
	RV1192 (PROJECT NUMBER)

Initials: KW	Date: 10/8/2020	Sheet 3	of 22
Pile Type: (Bearing / Batter, Fender)	Material: See Figures	**LEVEL I/II INSPECTION FORM**	

LOCATION/GRID	L-II	L-III*	CONDITION RATING					NOTES
			ND	MN	MD	MJ	SV	
18A				X				STEEL
18A BATTER				X				MINOR HOLLOW SOUNDINGS
18B								REMOVED
18C				X				STEEL
18C FENDER				X				
19A				X				STEEL
19A BATTER				X				
19B				X				
19C				X				STEEL
20A				X				
20B				X				MINOR HOLLOW SOUNDINGS
20C				X				STEEL
21A				X				STEEL
21B				X				
21C				X				STEEL
22A				X				STEEL
22B								REMOVED
22C				X				STEEL
23A				X				
23A BATTER				X				
23B				X				STEEL
23B BATTER				X				
23C								REMOVED
23D								REMOVED
23E				X				STEEL
23E BATTER				X				
23F				X				
23F BATTER				X				
23F FENDER							X	COMPLETE BREAK @ WATER LEVEL/HANGING

*Details on a separate form

General Notes: # = 1ST TAG * = RESISTOGRAPH ** = STRINGER LOCATED BETWEEN BENTS 6 AND 7, SECOND NEAREST TO UPSTREAM SIDE
(R) OFFSHORE SIDE OF BENT (S) ONSHORE SIDE OF BENT

1.. BROKEN BRACE BETWEEN A&B
2. LARGE CRACK IN BRACE BETWEEN B&C
3. LOWER CROSS BRACE SPLIT @ BOLT CONNECTION (BROKEN OFF)

Norwest Engineering
Consulting Engineers

Phillips 66	CLIENT NAME
Portland, OR	TERMINAL NAME/LOCATION
Portland Dock Condition Assessment	PROJECT NAME
RV1192	PROJECT NUMBER

Initials: KW | Date: 10/8/2020 | Sheet 4 of 22

Pile Type: (Bearing / Batter, Fender) | Material: See Figures

LEVEL I/II INSPECTION FORM

LOCATION/GRID	L-II	L-III*	CONDITION RATING ND	MN	MD	MJ	SV	NOTES
24A FENDER							X	COMPLETE BREAK/ HOLLOW
24A				X				
24A BATTER				X				
24B				X				
24B BATTER				X				
24C				X				
24D				X				
24E				X				
24E BATTER				X				
24F				X				
24F BATTER				X				
24F FENDER (R)					X			HOLLOW SOUNDINGS/ CHECKING
24F FENDER (S)					X			HOLLOW SOUNDINGS/ CHECKING
25A FENDER					X			HOLLOW SOUNDINGS/ CHECKING
25A				X				STEEL
25A BATTER				X				STEEL
25B				X				STEEL
25B				X				STEEL
25C								REMOVED
25D								REMOVED
25E				X				STEEL
25E BATTER				X				STEEL
25F				X				STEEL
25F BATTER				X				STEEL
25F FENDER (R)							X	HEAVY HOLLOW SOUNDINGS/ CHECKING
25F FENDER (S)							X	COMPLETE BREAK @ WATERLINE

*Details on a separate form

General Notes: # = 1ST TAG * = RESISTOGRAPH ** = STRINGER LOCATED BETWEEN BENTS 6 AND 7, SECOND NEAREST TO UPSTREAM SIDE
(R) OFFSHORE SIDE OF BENT (S) ONSHORE SIDE OF BENT

1.. BROKEN BRACE BETWEEN A&B

2. LARGE CRACK IN BRACE BETWEEN B&C

3. LOWER CROSS BRACE SPLIT @ BOLT CONNECTION (BROKEN OFF)

Norwest Engineering Consulting Engineers

Phillips 66	CLIENT NAME
Portland, OR	TERMINAL NAME/LOCATION
Portland Dock Condition Assessment	PROJECT NAME
RV1192	PROJECT NUMBER

Initials: KW | Date: 10/8/2020 | Sheet 5 of 22

Pile Type: (Bearing / Batter, Fender) | Material: See Figures

LEVEL I/II INSPECTION FORM

LOCATION/GRID	L-II	L-III*	CONDITION RATING ND	MN	MD	MJ	SV	NOTES
26A FENDER					X			MN HOLLOW SOUNDINGS/ MD CHECKING
26A				X				
26A BATTER				X				
*26B		X						MINOR HOLLOW SOUNDINGS
26B BATTER				X				
26C				X				
26D				X				
26E				X				
26E BATTER				X				
26F				X				
26F BATTER				X				
26F FENDER							X	COMPLETE BREAK/ NEEDS REMOVAL
*27A		X						MINOR HOLLOW SOUNDINGS
27A BATTER				X				
27B				X				MINOR HOLLOW SOUNDINGS
27B BATTER				X				
27C				X				
27D				X				
27E				X				
27E BATTER				X				
27F				X				MODERATE CHECKING/ FULL VISIBLE LENGTH
27F BATTER				X				MODERATE CHECKING/ TOP 25'
27F FENDER							X	COMPLETE BREAK

*Details on a separate form

General Notes: # = 1ST TAG * = RESISTOGRAPH ** = STRINGER LOCATED BETWEEN BENTS 6 AND 7, SECOND NEAREST TO UPSTREAM SIDE

(R) OFFSHORE SIDE OF BENT (S) ONSHORE SIDE OF BENT

1.. BROKEN BRACE BETWEEN A&B

2. LARGE CRACK IN BRACE BETWEEN B&C

3. LOWER CROSS BRACE SPLIT @ BOLT CONNECTION (BROKEN OFF)

Norwest Engineering
Consulting Engineers

Phillips 66	CLIENT NAME
Portland, OR	TERMINAL NAME/LOCATION
Portland Dock Condition Assessment	PROJECT NAME
RV1192	PROJECT NUMBER

Initials: KW	Date: 10/8/2020	Sheet 6	of 22
Pile Type: (Bearing / Batter, Fender)	Material: See Figures	**LEVEL I/II INSPECTION FORM**	

LOCATION/GRID	L-II	L-III*	CONDITION RATING ND	MN	MD	MJ	SV	NOTES
28A				X				MINOR HOLLOW SOUNDINGS
28A BATTER				X				
*28B		X						MINOR HOLLOW SOUNDINGS
28B BATTER				X				
28C				X				
28D				X				
*28E		X						MODERATE HOLLOW SOUNDINGS
28E BATTER				X				
28F				X				MAJOR SPLIT UPPER SECTION
28F BATTER				X				
28F FENDER (R)				X				MODERATE HOLLOW SOUNDINGS/ CRACKING
28F FENDER (S)				X				MODERATE HOLLOW SOUNDINGS
29A FENDER				X				STEEL
29A				X				
29A BATTER				X				
29B				X				MINOR HOLLOW SOUNDINGS
29B BATTER				X				
29C				X				
29D				X				
29E				X				
29E BATTER				X				
29F				X				STEEL
29F BATTER				X				
29F FENDER (R)				X				MINOR HOLLOW SOUNDINGS
29F FENDER (S)				X				MINOR HOLLOW SOUNDINGS

*Details on a separate form

General Notes: # = 1ST TAG * = RESISTOGRAPH ** = STRINGER LOCATED BETWEEN BENTS 6 AND 7, SECOND NEAREST TO UPSTREAM SIDE
(R) OFFSHORE SIDE OF BENT (S) ONSHORE SIDE OF BENT
1.. BROKEN BRACE BETWEEN A&B
2. LARGE CRACK IN BRACE BETWEEN B&C
3. LOWER CROSS BRACE SPLIT @ BOLT CONNECTION (BROKEN OFF)

Norwest Engineering Consulting Engineers	Phillips 66 (CLIENT NAME)
	Portland, OR (TERMINAL NAME/LOCATION)
	Portland Dock Condition Assessment (PROJECT NAME)
	RV1192 (PROJECT NUMBER)

Initials: KW	Date: 10/8/2020	Sheet 7	of 22
Pile Type: (Bearing / Batter, Fender)	Material See Figures	**LEVEL I/II INSPECTION FORM**	

LOCATION/GRID	L-II	L-III*	CONDITION RATING ND	MN	MD	MJ	SV	NOTES
30A FENDER				X				STEEL
30A				X				STEEL
30A BATTER				X				STEEL
30B				X				
30B BATTER				X				
30C				X				
30D				X				
30E				X				
30E BATTER				X				
30F				X				STEEL
31A FENDER (R)				X				MODERATE CHECKING/ SOFT SPOTS
31A FENDER (S)				X				MODERATE CHECKING
31A				X				
31A BATTER				X				
31B				X				
31B BATTER				X				
31C				X				
31D				X				
*31E		X						SOFT SPOTS
31E BATTER				X				
31F				X				MODERATE CHECKING UPPER SECTION
31F BATTER				X				
31F FENDER					X			SOFT SPOTS/ ABRASION

*Details on a separate form

General Notes: # = 1ST TAG * = RESISTOGRAPH ** = STRINGER LOCATED BETWEEN BENTS 6 AND 7, SECOND NEAREST TO UPSTREAM SIDE
(R) OFFSHORE SIDE OF BENT (S) ONSHORE SIDE OF BENT

1.. BROKEN BRACE BETWEEN A&B
2. LARGE CRACK IN BRACE BETWEEN B&C
3. LOWER CROSS BRACE SPLIT @ BOLT CONNECTION (BROKEN OFF)

Norwest Engineering Consulting Engineers	Phillips 66 CLIENT NAME
	Portland, OR TERMINAL NAME/LOCATION
	Portland Dock Condition Assessment PROJECT NAME
	RV1192 PROJECT NUMBER

Initials: KW	Date: 10/8/2020	Sheet 8	of 22
Pile Type: (Bearing / Batter, Fender)	Material See Figures	**LEVEL I/II INSPECTION FORM**	

LOCATION/GRID	L-II	L-III*	CONDITION RATING ND	MN	MD	MJ	SV	NOTES
32A FENDER				X				STEEL
32A				X				MINOR CHECKING TOP SECTION
*32A BATTER		X						MODERATE HOLLOW SOUNDINGS
32B				X				
32B BATTER				X				
32C				X				
*32D		X						MODERATE HOLLOW SOUNDINGS
32E				X				
32E BATTER				X				
32F				X				
32F BATTER				X				
32F FENDER (R)				X				STEEL
33A FENDER				X				STEEL
33A				X				
33A BATTER				X				
33B				X				
33B BATTER				X				
33C				X				
33D				X				
33E				X				
33E BATTER				X				
33F				X				
33F BATTER				X				MINOR HOLLOW SOUNDINGS
33F FENDER				X				STEEL

*Details on a separate form

General Notes: # = 1ST TAG * = RESISTOGRAPH ** = STRINGER LOCATED BETWEEN BENTS 6 AND 7, SECOND NEAREST TO UPSTREAM SIDE
(R) OFFSHORE SIDE OF BENT (S) ONSHORE SIDE OF BENT

1.. BROKEN BRACE BETWEEN A&B
2. LARGE CRACK IN BRACE BETWEEN B&C
3. LOWER CROSS BRACE SPLIT @ BOLT CONNECTION (BROKEN OFF)

Norwest Engineering Consulting Engineers	Phillips 66 (CLIENT NAME)
	Portland, OR (TERMINAL NAME/LOCATION)
	Portland Dock Condition Assessment (PROJECT NAME)
	RV1192 (PROJECT NUMBER)

Initials: KW	Date: 10/8/2020	Sheet 9	of 22
Pile Type: (Bearing / Batter, Fender)	Material See Figures	**LEVEL I/II INSPECTION FORM**	

LOCATION/GRID	L-II	L-III*	CONDITION RATING ND	MN	MD	MJ	SV	NOTES
34A FENDER				X				
34A				X				
*34A BATTER		X						MODERATE HOLLOW SOUNDINGS
34B				X				
34B BATTER				X				MINOR HOLLOW SOUNDINGS
34C				X				
34D				X				
34E				X				
34E BATTER				X				
34F				X				
34F BATTER				X				
34F FENDER (R)				X				SOFT SPOTS
34F FENDER (S)				X				SOFT SPOTS
35A FENDER				X				STEEL
35A				X				STEEL
35A BATTER				X				
35B				X				
35B BATTER				X				
35C				X				
35D				X				
35E				X				
35E BATTER				X				MINOR HOLLOW SOUNDINGS/ MINOR CHECKING
35F				X				
35F BATTER				X				MINOR SECTION LOSS @ WATERLINE
35F FENDER				X				STEEL

*Details on a separate form

General Notes: # = 1ST TAG * = RESISTOGRAPH ** = STRINGER LOCATED BETWEEN BENTS 6 AND 7, SECOND NEAREST TO UPSTREAM SIDE
(R) OFFSHORE SIDE OF BENT (S) ONSHORE SIDE OF BENT

1.. BROKEN BRACE BETWEEN A&B
2. LARGE CRACK IN BRACE BETWEEN B&C
3. LOWER CROSS BRACE SPLIT @ BOLT CONNECTION (BROKEN OFF)

Norwest Engineering Consulting Engineers	Phillips 66 (CLIENT NAME)
	Portland, OR (TERMINAL NAME/LOCATION)
	Portland Dock Condition Assessment (PROJECT NAME)
	RV1192 (PROJECT NUMBER)

Initials: KW | Date: 10/8/2020 | Sheet 10 of 22

Pile Type: (Bearing / Batter, Fender) | Material: See Figures

LEVEL I/II INSPECTION FORM

LOCATION/GRID	L-II	L-III*	CONDITION RATING					NOTES
			ND	MN	MD	MJ	SV	
36A FENDER				X				STEEL
36A				X				STEEL
36B				X				
36B BATTER				X				
36C				X				
36D				X				MINOR CHECKING
36E				X				
36E BATTER				X				
36F				X				STEEL
36F BATTER				X				MINOR CHECKING
36F FENDER				X				STEEL
37A FENDER (R)							X	THROUGH CRACK
37A FENDER (S)							X	THROUGH CRACK
37A				X				
37A BATTER				X				
37B				X				
37B BATTER				X				
37C				X				
37D				X				
37E				X				
37E BATTER				X				
37F				X				
37F BATTER				X				
37F FENDER (R)				X				STEEL
37F FENDER (S)							X	PILE HEAD

*Details on a separate form

General Notes: # = 1ST TAG * = RESISTOGRAPH ** = STRINGER LOCATED BETWEEN BENTS 6 AND 7, SECOND NEAREST TO UPSTREAM SIDE

(R) OFFSHORE SIDE OF BENT (S) ONSHORE SIDE OF BENT

1.. BROKEN BRACE BETWEEN A&B

2. LARGE CRACK IN BRACE BETWEEN B&C

3. LOWER CROSS BRACE SPLIT @ BOLT CONNECTION (BROKEN OFF)

Norwest Engineering
Consulting Engineers

Phillips 66	CLIENT NAME
Portland, OR	TERMINAL NAME/LOCATION
Portland Dock Condition Assessment	PROJECT NAME
RV1192	PROJECT NUMBER

Initials: KW	Date: 10/8/2020	Sheet 11	of 22
Pile Type: (Bearing / Batter, Fender)	Material See Figures	**LEVEL I/II INSPECTION FORM**	

LOCATION/GRID	L-II	L-III*	CONDITION RATING ND	MN	MD	MJ	SV	NOTES
38A FENDER				X				STEEL
38A				X				
*38A BATTER		X		X				MINOR HOLLOW SOUNDINGS
38B				X				
38B BATTER				X				MODERATE CHECKING
38C				X				
38D				X				
38E				X				
38E BATTER				X				
38F				X				
38F BATTER								REMOVED
38F FENDER				X				STEEL
39A FENDER				X				STEEL
39A				X				
39A BATTER				X				
39B				X				
39B BATTER				X				
39C				X				
39D				X				
39E				X				MINOR HOLLOW SOUNDINGS/ MINOR CHECKING
39E BATTER				X				
39F				X				STEEL
39F BATTER								REMOVED
39F FENDER				X				STEEL

*Details on a separate form

General Notes: # = 1ST TAG * = RESISTOGRAPH ** = STRINGER LOCATED BETWEEN BENTS 6 AND 7, SECOND NEAREST TO UPSTREAM SIDE
(R) OFFSHORE SIDE OF BENT (S) ONSHORE SIDE OF BENT
1.. BROKEN BRACE BETWEEN A&B
2. LARGE CRACK IN BRACE BETWEEN B&C
3. LOWER CROSS BRACE SPLIT @ BOLT CONNECTION (BROKEN OFF)

Norwest Engineering
Consulting Engineers

Phillips 66	CLIENT NAME
Portland, OR	TERMINAL NAME/LOCATION
Portland Dock Condition Assessment	PROJECT NAME
RV1192	PROJECT NUMBER

Initials: KW | Date: 10/8/2020 | Sheet 12 of 22

Pile Type: (Bearing / Batter, Fender) | Material: See Figures

LEVEL I/II INSPECTION FORM

LOCATION/GRID	L-II	L-III*	CONDITION RATING ND	MN	MD	MJ	SV	NOTES
40A FENDER (R)						X		CRACK FAILURE
40A FENDER (S)				X				MODERATE HOLLOW SOUNDINGS
40A				X				
40A BATTER				X				
40B				X				
40B BATTER				X				MINOR HOLLOW SOUNDINGS
40C				X				
40D				X				
40E				X				
40E BATTER				X				
40F				X				
40F BATTER								REMOVED
40F FENDER (R)				X				
40F FENDER (S)				X				
41A FENDER				X				STEEL
41A				X				STEEL
41A BATTER				X				MINOR HOLLOW SOUNDINGS
41B				X				
41B BATTER								CUT OFF AT DECK LEVEL
41C				X				
41D				X				
41E				X				
41E BATTER				X				
41F				X				STEEL
41F BATTER								CUT OFF
41F FENDER				X				STEEL

*Details on a separate form

General Notes: # = 1ST TAG * = RESISTOGRAPH ** = STRINGER LOCATED BETWEEN BENTS 6 AND 7, SECOND NEAREST TO UPSTREAM SIDE
(R) OFFSHORE SIDE OF BENT (S) ONSHORE SIDE OF BENT
1.. BROKEN BRACE BETWEEN A&B
2. LARGE CRACK IN BRACE BETWEEN B&C
3. LOWER CROSS BRACE SPLIT @ BOLT CONNECTION (BROKEN OFF)

Norwest Engineering Consulting Engineers	
Phillips 66	CLIENT NAME
Portland, OR	TERMINAL NAME/LOCATION
Portland Dock Condition Assessment	PROJECT NAME
RV1192	PROJECT NUMBER

Initials: KW	Date: 10/8/2020	Sheet 13	of 22
Pile Type: (Bearing / Batter, Fender)	Material See Figures	**LEVEL I/II INSPECTION FORM**	

LOCATION/GRID	L-II	L-III*	CONDITION RATING ND	MN	MD	MJ	SV	NOTES
42A FENDER				X				STEEL
42A				X				
42A BATTER				X				
42B				X				
42B BATTER				X				
42C				X				
42D				X				
42E				X				
42E BATTER				X				MINOR HOLLOW SOUNDINGS TOP 3 FT
42F				X				STEEL
*42F BATTER		X						MINOR HOLLOW SOUNDING
43A FENDER (R)				X				MN SOFT SPOTS/ ABRAISION/MN SECTION LOSS @ WATERLINE
43A FENDER (S)				X				MN SOFT SPOTS/ ABRAISION/MN SECTION LOSS @ WATERLINE
43A				X				
43A BATTER				X				
43B				X				
43B BATTER				X				
43C				X				
43D				X				
43E				X				
43E BATTER				X				
43F				X				
43F BATTER				X				
43F FENDER (R)				X				SOFT SPOTS
43F FENDER (S)				X				SOFT SPOTS

*Details on a separate form

General Notes: # = 1ST TAG * = RESISTOGRAPH ** = STRINGER LOCATED BETWEEN BENTS 6 AND 7, SECOND NEAREST TO UPSTREAM SIDE
(R) OFFSHORE SIDE OF BENT (S) ONSHORE SIDE OF BENT
1.. BROKEN BRACE BETWEEN A&B
2. LARGE CRACK IN BRACE BETWEEN B&C
3. LOWER CROSS BRACE SPLIT @ BOLT CONNECTION (BROKEN OFF)

Norwest Engineering Consulting Engineers	Phillips 66 (CLIENT NAME)
	Portland, OR (TERMINAL NAME/LOCATION)
	Portland Dock Condition Assessment (PROJECT NAME)
	RV1192 (PROJECT NUMBER)

Initials: KW	Date: 10/8/2020	Sheet 14	of 22
Pile Type: (Bearing / Batter, Fender)	Material See Figures	**LEVEL I/II INSPECTION FORM**	

LOCATION/GRID	L-II	L-III*	CONDITION RATING ND	MN	MD	MJ	SV	NOTES
44A FENDER				X				STEEL
44A				X				STEEL
44A BATTER				X				
*44B		X		X				MINOR HOLLOW SOUNDINGS
44B BATTER				X				
44C				X				
44D				X				
44E				X				
44E BATTER				X				
44F				X				
44F BATTER				X				
44F FENDER				X				STEEL
45A FENDER				X				STEEL
45A				X				STEEL
*45A BATTER		X						MINOR HOLLOW SOUNDINGS
45B				X				STEEL
45B BATTER				X				
45C								REMOVED
45D								REMOVED
45E				X				STEEL
45E BATTER				X				
45F				X				STEEL
45F BATTER				X				STEEL
45F FENDER				X				STEEL

*Details on a separate form

General Notes: # = 1ST TAG * = RESISTOGRAPH ** = STRINGER LOCATED BETWEEN BENTS 6 AND 7, SECOND NEAREST TO UPSTREAM SIDE
(R) OFFSHORE SIDE OF BENT (S) ONSHORE SIDE OF BENT

1.. BROKEN BRACE BETWEEN A&B
2. LARGE CRACK IN BRACE BETWEEN B&C
3. LOWER CROSS BRACE SPLIT @ BOLT CONNECTION (BROKEN OFF)

Norwest Engineering Consulting Engineers	Phillips 66 (CLIENT NAME)
	Portland, OR (TERMINAL NAME/LOCATION)
	Portland Dock Condition Assessment (PROJECT NAME)
	RV1192 (PROJECT NUMBER)

Initials: KW	Date: 10/8/2020	Sheet 15	of 22
Pile Type: (Bearing / Batter, Fender)	Material See Figures	**LEVEL I/II INSPECTION FORM**	

LOCATION/GRID	L-II	L-III*	CONDITION RATING					NOTES
			ND	MN	MD	MJ	SV	
46A FENDER (R)				X				MINOR HOLLOW SOUNDINGS
46A FENDER (S)					X			MN HOLLOW SOUNDINGS/ MJ SPLIT UPPER SECTION
46A				X				
46A BATTER				X				
46B				X				STEEL
46C								REMOVED
46D								REMOVED
46E				X				STEEL
46E BATTER								REMOVED
46F				X				STEEL
*46F BATTER		X						MINOR HOLLOW SOUNDINGS @ LOWER WALKWAY
46F FENDER (R)				X				SOFT SPOTS
46F FENDER (S)				X				SOFT SPOTS
47A FENDER				X				STEEL
47A				X				STEEL
47A BATTER				X				
47B				X				STEEL
47B BATTER				X				
47C								REMOVED
47D								REMOVED
47E				X				STEEL
47E BATTER				X				
47F				X				
47F BATTER				X				
47F FENDER				X				STEEL

*Details on a separate form

General Notes: # = 1ST TAG * = RESISTOGRAPH ** = STRINGER LOCATED BETWEEN BENTS 6 AND 7, SECOND NEAREST TO UPSTREAM SIDE
(R) OFFSHORE SIDE OF BENT (S) ONSHORE SIDE OF BENT
1.. BROKEN BRACE BETWEEN A&B
2. LARGE CRACK IN BRACE BETWEEN B&C
3. LOWER CROSS BRACE SPLIT @ BOLT CONNECTION (BROKEN OFF)

Norwest Engineering Consulting Engineers	
	Phillips 66 — CLIENT NAME
	Portland, OR — TERMINAL NAME/LOCATION
	Portland Dock Condition Assessment — PROJECT NAME
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Initials: KW	Date: 10/8/2020	Sheet 16	of 22
Pile Type: (Bearing / Batter, Fender)	Material See Figures	**LEVEL I/II INSPECTION FORM**	

LOCATION/GRID	L-II	L-III*	CONDITION RATING ND	MN	MD	MJ	SV	NOTES
48A FENDER				X				STEEL
*48A		X		X				MINOR HOLLOW SOUNDINGS
48A BATTER				X				MINOR HOLLOW SOUNDINGS
48B				X				
48B BATTER				X				
48C				X				
48D				X				
48E				X				
48E BATTER				X				
48F				X				MINOR HOLLOW SOUNDINGS
48F BATTER				X				
48F FENDER				X				STEEL
49A FENDER (R)				X				MINOR HOLLOW SOUNDINGS
49A FENDER (S)				X				MINOR HOLLOW SOUNDINGS
49A				X				SEE GENERAL NOTE 1
49A BATTER				X				
49B				X				SEE GENERAL NOTE 1 & 2
49B BATTER				X				
49C				X				SEE GENERAL NOTE 2
*49D		X						MINOR HOLLOW SOUNDINGS
49E				X				
49E BATTER				X				
49F				X				
49F BATTER				X				
49F FENDER (R)							X	PILE HEAD
49F FENDER (S)							X	PILE HEAD

*Details on a separate form

General Notes: # = 1ST TAG * = RESISTOGRAPH ** = STRINGER LOCATED BETWEEN BENTS 6 AND 7, SECOND NEAREST TO UPSTREAM SIDE
(R) OFFSHORE SIDE OF BENT (S) ONSHORE SIDE OF BENT

1.. BROKEN BRACE BETWEEN A&B
2. LARGE CRACK IN BRACE BETWEEN B&C
3. LOWER CROSS BRACE SPLIT @ BOLT CONNECTION (BROKEN OFF)

Norwest Engineering
Consulting Engineers

Phillips 66	CLIENT NAME
Portland, OR	TERMINAL NAME/LOCATION
Portland Dock Condition Assessment	PROJECT NAME
RV1192	PROJECT NUMBER

Initials: KW	Date: 10/8/2020	Sheet 17	of 22
Pile Type: (Bearing / Batter, Fender)	Material: See Figures	**LEVEL I/II INSPECTION FORM**	

LOCATION/GRID	L-II	L-III*	CONDITION RATING ND	MN	MD	MJ	SV	NOTES
50A FENDER				X				STEEL
50A				X				
*50A BATTER		X						MINOR HOLLOW SOUNDINGS
50B				X				
50B BATTER				X				MODERATE CHECK @ WATER LEVEL
50C				X				
50D				X				
50E				X				
50E BATTER				X				
50F				X				
50F BATTER				X				MINOR HOLLOW SOUNDINGS
50F FENDER				X				STEEL
51A FENDER				X				STEEL
51A				X				STEEL
51A BATTER				X				
51B				X				
51B BATTER				X				
51C				X				
51D				X				
*51E		X						MINOR HOLLOW SOUNDINGS
51E BATTER				X				MINOR HOLLOW SOUNDINGS
51F				X				
51F BATTER				X				MINOR HOLLOW SOUNDINGS
51F FENDER				X				STEEL

*Details on a separate form

General Notes: # = 1ST TAG * = RESISTOGRAPH ** = STRINGER LOCATED BETWEEN BENTS 6 AND 7, SECOND NEAREST TO UPSTREAM SIDE
(R) OFFSHORE SIDE OF BENT (S) ONSHORE SIDE OF BENT
1.. BROKEN BRACE BETWEEN A&B
2. LARGE CRACK IN BRACE BETWEEN B&C
3. LOWER CROSS BRACE SPLIT @ BOLT CONNECTION (BROKEN OFF)

Norwest Engineering Consulting Engineers	Phillips 66 CLIENT NAME
	Portland, OR TERMINAL NAME/LOCATION
	Portland Dock Condition Assessment PROJECT NAME
	RV1192 PROJECT NUMBER

Initials: KW	Date: 10/8/2020	Sheet 18	of 22
Pile Type: (Bearing / Batter, Fender)	Material See Figures	**LEVEL I/II INSPECTION FORM**	

LOCATION/GRID	L-II	L-III*	CONDITION RATING ND	MN	MD	MJ	SV	NOTES
52A FENDER								REMOVED
52A				X				
52A BATTER				X				
52B				X				
52B BATTER				X				
52C				X				
52D				X				
52E				X				
52E BATTER				X				MINOR HOLLOW SOUNDINGS
52F				X				
52F BATTER				X				
52F FENDER (R)							X	PILE HEAD
52F FENDER (S)				X				
52.5A FENDER (S)							X	PILE HEAD
53A FENDER (R)							X	COMPLETE BREAK / HANGING
53A				X				MINOR HOLLOW SOUNDS
53A BATTER				X				
53B				X				
53B BATTER				X				
53C				X				
53D				X				
53E				X				
53E BATTER				X				
53F				X				
*53F BATTER		X						MINOR HOLLOW SOUNDINGS
53F FENDER (R)							X	MODERATE ABRAISION/THROUGH CRACK
53F FENDER (S)							X	MODERATE ABRAISION/THROUGH CRACK

*Details on a separate form

General Notes: # = 1ST TAG * = RESISTOGRAPH ** = STRINGER LOCATED BETWEEN BENTS 6 AND 7, SECOND NEAREST TO UPSTREAM SIDE

(R) OFFSHORE SIDE OF BENT (S) ONSHORE SIDE OF BENT

1.. BROKEN BRACE BETWEEN A&B

2. LARGE CRACK IN BRACE BETWEEN B&C

3. LOWER CROSS BRACE SPLIT @ BOLT CONNECTION (BROKEN OFF)

Norwest Engineering Consulting Engineers	Phillips 66 (CLIENT NAME)
	Portland, OR (TERMINAL NAME/LOCATION)
	Portland Dock Condition Assessment (PROJECT NAME)
	RV1192 (PROJECT NUMBER)

Initials: KW	Date: 10/8/2020	Sheet 19	of 22
Pile Type: (Bearing / Batter, Fender)	Material See Figures	**LEVEL I/II INSPECTION FORM**	

LOCATION/GRID	L-II	L-III*	CONDITION RATING ND	MN	MD	MJ	SV	NOTES
54A FENDER				X				MODERATE CHECKING/MINOR HOLLOW SOUNDINGS
*54A		X						MINOR HOLLOW SOUNDINGS
54A BATTER				X				
*54B		X						MINOR HOLLOW SOUNDINGS
54B BATTER				X				
54C				X				MINOR HOLLOW SOUNDINGS
54D				X				
54E				X				
54E BATTER				X				
54F				X				
54F BATTER				X				
54F FENDER (R)							X	PILE CAP
54F FENDER (S)							X	PILE CAP
55A FENDER								REMOVED
55A				X				SEE GENERAL NOTE 1
55A BATTER				X				REMOVED
55B				X				SEE GENERAL NOTE 1
55B BATTER				X				
55C				X				
55D				X				
55E				X				
55E BATTER				x				
55F				X				
55F BATTER				X				
55F FENDER (R)							X	PILE HEAD
55F FENDER (S)							X	PILE HEAD

*Details on a separate form

General Notes: # = 1ST TAG * = RESISTOGRAPH ** = STRINGER LOCATED BETWEEN BENTS 6 AND 7, SECOND NEAREST TO UPSTREAM SIDE
(R) OFFSHORE SIDE OF BENT (S) ONSHORE SIDE OF BENT
1.. BROKEN BRACE BETWEEN A&B
2. LARGE CRACK IN BRACE BETWEEN B&C
3. LOWER CROSS BRACE SPLIT @ BOLT CONNECTION (BROKEN OFF)

Norwest Engineering Consulting Engineers	Phillips 66 CLIENT NAME
	Portland, OR TERMINAL NAME/LOCATION
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	RV1192 PROJECT NUMBER

Initials: KW	Date: 10/8/2020	Sheet 20	of 22
Pile Type: (Bearing / Batter, Fender)	Material See Figures	LEVEL I/II INSPECTION FORM	

LOCATION/GRID	L-II	L-III*	CONDITION RATING ND	MN	MD	MJ	SV	NOTES
56A FENDER (R)							X	BREAK @ WATERLINE / HANGING
56A FENDER (S)							X	COMPLETE BREAK @WATERLINE
56A				X				
56A BATTER				X				
56B				X				
56B BATTER				X				
56C				X				
56D				X				
56E				X				
56E BATTER				X				
56F				X				SEE GENERAL NOTE 3
56F BATTER				X				
56F FENDER (R)							X	PILE HEAD
56F FENDER (S)							X	PILE HEAD
57A				X				
57A BATTER				X				
57B				X				
57B BATTER				X				
57C				X				
57D				X				
57E				X				
57E BATTER				X				
57F				X				
57F BATTER				X				
58A				X				
58B				X				
58C				X				

*Details on a separate form

General Notes: # = 1ST TAG * = RESISTOGRAPH ** = STRINGER LOCATED BETWEEN BENTS 6 AND 7, SECOND NEAREST TO UPSTREAM SIDE
(R) OFFSHORE SIDE OF BENT (S) ONSHORE SIDE OF BENT

1.. BROKEN BRACE BETWEEN A&B
2. LARGE CRACK IN BRACE BETWEEN B&C
3. LOWER CROSS BRACE SPLIT @ BOLT CONNECTION (BROKEN OFF)

Norwest Engineering Consulting Engineers	
Phillips 66	CLIENT NAME
Portland, OR	TERMINAL NAME/LOCATION
Portland Dock Condition Assessment	PROJECT NAME
RV1192	PROJECT NUMBER

Initials: KW	Date: 10/8/2020	Sheet 21	of 22
Pile Type: (Bearing / Batter, Fender)	Material See Figures	**LEVEL I/II INSPECTION FORM**	

LOCATION/GRID	L-II	L-III*	CONDITION RATING ND	MN	MD	MJ	SV	NOTES
57A - 58A			ND	MN	MD	MJ	X	(4) FENDER PILE GROUPS
Notes MULTIPLE PILE HEADS AND COMPLETE BREAKS NEAR WATERLINE								
57E - 58E			ND		X	MJ	SV	TOP OF PILE
Notes VISIBLE ROT AT TOP OF PILE BENEATH COLLAPSED DECK BOARDS								
58D - 58E			ND	MN	MD	X	SV	PILE CAP
Notes SPLIT DAMAGE								
57F - 58F			ND	MN	MD	MJ	X	(4) FENDER PILE GROUPS
Notes COMPLETE FAILURE / BROKEN AT WATERLINE								
LOC/GRID			ND	MN	MD	MJ	SV	
Notes								
LOC/GRID			ND	MN	MD	MJ	SV	
Notes								
LOC/GRID			ND	MN	MD	MJ	SV	
Notes								
LOC/GRID			ND	MN	MD	MJ	SV	
Notes								
LOC/GRID			ND	MN	MD	MJ	SV	
Notes								
LOC/GRID			ND	MN	MD	MJ	SV	
Notes								
LOC/GRID			ND	MN	MD	MJ	SV	
Notes								

*Details on a separate form

General Notes:

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	Portland, OR (TERMINAL NAME/LOCATION)
	Portland Dock Condition Assessment (PROJECT NAME)
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Superstructure/Substructure	Area: Bollards	**LEVEL I/II INSPECTION FORM**

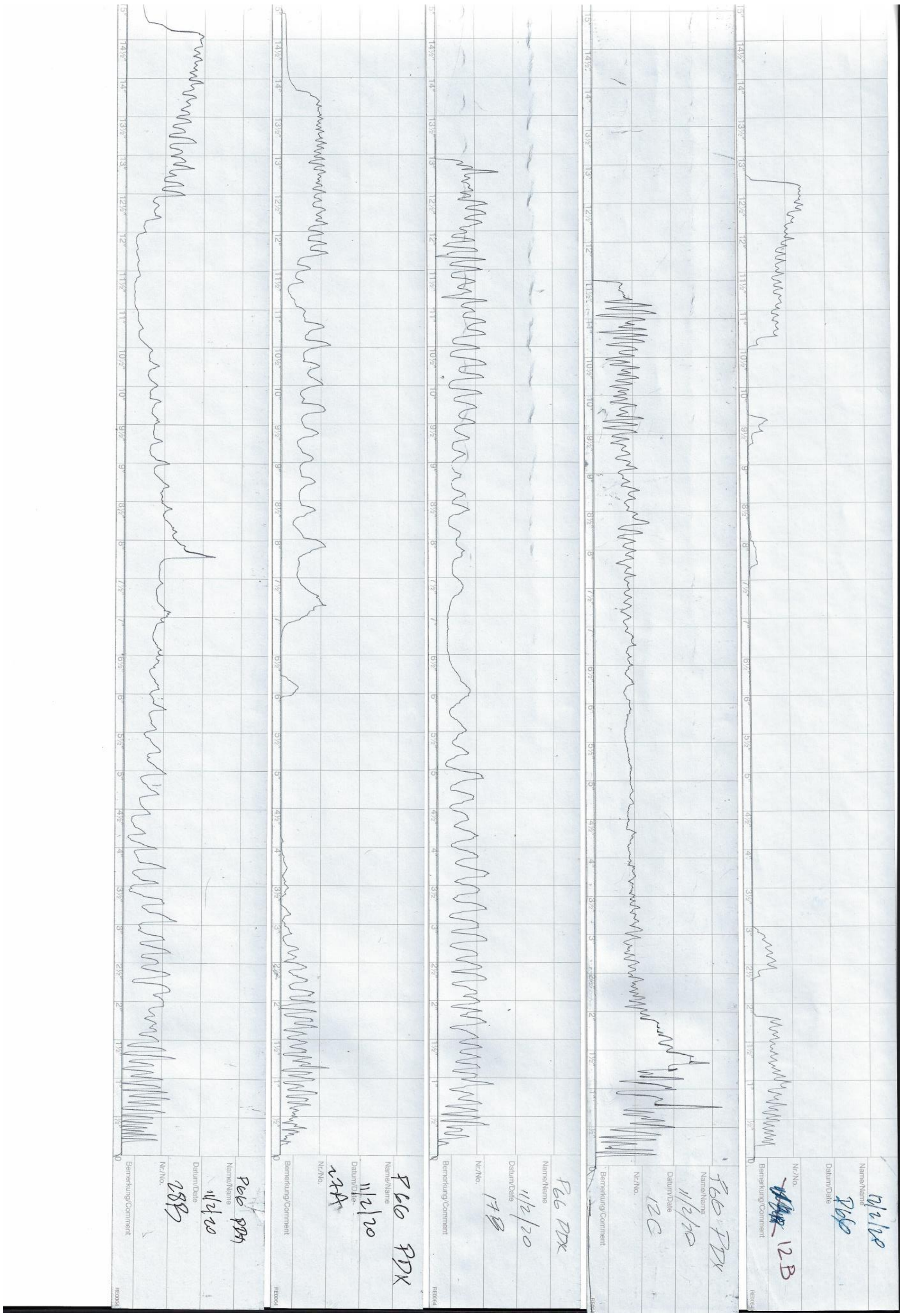
LOCATION/ID	ELEMENT TYPE	L-III*	CONDITION RATING					NOTES
			ND	MN	MD	MJ	SV	
A	DBL. BITT		X					
B	DBL. BITT			X				SCALE, ABRASION AT BASE (LINE RUB)
C	DBL. BITT			X				SCALE, ABRASION AT BASE (LINE RUB)
D	SINGLE BITT			X				
E	DBL. BITT		X					LOOSE BOLTS
F	SINGLE BITT		X					
G	DBL. BITT				X			SCALE, CORROSION
H	DBL. BITT		X					
J	DBL. BITT			X				
K	DBL. BITT			X				
L	DBL. BITT			X				
M	DBL. BITT		X					
N	DBL. BITT			X				

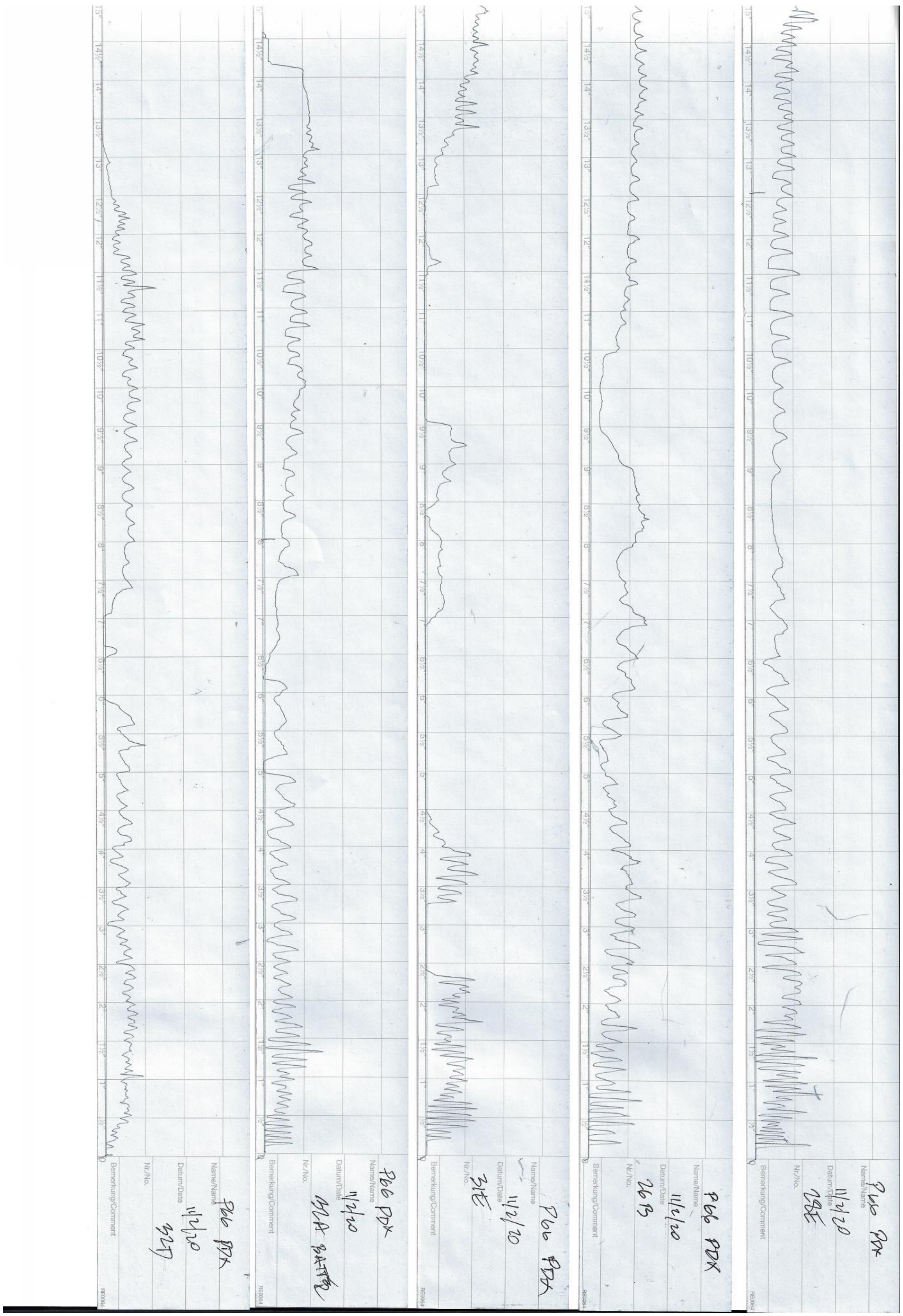
*Details on a separate form

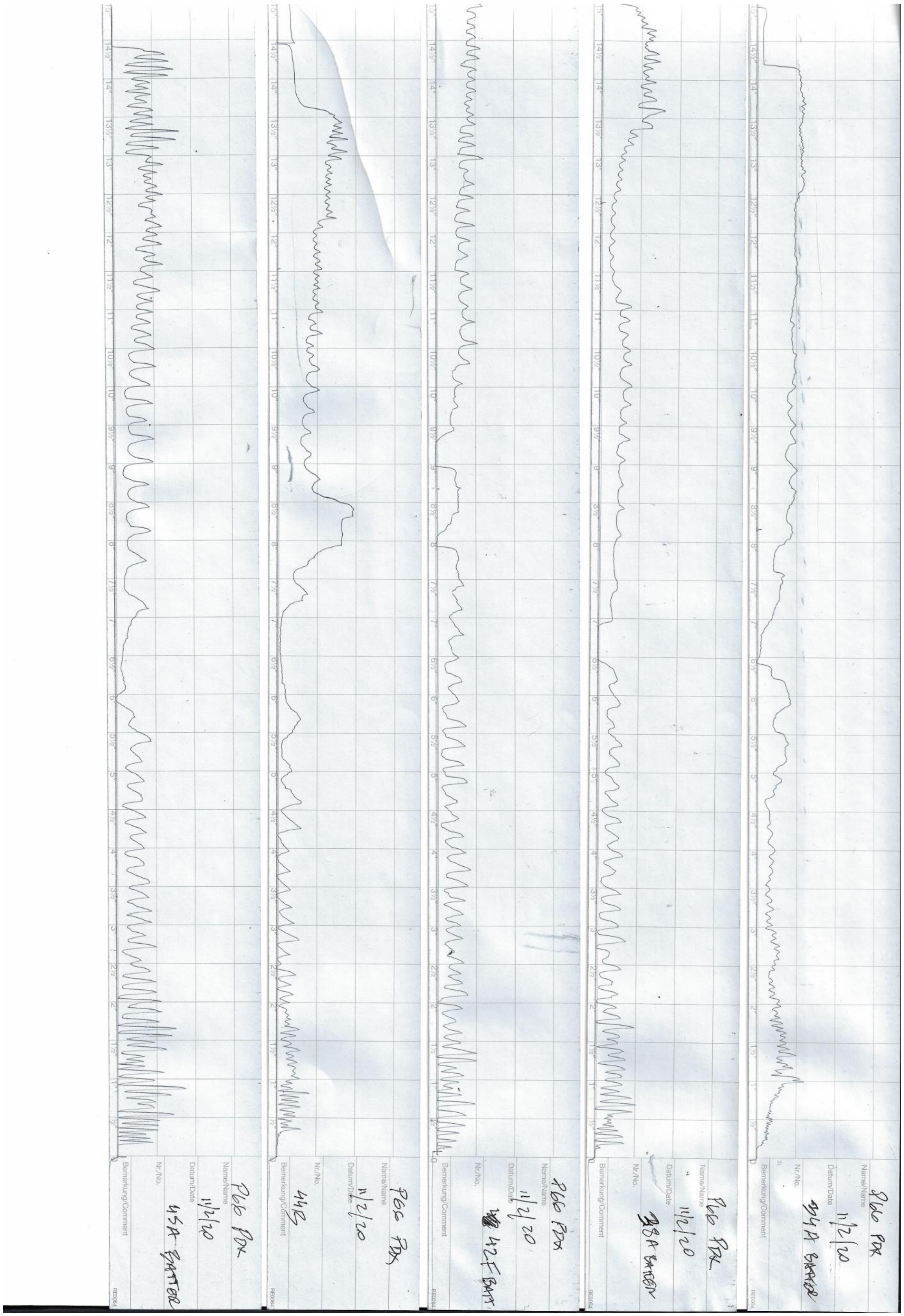
General Notes:

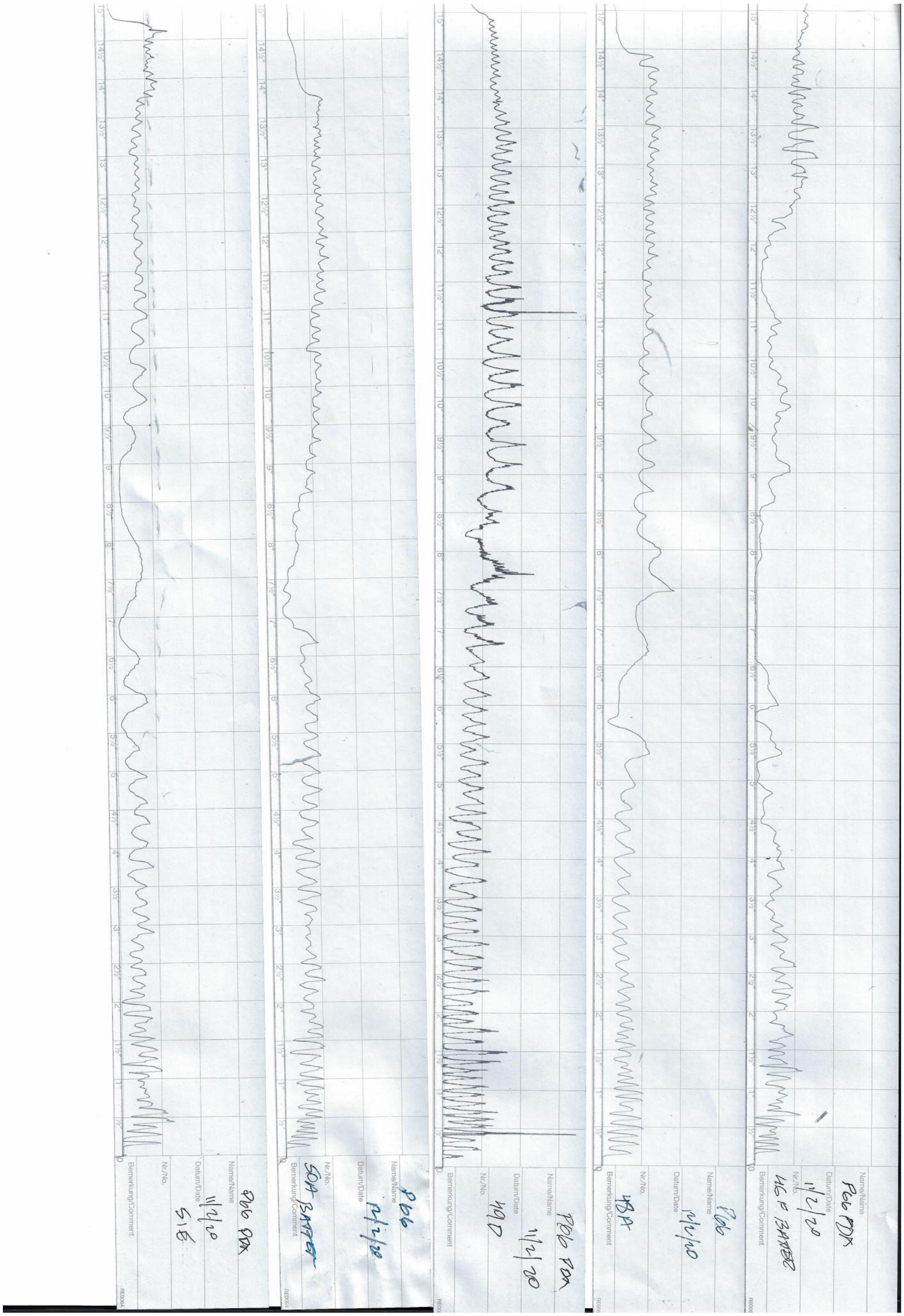
APPENDIX F:

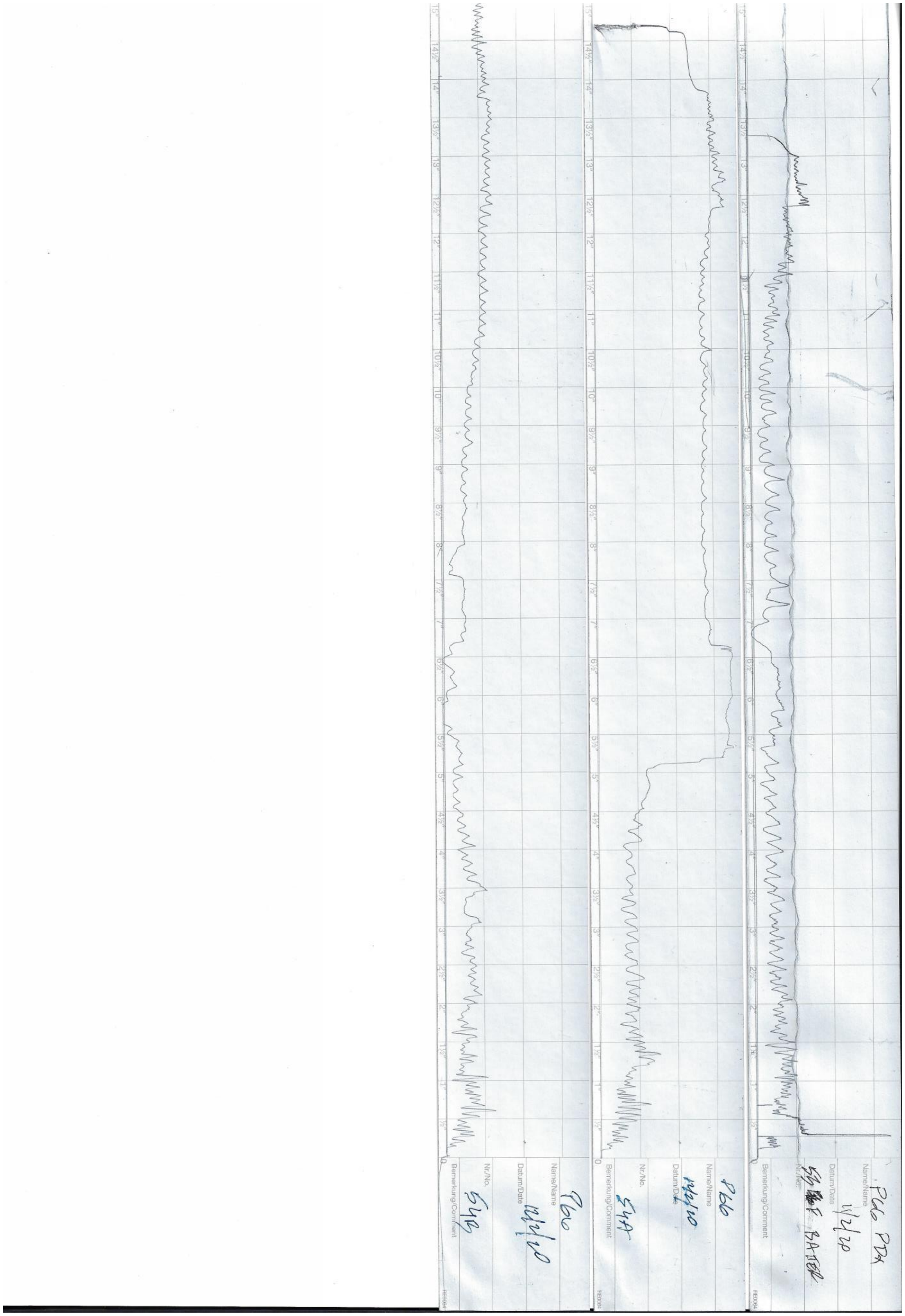
RESISTOGRAPH TESTING RESULTS


Name/Name P66 PDX
Datum/Date 11/2/20
Nr./No. 28B
Bemerkung/Comment
Name/Name P66 PDX
Datum/Date 11/2/20
Nr./No. 27A
Bemerkung/Comment
Name/Name P66 PDX
Datum/Date 11/2/20
Nr./No. 17B
Bemerkung/Comment
Name/Name P66 PDX
Datum/Date 11/2/20
Nr./No. 12C
Bemerkung/Comment
Name/Name 11/2/20
Datum/Date P66
Nr./No. 12B
Bemerkung/Comment









Appendix E
Berms and Dikes

E. Form 6: Berms and Dikes Surrounding Tank Farms (BER)

E.1 BER1

1. *Does the geotechnical investigation or report show any variation between each length (all sides) of the berm/dike? Is the berm susceptible to settlement or liquefaction? Can the protective layer remain intact under differential settlement or liquefaction conditions?*

Most of the tank farms are surrounded by cast-in-place concrete secondary containment walls. A portion of Tank Farm 3 is surrounded by an earthen berm. The seismic characteristics of the earthen berm are discussed in the geotechnical engineering report.

It is understood that the majority of the site soil is susceptible to liquefaction and liquefaction-induced settlement. Given the age of the cast-in-place concrete secondary containment walls, which are many decades old, it is unlikely they were designed considering the site's seismic liquefaction potential. At this time, it is understood that the site's liquefaction potential has only been evaluated by the geotechnical engineer using high-level empirical formulations. Additional engineering work must be done to more precisely understand the magnitude of total and differential vertical settlement and lateral deformation caused by seismic soil liquefaction. Once the precise magnitude of the site's total and differential seismic vertical settlement and lateral deformation is understood, the concrete secondary containment walls can be evaluated for their ability to accommodate the settlement.

E.2 BER2

2. *Are there cracks in the concrete? For LNG tanks, concrete dikes are required. What is the general condition; is there any exposed rebar? Provide cross-sectional drawings to facilitate the review and evaluate fitness-for-purpose, including the evaluation of seismic loads (demands and capacity) calculations including possible overturning, stability, and potential for differential settlement. Provide the age of the dike and all structural properties.*

The cumulative length of the site's cast-in-place concrete secondary containment walls exceeds 4,300 linear feet. A comprehensive condition assessment of the site's secondary containment walls, including presence or lack of concrete cracking and condition of reinforcing steel, has not been undertaken at this time. In addition, record drawings have not been available for review. Given the age of the cast-in-place concrete secondary containment walls, which are many decades old, the walls would not have been designed using commensurate seismic loads to those that are used in the current adopted building code. The precise age of the site's concrete secondary containment walls is not known at this time.

E.3 BER3

3. *Are there any penetrations for piping or drainage – explain and sketch.*

The cumulative length of the site's cast-in-place concrete secondary containment walls exceeds 4,300 linear feet. A comprehensive assessment of all penetrations through the concrete walls has not been undertaken at this time. Given known information at this time, there does not appear to be a significant quantity of widespread penetrations through the concrete walls. For example, most piping is either above grade or buried and generally does not penetrate through the concrete walls. Additional assessment must be undertaken to catalog locations of penetrations through the walls where they may exist.

E.4 BER4

4. *Is there evidence of water ponding at the base. If the tank farm is on a slope, is the downslope dike length and volume sufficient to facilitate the maximum postulated tank farm leakage?*

The tank farm areas are generally flat with only slight slopes. Based on the Phillips 66 Portland Terminal and Lubricants Spill Prevention and Control and Countermeasure (SPCC) Plan dated August 2020, each of the major tank farms have sufficient containment system capacity to accommodate the entirety of the product from the single largest tank within their containment area or 10-percent of the cumulative total volume of products within their containment area. Each tank farm containment area also has catch basins that are connected to the site's process water systems.

E.5 BER5

5. *Is the design sufficient (demand and capacity) to be fit-for-purpose post DE event? Will the potential leakage be contained post-earthquake or fire.*

The site's cast-in-place concrete secondary containment walls are many decades old and would not have been designed using commensurate seismic loads to those that are used in the current adopted building code. In addition, the site is susceptible to soil liquefaction as is described in the geotechnical engineering report. Additional assessment is required by the geotechnical engineer to more precisely calculate the site's expected vertical settlement and lateral deformation due to soil liquefaction. In addition, additional assessment is required to understand the potential for leaking of the secondary containment walls.

E.6 BER6

6. *What is the plan to evacuate the spillage after an event?*

The fuel terminal has a detailed Spill Prevention and Control and Countermeasure (SPCC) Plan. The most recent version of this document is dated August 2020. This document describes the “countermeasures” that would take place given a spill that includes, among other things, ensuring the safety of citizens and response personnel, managing a coordinated response effort, containing and recovering spilled material, recovery and rehabilitation of injured wildlife and removal of oil from impacted areas. The oil spill contingency plan is part of a separate referenced document entitled the Integrated Contingency Plan.

E.7 BER7

7. *Are there any penetrations, pipelines or other possible openings in the dike. Are there rigid pipeline penetrations that might rupture during seismic displacement?*

Please see the response to BER3.

E.8 BER8

8. *Any evidence of other damage to the existing dike, and does it satisfy the DE requirements of OAR 340-300-0003.*

Please see the response to BER2.

E.9 BER9

9. *Are the secondary containment systems designed to withstand the effects of the Maximum Considered Earthquake ground motion when empty and two-thirds of the Maximum Considered Earthquake ground motion when full, including all hydrodynamic forces per ASCE 7-22 Section 15.6.5.*

No. The site’s cast-in-place concrete secondary containment walls are many decades old and would not have been designed using commensurate seismic loads to those that are used in the current adopted building code.

Appendix F
Building and Building Structures

F. Form 7: Buildings and Building-Like Structures (BLG)

F.1 BLG1

1. *Obtain all structural drawings, calculations, geotechnical reports and possible damage reports for each structure. If no drawings or sets of relevant calculations exist, prepare a "baseline inspection" set of drawings used for the seismic evaluation (See Section 3.2 [of ASCE 41]). Provide the building type, per Table 3-1*
   - *W = Wood*
   - *S = Steel*
   - *CFS = Light Steel*
   - *C = Concrete*
   - *PC = Precast Concrete*
   - *RM = Reinforced Masonry*

The facility's site has three buildings on it that serve a product storage or handling function. These buildings include the Lube Blending and Packaging Warehouse, the Lube Oil Annex, and the Boiler House. The approximate floor area of each building is 56,000 square feet, 10,100 square feet, and 4,000 square feet, respectively. Existing structural drawings, calculations, and geotechnical reports have not been able to be obtained at this time.

It is believed that the Lube Blending and Packaging Warehouse is a steel frame structure (S). It is believed that the Lube Oil Annex is a pre-engineered metal building (S3, metal building frame). It is believed that the Boiler House is a reinforced masonry structure (RM).

Additional engineering work must be undertaken to review existing building structural drawings, understand the building construction types and construction details and understand the potential vulnerabilities of these buildings.

F.2 BLG2

2. *From Table 2-1, determine the structural performance category, either S-1 and S-2 to comply with OAR 340-300-0003 and the mitigation plan requirements to satisfy Risk Category IV, which satisfy the intent of OAR 340-300-0004(1)(a). The non-structural performance requirement should remain "operational", category "N-A" and have an importance factor Ip = 1.5. Per DEQ, the risk category is IV (Table 2-3). The BSE-1E cited is based on 20% in 50 years, but the DEQ requires the DE (2475-year return period). For the different types of structures, Table 3-4 provides references for the seismic evaluation and retrofit of structures (Risk Category IV). Use the appropriate ASCE/FEMA references. Provide criteria for Risk Category IV for this specific type of structure. From Table 2-1, Risk Category IV, BSE-1N states that for non-structural components, use 1-A and for BSE-2N, use 3-D.*

In accordance with OAR 340-300-0003(1)(f), it is anticipated that the building assessment will evaluate the potential for a spill greater than the MAUS during or after the Design Level

Earthquake. With the exception of some piping that penetrates the envelope of some of the buildings, the building superstructures generally do not support product or product storage. The storage of product is only supported by the ground floor systems of the buildings. As such, it is anticipated that the collapse potential for the buildings will be evaluated in accordance with the OAR 340-300-0003(1)(f) to evaluate whether the performance objective is met.

F.3 BLG3

3. *The scope of the investigation or inspection is described in Section 4.2 for each specific structural type. The table 4-1 for Tier 1 evaluations delineates areas to inspect and report for each type of building. Use Chapter 17 tables for the appropriate structural configuration and risk level IV to respond to each relevant question.*

ASCE 41 Tier 1 evaluations of the buildings have not been able to be completed yet. It is anticipated that ASCE 41 Tier 1 evaluations will be a component of the analysis assessing whether the existing buildings meet the OAR 340-300 performance objective.

F.4 BLG4

4. *For Tier 1 evaluation, Chapter 4 prescribes the procedure. Table 4-1 provides the direction for the structural inspection. Tier 1 checklist is in Chapter 17.*

Please see response to BLG3.

F.5 BLG5

5. *If Tier 2 is required, it includes analyses to determine the seismic capacity and demand, but using the deficiencies already reported in Tier 1. Procedure is to follow the flowchart in Figure 5-1. Chapter 7 prescribes analyses methodologies following Tier 2 evaluation.*

ASCE 41 Tier 1 evaluations of the buildings have not been able to be completed yet. It is not yet known if Tier 2 procedures will be needed.

Appendix G
Fire Detection and Suppression

ALPHA TECHNICAL GROUP INC.
Consulting Engineers
2929 N.W. 29th Avenue, Portland, OR. 97210
Phone (503) 227-3317   Fax (503) 227-3244

P66 Pipeline LLC- Portland Terminal
Seismic Vulnerability Assessment (SVA)
Fire Protection

Rev Date: May 10, 2024

REGISTERED PROFESSIONAL
ENGINEER
15471
OREGON
JULY 25, 1991
JOHN DEPPA
EXPIRES 12/31/25

PRELIMINARY

Table of Contents

Appendix

1.0 Introduction & Scope

This report summarizes the results of a Seismic Vulnerability Assessment (SVA) of the P66 Pipeline's Portland Terminal's Fire Protection Systems. This assessment is intended to directly address the requirements outlined in the recently adopted Oregon DEQ "Fuel Tank Seismic Stability Rules" OAR 300. Alpha Technical Group Inc (ATGI) designed a major upgrade to the Terminals Petroleum Fire Protection System which was constructed in 2008. Building and Fire Permits were obtained from the City of Portland at that time. A Butane Blending & Tank was installed in approximately 2018 along with additional Fire system improvements and Building and Fire Permits were obtained from the City of Portland.

2.0 Fire Protection Systems & Operations Overview

The main Terminal Petroleum Fire Pump/Foam System provides protection for petroleum products stored at Tank Farms 1/2 and the Main Truck Load Rack (located on Doane Ave). A separate Fire Water/Sprinkler System protects the warehouse which stores lubricant products.

3.0 Seismic Risk Assessment & Methodology

The flammable product storage at Tank Farms 1/2/3, loading at the Main Truck Load Rack, and unloading/storage of butane presents the highest risks from a fire protection perspective.

<u>3.1 Assessment of Fire Protection System(s) functionality during Design Earthquake Event:</u>
This assessment will assess the ability of the Fire Protection System(s) to function during the design earthquake event and limit any damage to the Terminal. This is important to protect life safety as well as ensure that the Terminal can get back online as soon as possible in order to provide fuel for emergency services. This assessment will be conducted by John Deppa PE and P66 Operations Staff using Process Hazard Analysis (PHA) procedures. This assessment will be completed after further geotechnical investigations are completed by Geo-Engineers Inc.-HOLD.

<u>3.2 Assessment of Fire Protection Pipe & Supports during Design Earthquake Event:</u>
This assessment will evaluate the main terminal Fire Protection Piping & Supports at petroleum storage and load racks during the design earthquake event. This assessment will be completed after further geotechnical investigations by Geo-Engineers Inc.-HOLD.

4.0 Assessment Findings-HOLD

We look forward to discussing this report with you in more detail and hopefully can answer any questions you may have. If you have any other questions or require additional information, please feel free to contact me at your convenience.

Sincerely,

John Deppa, P.E., S.E.
Principal Engineer
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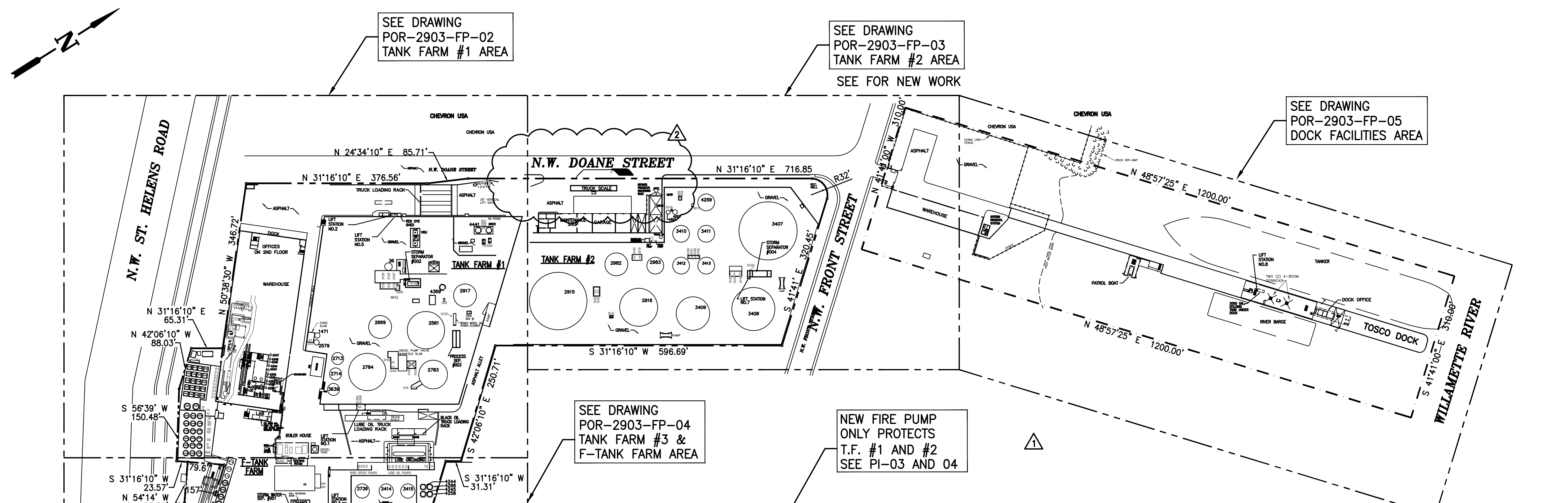
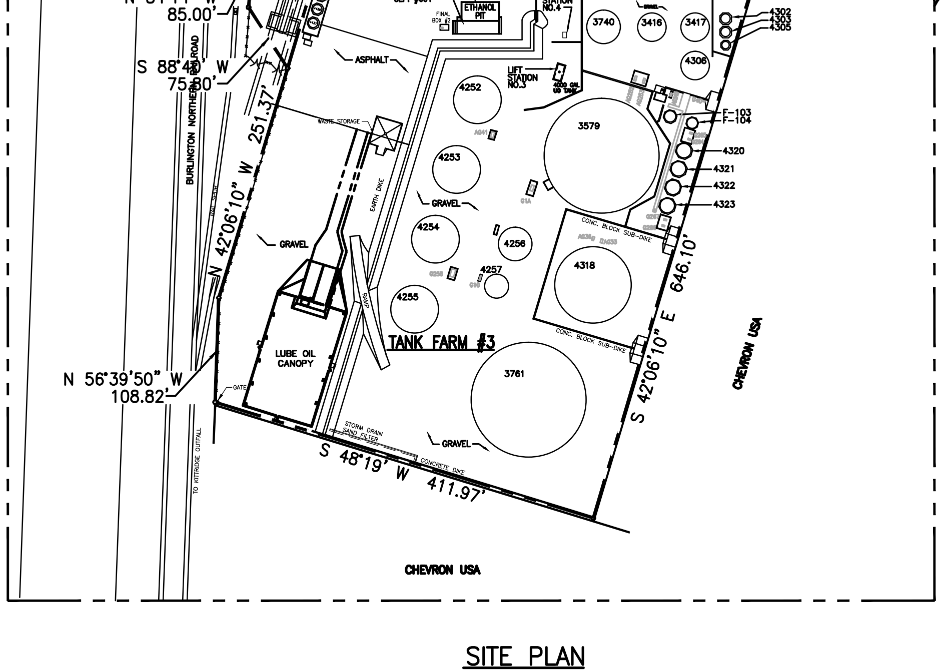
Appendix 1

- Fire Protection System Reference Drawings





SITE PLAN

Tank Number	Product Stored	Diameter	Height	Type of Construction	Safe Fill Capacity (gal)	Year of Construction
Tank Farm 1						
36	Slop Oil	14'-6"	16'	Riveted Steel	20,496	1907
1471	Hydraulic Tractor Fluid	10'-9"	28'-8½"	Riveted Steel	17,300	1921
2561	Marine Fuel Oil	78'-0"	46'-7"	Riveted Steel	1,569,582	1929
2579	Hydraulic Tractor Fluid	10'-9"	29'-8"	Welded Steel	18,000	1929
2669	Marine Diesel	48'-0"	35'-3"	Riveted Steel	449,694	1931
2713	Unax AW 46	24'-0"	35'-0"	Welded Steel	109,000	1937
2714	Guardol 15W/40	24'-0"	35'-0"	Welded Steel	109,000	1937
2783	Decant Oil	60'-0"	46'-7"	Riveted Steel	948,066	1937
2784	Diesel #2	78'-0"	40'-10"	Riveted Steel	1,439,130	1937
2917	RLOP 220 N	48'-0"	48'-0"	Welded Steel	612,000	1938
3623	HiTec 6576	10'-9"	30'-0"	Welded Steel	18,228	1950
3639	SUP SYN BL 5W/30	25'-0"	36'-0"	Welded Steel	120,000	1951
4369	RLOP 220N	10'-9"	30'-0"	Welded Steel	17,500	1979
4441	Octel 9056	10'-9"	30'-0"	Welded Steel	18,648	1983
Tank Farm 2						
2915*	Unleaded Gasoline	120'-0"	42'-0"	Welded Steel	3,262,056	1938
2916	Diesel #2	78'-0"	48'-0"	Welded Steel	1,652,196	1938
2982	Diesel #1	48'-0"	36'-0"	Welded Steel	416,262	1941
2983	RLOP 220 N	36'-0"	42'-0"	Welded Steel	304,000	1941
3407	Unleaded Gasoline	120'-0"	40'-0"	Welded Steel	2,955,540	1949
3408*	Unleaded Gasoline	90'-0"	40'-0"	Welded Steel	1,639,680	1949
3409	Unleaded Gasoline	70'-0"	40'-0"	Welded Steel	948,654	1949
3410	Ethanol	35'-0"	42'-0"	Welded Steel	278,964	1949
3411	Unleaded Gasoline	35'-0"	42'-0"	Welded Steel	259,350	1949
3412	Diesel #1	35'-0"	42'-0"	Welded Steel	279,426	1949
3413	Unleaded Gasoline	35'-0"	42'-0"	Welded Steel	259,560	1949
4223	Slop Oil	10'-9"	30'-0"	Welded Steel	18,690	1968
4259	Transmix	35'-0"	32'-0"	Welded Steel	205,506	1968
4327	Gasoline Slops	8'-6"	24'-0"	Welded Steel	10,080	1974

Tank Number	Product Stored	Diameter	Height	Type of Construction	Safe Fill Capacity (gal)	Year of Construction
Lube Blending Warehouse						
4338	Conoco AN 801	3'-6"	14'-0"	Welded Steel	1000	1974
4339	Ethyl HITEC 534	3'-6"	14'-0"	Welded Steel	1000	1974
4340	Ethyl HITEC 385	3'-6"	14'-0"	Welded Steel	1000	1974
4341	Utility	3'-6"	14'-0"	Welded Steel	1000	1974
4342	HITEC 008	3'-6"	14'-0"	Welded Steel	1000	1974
4343	Viscoplex 1-302	3'-6"	14'-0"	Welded Steel	1000	1974
4344	Oleic Acid/Emersol 213	3'-6"	14'-0"	Welded Steel	1000	1974
4345	Lubrizol 8790	3'-6"	14'-0"	Welded Steel	1000	1974
4346	Utility	3'-6"	14'-0"	Welded Steel	1000	1974
4347	Utility	3'-6"	14'-0"	Welded Steel	1000	1974
F-14	Flush - Under Lab	5'-3"	30'-0"	Welded Steel	5000	1981
F-8	Utility			Welded Steel	1000	1968
F-9	Line Clippings			Welded Steel	2200	1954
F-13	Slops - Under Lab	5'-3"	30'-0"	Welded Steel	5000	1981

Tank Number	Product Stored	Diameter	Height	Type of Construction	Safe Fill Capacity (gal)	Year of Construction
Tank Farm 3						
3414	RLOP 220 N	30'-0"	42'-0"	Welded Steel	200,000	1949
3415	SUN 525	30'-0"	42'-0"	Welded Steel	200,000	1949
3416	RLOP 100N	30'-0"	42'-0"	Welded Steel	200,000	1949
3417	ULTRA S-4	30'-0"	42'-0"	Welded Steel	200,000	1949
3579	Industrial Fuel Oil	120'-0"	40'-0"	Welded Steel	3,307,668	1950
3739	SUN 150 B/S	30'-0"	42'-0"	Welded Steel	200,000	1954
3740	RLOP 600 N	35'-0"	42'-0"	Welded Steel	277,000	1954
3761	Diesel #2	120'-0"	40'-0"	Welded Steel	3,240,342	1954
4244	Mohawk 450	10'-9"	30'-0"	Welded Steel	17,500	1968
4246	SUN 525	10'-9"	30'-0"	Welded Steel	17,500	1968
4252	Residual Fuel Oil	50'-0"	32'-0"	Welded Steel	458,640	1968
4253	Residual Fuel Oil	50'-0"	32'-0"	Welded Steel	451,290	1968
4254	PS 300	50'-0"	32'-0"	Welded Steel	459,312	1968
4255	Out of Service	50'-0"	32'-0"	Welded Steel	404,250	1968
4256	Out of Service	35'-0"	32'-0"	Welded Steel	195,408	1968
4257	Out of Service	25'-0"	16'-0"	Welded Steel	38,367	1968
4258	Line Clippings	10'-9"	30'-0"	Welded Steel	18,000	1968
4266	Flush	10'-9"	30'-0"	Welded Steel	17,500	1968
4302	RLOP 600N	10'-9"	30'-0"	Welded Steel	17,500	1971
4303	RLOP 100N	10'-9"	30'-0"	Welded Steel	17,500	1971
4305	Out of Service	8'-6"	24'-0"	Welded Steel	8,900	1971
4306	RLOP 100N	30'-0"	42'-2½"	Welded Steel	200,000	1971
4318	Diesel #2	80'-0"	43'-3"	Welded Steel	1,422,456	1973
4320	Sup Syn BL 10W/30	15'-0"	32'-2½"	Welded Steel	35,000	1973
4321	Uniguide II 100	15'-0"	32'-2½"	Welded Steel	35,000	1973
4322	T5X HD 15W/40	15'-0"	32'-2½"	Welded Steel	35,000	1973
4323	Super ATF	15'-0"	32'-2½"	Welded Steel	35,000	1973
F103	UTRA 58	12'-6"	32'-0"	Welded Steel	25,500	1973
F104	UTRA 58	10'-9"	32'-0"	Welded Steel	17,500	1973
F-Tank Farm						
4335	Utility	10'-9"	30'-0"	Welded Steel	17,500	1973
4336	Utility	10'-9"	30'-0"	Welded Steel	17,500	1973
4337	Utility	10'-9"	30'-0"	Welded Steel	17,500	1973
4436	Unax AW 32	10'-9"	30'-0"	Welded Steel	17,500	1990
4437	Unax AW WR 32	10'-9"	30'-0"	Welded Steel	17,500	1990
F10	Utility	10'-9"	10'-0"	Welded Steel	5,200	1954
F11	Utility	10'-9"	10'-0"	Welded Steel	5,200	1954
F12	Utility	10'-9"	10'-0"	Welded Steel	5,200	1954

Tank Number	Product Stored	Diameter	Height	Type of Construction	Safe Fill Capacity (gal)	Year of Construction
Upper Lube Cell						
3741	Ramax CLF 17E	10'-9"	30'-0"	Welded Steel	17,500	1954
3742	MP Gear Lube 80/90	10'-9"	30'-0"	Welded Steel	17,500	1954
3743	Utility	10'-9"	30'-0"	Welded Steel	16,800	1954
3744	HYNAP N100	10'-9"	30'-0"	Welded Steel	17,500	1954
3745	HITEC 6751	10'-9"	30'-0"	Welded Steel	17,500	1954
3746	Lubrizol 4998C	10'-9"	30'-0"	Welded Steel	17,500	1954
3747	Lubrizol 4990CH	10'-9"	30'-0"	Welded Steel	17,500	1954
3757	HITEC 1123	10'-9"	30'-0"	Welded Steel	17,500	1954
3760	Raffene 750L	10'-9"	30'-0"	Welded Steel	17,500	1954
4191	Lubrizol 8820A	10'-9"	30'-0"	Welded Steel	17,500	1954
4192	Lubrizol 7075F	10'-9"	30'-0"	Welded Steel	17,500	1954
4241	UNAX AW 68	10'-9"	30'-0"	Welded Steel	17,500	1968
4242	UNAX AW 68	10'-9"	30'-0"	Welded Steel	17,500	1968
4243	HT4P10W	10'-9"	30'-0"	Welded Steel	17,500	1968
4281	Versa Tran ATF	10'-9"	30'-0"	Welded Steel	17,500	1969
4332	Super ATF	10'-9"	30'-0"	Welded Steel	17,500	1973
4333	Point Premier 10W/30	10'-9"	30'-0"	Welded Steel	17,500	1973
4334	Super 5W/20	10'-9"	30'-0"	Welded Steel	17,500	1973
Lower Lube Cell						
4300	Ramax CLF 17E	12'-6"	32'-0"	Welded Steel	25,000	1968
4331	Ethyl HITEC 8888	12'-6"	32'-0"	Welded Steel	25,000	1973
4388	Utility	9'-4"	23'-0"	Welded Steel	13,500	1984
4389	Utility	9'-4"	23'-0"	Welded Steel	13,500	1984
4390	Bar & Chain 150	9'-4"	23'-0"	Welded Steel	13,500	1986
4391	Utility	9'-4"	23'-0"	Welded Steel	13,500	1986
4392	Utility	9'-4"	23'-0"	Welded Steel	13,600	1986
4393	Utility	9'-4"	23'-0"	Welded Steel	13,600	1986
4394	Utility	9'-4"	23'-0"	Welded Steel	13,600	1986
4395	Utility	9'-4"	23'-0"	Welded Steel	13,600	1986
4397	Lubrizol 9602A	9'-4"	23'-0"	Welded Steel	13,600	1986
4398	HITEC 1129A	9'-4"	23'-0"	Welded Steel	13,600	1986
4399	Fleetol 15W40	9'-4"	23'-0"	Welded Steel	13,600	1986
4400	Guardol 30	9'-4"	23'-0"	Welded Steel	13,600	1986
4401	Mohawk 150	9'-4"	23'-0"	Welded Steel	13,600	1986
4402	T5X 10/10	9'-4"	23'-0"	Welded Steel	13,500	1986
4403	HT4P30W	9'-4"	23'-0"	Welded Steel	13,600	1986
4404	Fleet Sup EC 15W40	9'-4"	23'-0"	Welded Steel	13,600	1986
4405	HITEC 3472	9'-4"	23'-0"	Welded Steel	13,600	1987
4406	Lubrizol 9990A	9'-4"	23'-0"	Welded Steel	13,600	1987
4407	Ethyl HITEC 388	9'-4"	23'-0"	Welded Steel	13,600	1987
4408	Ethyl HITEC 5750	9'-4"	23'-0"	Welded Steel	13,600	1987

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SCALE: 1"=100' (22x34)
SCALE: 1"=200' (11x17)

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POR-2903-FP-03	TANK FARM #2 AREA
POR-2903-FP-04	F-TANK FARM AREA
POR-2903-FP-05	DOCK FACILITIES AREA

NO.	REVISION DESCRIPTION	REV BY	CHK BY	APPR. BY	DATE
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1	FIRE PUMP REPLACEMENT PROJECT	CWP	JPD	JPD	09/16/08
2	NEW BACK FLOW PREVENTOR IN CITY VAULT	MJB	JPD		

NO.	REVISION DESCRIPTION	REV BY	CHK BY	APPR. BY	DATE

ConocoPhillips Pipe Line Company
5528 N.W. DOANE ST.
PORTLAND, OREGON 97210

DR. DAB		CONTRACTOR No.	
DSGN. DAB		PROJECT NO. 25-97	
CK. WHB	AP.	DWG. SCALE NONE	
DATE 4/01/03		PLOT SCALE	

ALPHA ENGINEERS and CONSTRUCTORS, INC.
2929 NW 29th AVE, PORTLAND, OR. 97210 Tel:(503)227-3317/Fax:(503)227-3244

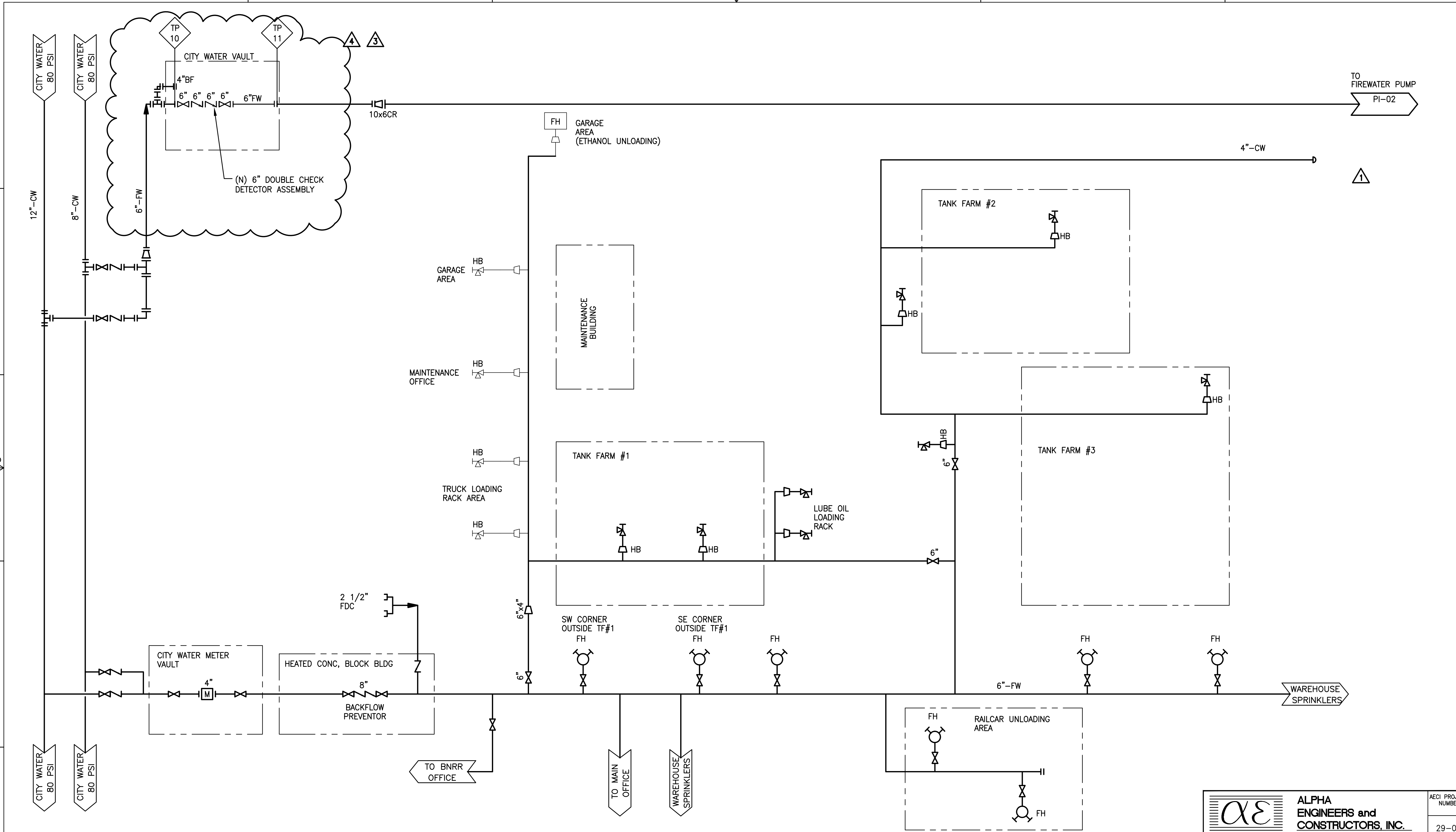
AECI PROJECT NUMBER 25-97

PORTLAND TERMINAL

TITLE:

FIRE PROTECTION SYSTEM
TANK FARM & DOCK AREA
OVERALL SITE PLAN

DWG. No. POR-2903-FP-01 SHEET 1 OF 1 REV. No. 2



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		1	GENERAL REVISION	CWP	JPD	JPD	12/04/08
		2	ADDED BACK FLOW PREVENTER	CWP	JPD	JPD	01/26/09
		3	ADD BACKFLOW PREVENTER IN CITY VAULT	MJB	JPD		06/18/09
		4	REVISED DCDA TO 6"	MJB	JPD		11/18/09

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ConocoPhillips Pipe Line Company
5528 N.W. DOANE ST.
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ALPHA ENGINEERS and CONSTRUCTORS, INC.
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AECI PROJECT NUMBER 29-03

DR. DAB	CONTRACTOR No.
DSGN. DAB	PROJECT NO. 2903-06
CK. WHB AP.	DWG. SCALE NONE
DATE 4/01/03	PLOT SCALE 1=1

PORTLAND TERMINAL

TITLE: PIPING & INSTRUMENTATION DIAGRAM
FIRE PROTECTION SYSTEM
MAIN TERMINAL WATER DISTRIBUTION SYSTEM

DWG. No. POR-2903-PI-01 SHEET 1 OF 1 REV. No. 4

P-1 FIRE WATER PUMP

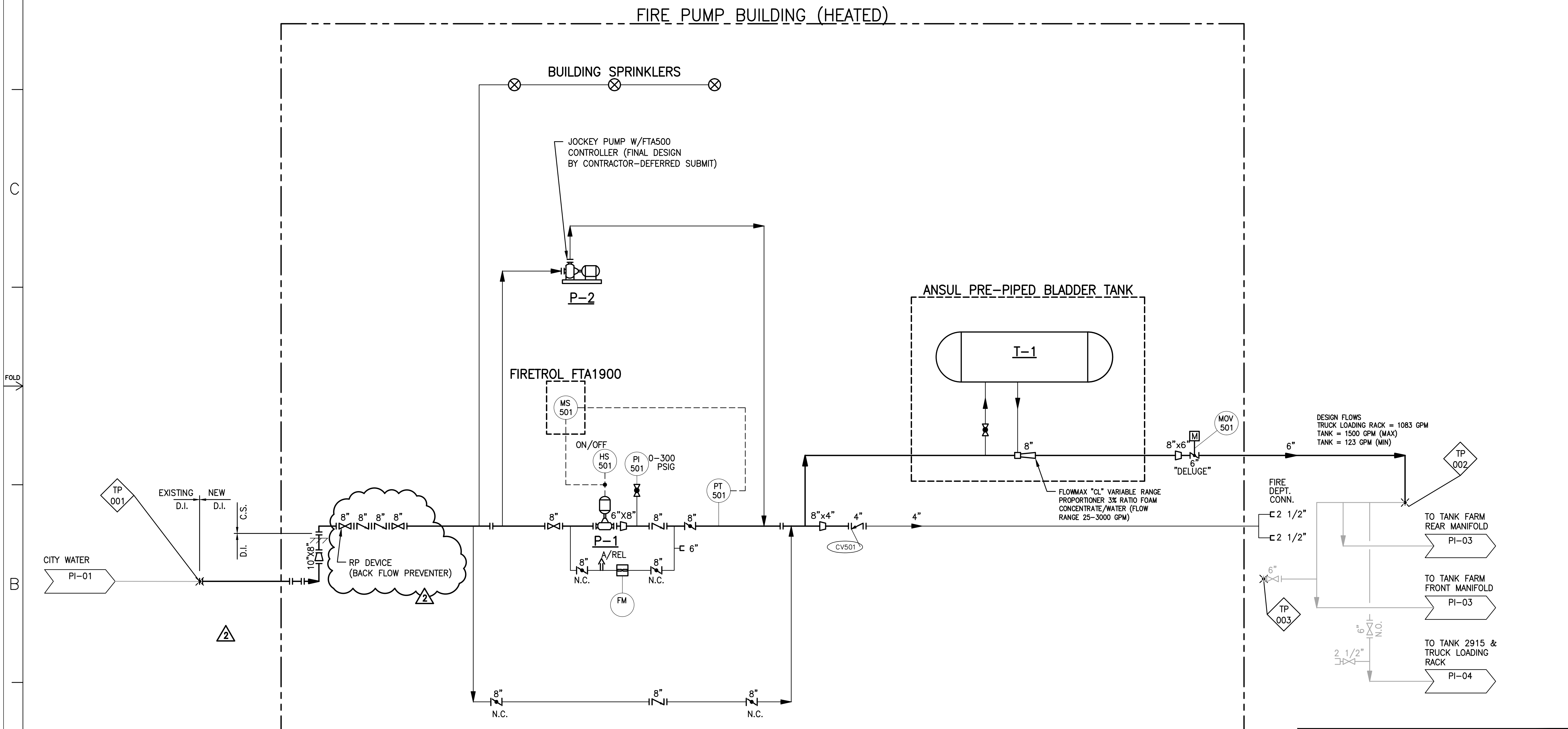
TYPE: CENTRIFUGAL
FLOW RATE: 1500 GPM @ 75 PSI
HP 100 @ 1770 RPM
460/3PH/60HZ
MFG.: AURORA TYPE 5-481-15
(8x6x15)

P-2 FIRE WATER JOCKEY PUMP

TYPE: CENTRIFUGAL
FLOW RATE: 20 GPM @ 75 PSI
HP 2 @ 3500 RPM
230/460/3PH/60HZ
MFG.: AURORA TYPE PVM4-50

T-1 FOAM CONCENTRATE TANK

TYPE: HORIZONTAL BLADDER
MFG: ANSUL
SIZE: 6'-0" DIA. X 13'-11" LONG
CAPACITY: 2200 GALLON



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		1	NEW FIRE PUMP AND FOAM TANK	CWP	JPD	JPD	09/16/08						
		2	REMOVED BOILER FEED/BACKFLOW PREVENTER	CWP	JPD	JPD	12/01/08						
		3	ADDED R.P. DEVICE	CWP	JPD	JPD	01/26/09						

ConocoPhillips Pipe Line Company
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AECI PROJECT NUMBER: 25-97

DR. DAB	CONTRACTOR No.
DSGN. DAB	PROJECT NO. 25-97
CK. WHB / AP.	DWG. SCALE NONE
DATE 4/01/03	PLOT SCALE 1=1

PORTLAND TERMINAL

TITLE: PIPING & INSTRUMENTATION DIAGRAM
FIRE PROTECTION SYSTEM
FIRE WATER BOOSTER PUMP & FOAM SYSTEM (NEW)

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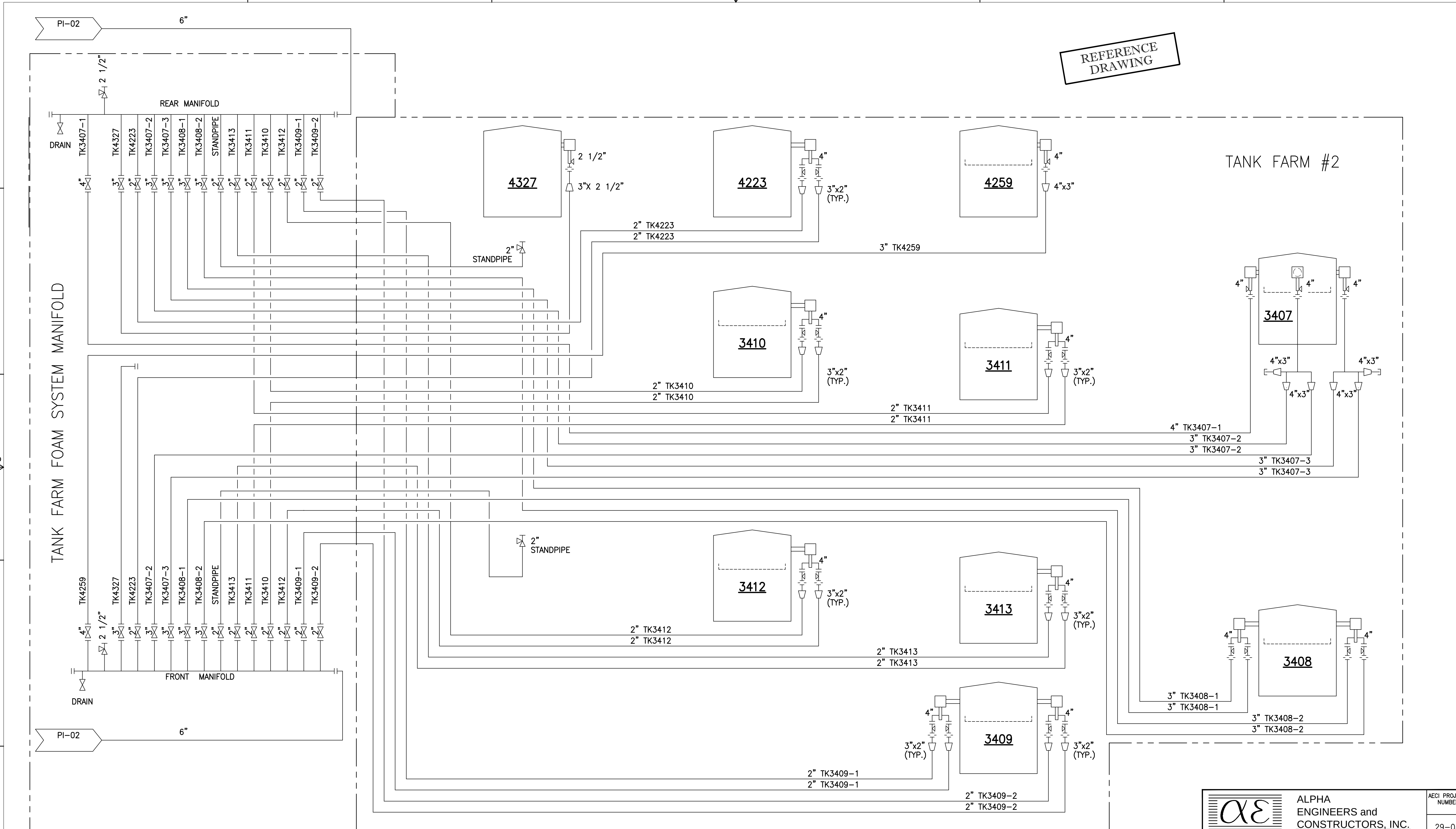

ALPHA ENGINEERS and CONSTRUCTORS, INC.
2929 NW 29th AVE, PORTLAND, OR. 97210 Tel:(503)227-3317/Fax:(503)227-3244

AECI PROJECT NUMBER 29-03

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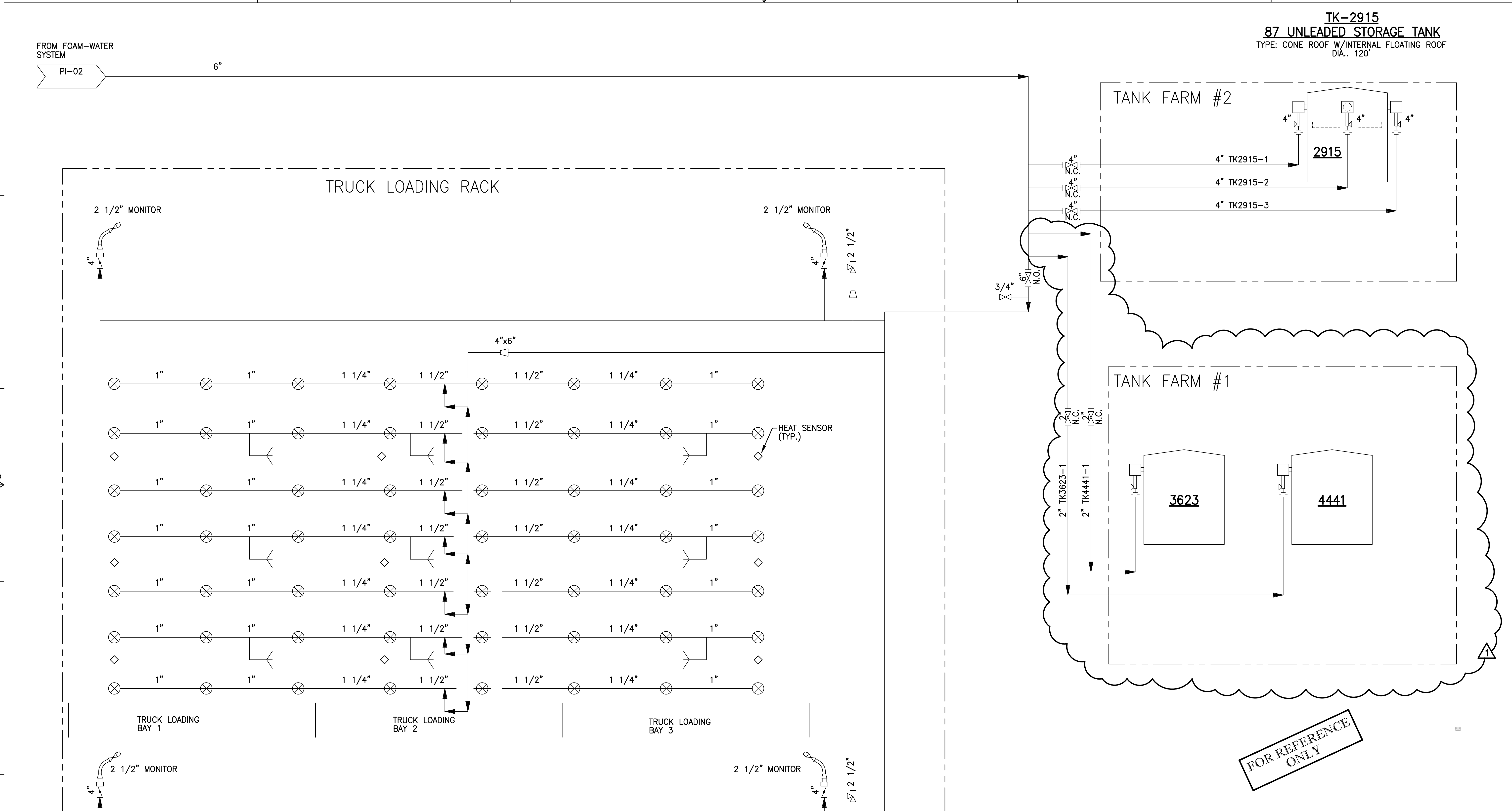
ConocoPhillips Pipe Line Company
5528 N.W. DOANE ST.
PORTLAND, OREGON 97210

DR. DAB	CONTRACTOR No.
DSGN. DAB	PROJECT NO. 2903-06
CK. WHB AP.	DWG. SCALE NONE
DATE 4/01/03	PLOT SCALE 1=1

PORTLAND TERMINAL

TITLE:
PIPING & INSTRUMENTATION DIAGRAM
FIRE PROTECTION SYSTEM
TANK FARM NO.2 FOAM WATER SYSTEM

DWG. No. POR-2903-PI-03 SHEET 1 OF 1 REV. No. 0



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AECI PROJECT NUMBER 29-03

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		1	ADDED TANKS	JDS	JPD	DAB	09/16/08						

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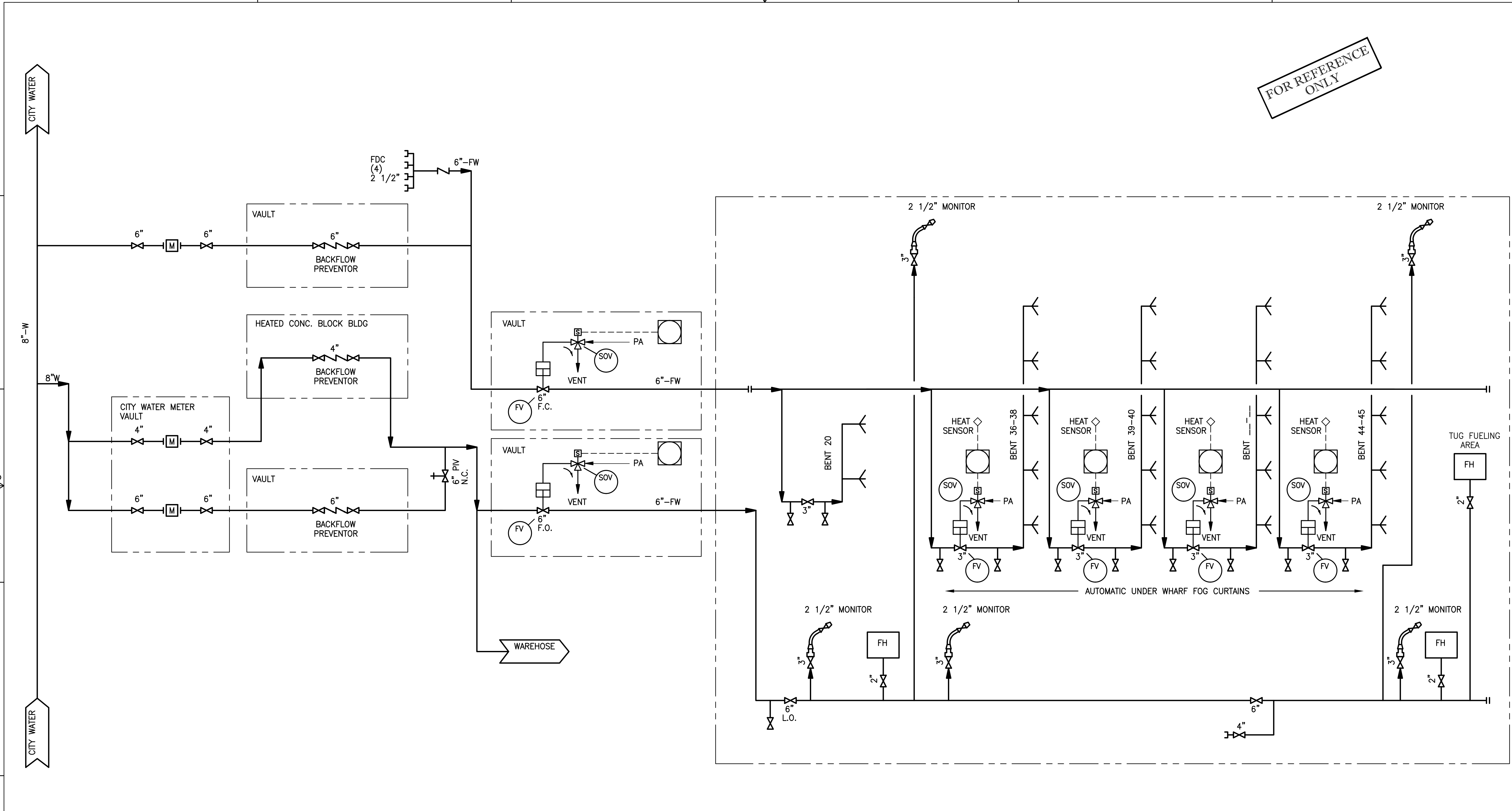
ConocoPhillips Pipe Line Company
5528 N.W. DOANE ST.
PORTLAND, OREGON 97210

DR. DAB	CONTRACTOR No.
DSGN. DAB	PROJECT NO. 2903-06
CK. WHB AP.	DWG. SCALE NONE
DATE 4/01/03	PLOT SCALE 1=1

PORTLAND TERMINAL

TITLE:
PIPING & INSTRUMENTATION DIAGRAM
FIRE PROTECTION SYSTEM
TRUCK LOADING RACK T.F. #1 & 2 FOAM SYSTEM

DWG. No. POR-2903-PI-04
SHEET 1 OF 1
REV. No. 1



ALPHA ENGINEERS and CONSTRUCTORS, INC.

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AECI PROJECT NUMBER: 29-03

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ConocoPhillips Pipe Line Company
5528 N.W. DOANE ST.
PORTLAND, OREGON 97210

DR. DAB	CONTRACTOR No.
DSGN. DAB	PROJECT NO. 2903-06
CK. WHB AP.	DWG. SCALE NONE
DATE 4/01/03	PLOT SCALE 1=1

PORTLAND TERMINAL

TITLE: PIPING & INSTRUMENTATION DIAGRAM
FIRE PROTECTION SYSTEM
WHARF FIRE WATER SYSTEM

DWG. No. POR-2903-PI-05 SHEET 1 OF 1 REV. No. 0

Appendix H
Control Systems

ALPHA TECHNICAL GROUP INC.
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P66 Pipeline LLC- Portland Terminal
Seismic Vulnerability Assessment (SVA)
Control Systems

Rev Date: May 10, 2024

REGISTERED PROFESSIONAL
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15471
OREGON
JULY 25, 1991
JOHN DEPPA
EXPIRES 12/31/25

PRELIMINARY

Table of Contents

1.0 Introduction & Scope

This report summarizes the results of a Seismic Vulnerability Assessment (SVA) of the P66 Pipeline's Portland Terminal's Fire Protection Systems. This assessment is intended to directly address the requirements outlined in the recently adopted Oregon DEQ "Fuel Tank Seismic Stability Rules" OAR 300. Alpha Technical Group Inc (ATGI) has designed various Control Systems Improvements and updates to the Facility's Control System Drawings over many years.

2.0 Control System(s) & Operations Overview

The Terminal has many Control Systems which allow for automated control of various pumps, valves, and related equipment. Programmable Logic Controllers (PLCs) are connected to various instrumentation devices throughout the Terminal. These devices include level transmitters on tanks and pressure/ temperature transmitters on product piping systems. The PLCs are programmed to shut off pumps and related safety equipment in the event of an upset condition and provide an extra measure of safety during operations. For example, high level gauging instruments on tanks are connected to the control system and will send signals to shut off pumps and prevent overfilling of tanks. There are many similar automated processes during operations along with manual operations.

3.0 Seismic Risk Assessment & Methodology

The control systems that have capability of limiting spills and protecting life safety will be the focus of the assessment.

<u>3.1 Assessment of Control System(s) functionality during Design Earthquake Event:</u>

This assessment will assess the ability of the Fire Protection System(s) to function during the design earthquake event and limit any damage to the Terminal. This is important to protect life safety as well as ensure that the Terminal can get back online as soon as possible in order to provide fuel for emergency services. This assessment will be conducted by John Deppa PE and P66 Operations Staff using Process Hazard Analysis (PHA) procedures. This assessment will be completed after further geotechnical investigations are completed by Geo-Engineers Inc.-HOLD.

<u>3.2 Assessment of Control System Equipment & Supports during Design Earthquake Event:</u>

This assessment will evaluate the main terminal Control System Equipment & Supports during the design earthquake event. This assessment will be completed after further geotechnical investigations by Geo-Engineers Inc.-HOLD.

4.0 Assessment Findings-HOLD

We look forward to discussing this report with you in more detail and hopefully can answer any questions you may have. If you have any other questions or require additional information, please feel free to contact me at your convenience.

Sincerely,

John Deppa, P.E., S.E.
Principal Engineer
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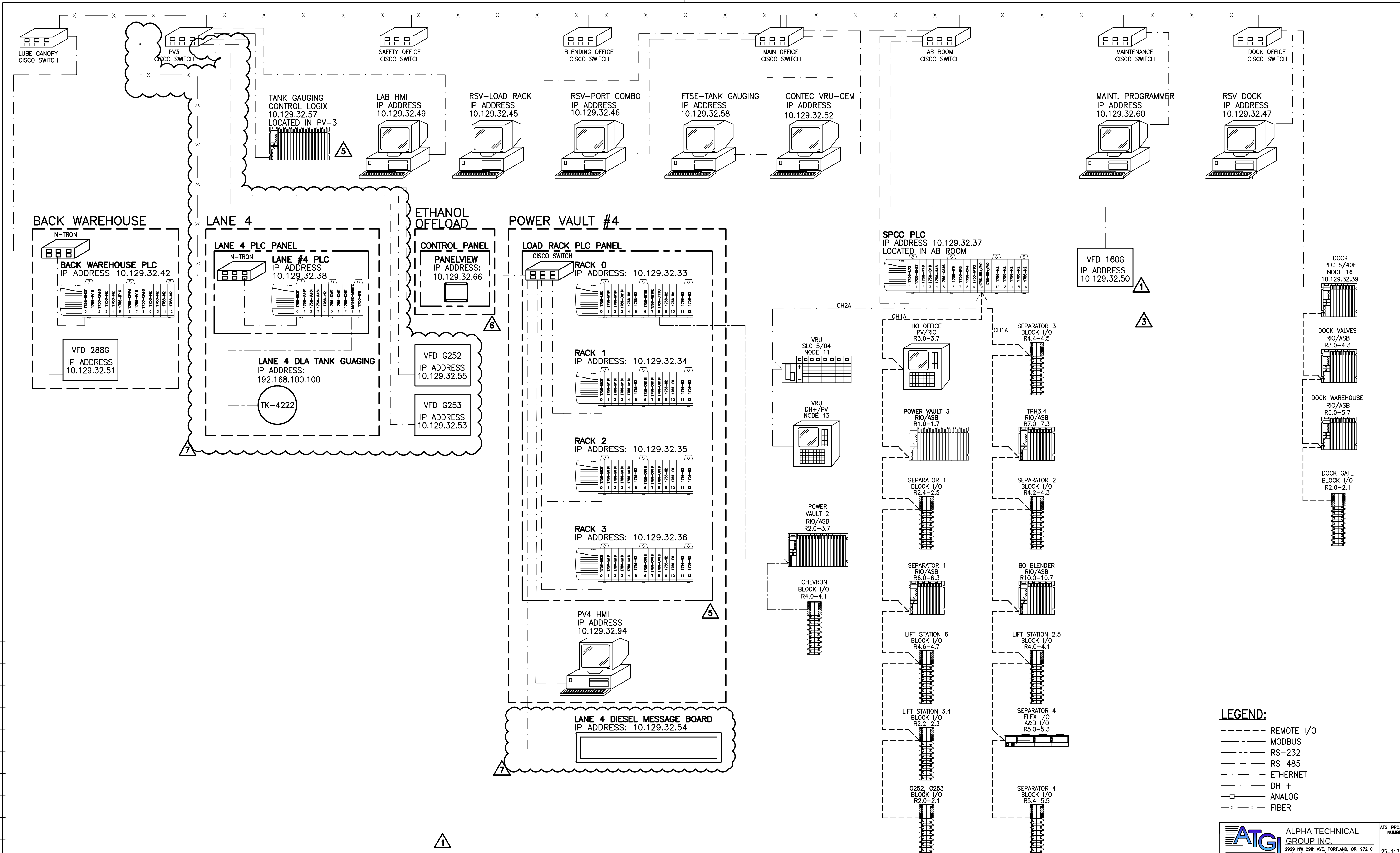
Appendix 1

- Control System(s) Reference Drawings



ATGI ALPHA TECHNICAL GROUP INC.
2929 NW 29th AVE, PORTLAND, OR. 97210 Tel:(503)227-3317/Fax:(503)227-3244

ATGI PROJECT NUMBER 25-113-03

NO.	REVISION	BY / CHKD	DATE / APP'D	NO.	REVISION	BY / CHKD	DATE / APP'D	NO.	REVISION	BY / CHKD	DATE / APP'D
0				3	REVISED PER PORTLAND OPTIMIZATION IMPRVMTS	JGS / AMW	06/23/17 / BDC	7	REVISED PER DIESEL LOADING – LANE 4	PW / AMW	12/27/19 / BDC
1	REVISED PER PORTLAND OPTIMIZATION IMPRVMTS	JGS / JPD	10/14/16 / JPD	4	RECORD DRAWING	JGS / AMW	02/16/18 / BDC				
2	REVISED PER BIO-DIESEL IMPROVEMENTS	JGS / AMW	04/07/17 / BDC	5	REVISED PER LOAD RACK PLC CONVERSION	JGS / AMW	12/15/17 / BDC				
				6	REVISED PER ETHANOL OFFLOAD PROJECT	JGS / AMW	09/21/18 / BDC				

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APP'D JEF

PORTLAND TERMINAL

PORTLAND LOAD RACK PLC CONVERSION
INSTRUMENTATION – CONTROL SYSTEM
ARCHITECTURE SHEET 1 OF 3

MULTNOMAH COUNTY, OREGON

SCALE NONE
PROJECT NO 25-113.1
FILE NAME (aka) DOCUMENT NUMBER PORT-IN-0043

FOR REFERENCE ONLY – OFFICIAL DOCUMENT STORED ELECTRONICALLY PLOT DATE = cpd PLOTTED BY : $PLOTUSER$ FILE NAME : file

LANE MASTER	PV-2-RR-2
PLC 5/40E	1771-P2
(0) 1771-IAD	1771-ASB
(1) 1771-IAD	(0) 1771-IAD
(2) 1771-OBD-OOS	(1) 1771-IAD
(3) 1771-OBD-OOS	(2) 1771-IAD
(4) 1771-OA	(3) 1771-OAD
(5) 1771-OB	(4) 1771-IAD
(6) 1771-IAD	(5) 1771-IAD
(7) 1771-IAD	(6) 1771-IAD
(8) 1771-DB/B-OOS	(7) 1771-IAD
(9) 1771-DB/B-OOS	(8) 1771-IAD
(10) 1771-DB/B	(9) 1771-OAD
(11) 1771-OW16	(10) 1771-OAD NON ESD
(12) 1771-IBD	(11) 1771-IAD NON ESD
(13) 1771-IBD	(12) SPACE
(14) 1771-IAD	(13) 1771-ID16
(15) 1771-P4S	(14) 1771-IAD
	(15) 1771-OAN

SPCC	PV-3-RR-1	TPH3.4-RR7
PLC 5/40E	1771-ASB	THP-UNIT 1
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(15) 1771-OAN	(15) SPACE	

DOCK	DOCK VALVES-RR-3	DOCK WAREHOUSE-RR-5
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(1) 1771-IAD
(2) 1771-OAD/B
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(4) 1771-OAD/B
(5) SPACE
(6) 1771-IFE/C
(7) SPACE
(8) 1771-OW
(9) 1771-IAD
(10) 1771-P4S
(11) 1771-P4S

LANE 3
PLC-5/20E
(0) 1771-OAD/B
(1) 1771-IAD
(2) 1771-OAD/B
(3) 1771-IAD
(4) 1771-OAD/B
(5) SPACE
(6) 1771-IFE/C
(7) SPACE
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(9) 1771-IAD
(10) 1771-P4S
(11) 1771-P4S

LUBE CANOPY
PLC-5/20E
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(5) 1771-IFE
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(10) SPACE
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(12) SPACE
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(14) SPACE
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(11) 1771-P4S

NOTE:

1. SEE PORT-IN-0043 FOR SHEET 1

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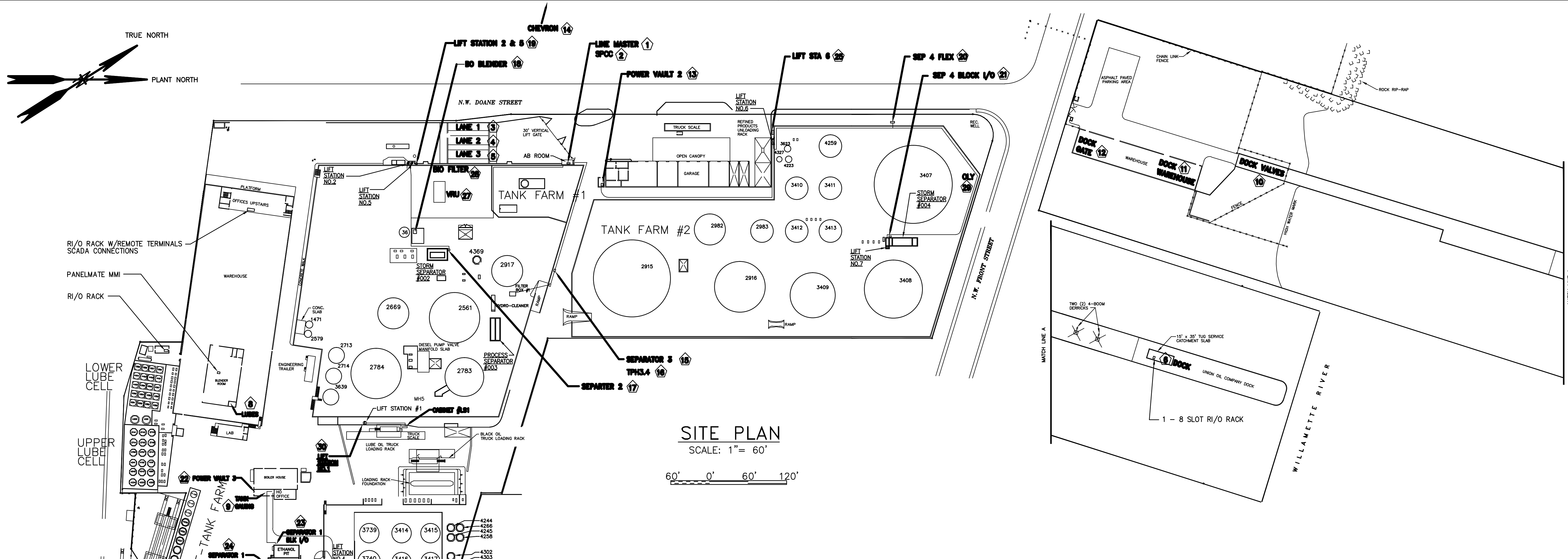
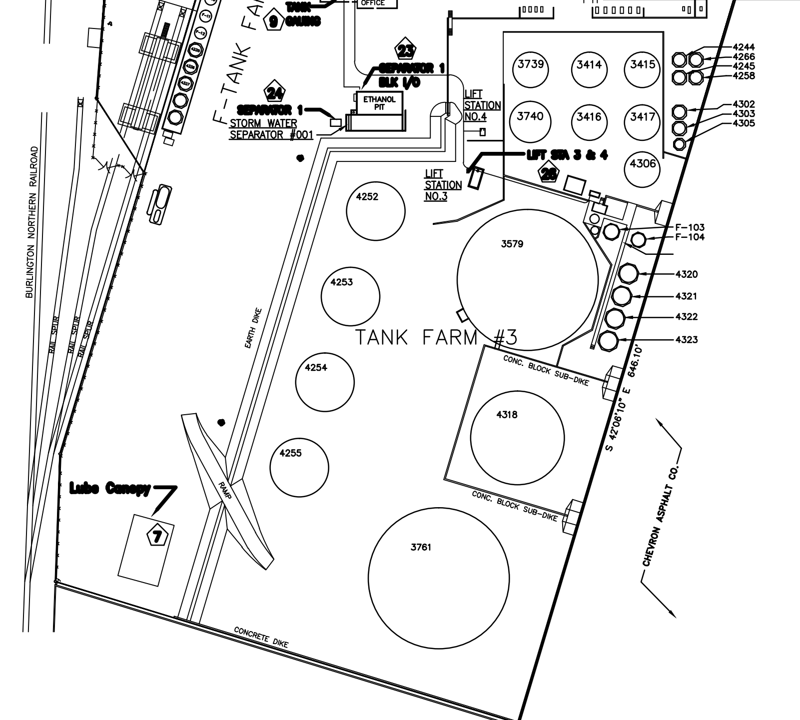
PORTLAND
CONTROL SYSTEM ARCHITECTURE
SHEET 2 OF 3
PORTLAND COUNTY, OREGON

SCALE
PROJECT NO
FILE NAME (aka) DOCUMENT NUMBER
PORT-IN-0044





PLC'S

Number	Name	Location	Parent	Comm.
1	Lane Master	AB Room	----	Ethernet
2	SPCC	AB Room	----	Ethernet
3	Lane 1	Lane 1	----	Ethernet
4	Lane 2	Lane 2	----	Ethernet
5	Lane 3	Lane 3	----	Ethernet
6	Dock	Dock House	----	Ethernet
7	Lube Canopy	Lube Canopy	----	Ethernet
8	Lubes	Lubes Blending Ctrl	----	Ethernet
9	Tank Gauging	PV-3	----	Ethernet
10	Dock Valves	Dock Entry	Dock	RIO
11	Dock Whs	E end of Whs	Dock	RIO
12	Dock Gate	W end of Whs	Dock	RIO
13	Power Vault 2	PV-2	Lane Master	RIO
14	Chevron	Chev Tower 1st Fl	Lane Master	RIO
15	Separator 3	Separator 3	SPCC	RIO
16	TPH3.4	Term Exit Alley	SPCC	RIO
17	Separator 2	Separator 2	SPCC	RIO

PLC'S

Number	Name	Location	Parent	Comm.
18	BO Blender	TK 36	SPCC	RIO
19	Lift Station 2 & 5	W end of Load Rk	SPCC	RIO
20	Sep 4 Flex	TF2 Doane St	SPCC	RIO
21	Sep 4 Block I/O	Sep arator 4	SPCC	RIO
22	Power Vault 3	PV-3	SPCC	RIO
23	Separtor 1 Blk I/O	Separtor 1	SPCC	RIO
24	Separtor 1	Separtor 1	SPCC	RIO
25	Lift Sta 6	Lift Station 6	SPCC	RIO
26	Lift Sta 3 & 4	Tween 3 & 4	SPCC	RIO
27	VRU	VRU	----	DH+
28	Bio Filters	Bio Filters	SPCC	RIO
29	OLY	OLY Manifold	SPCC	RIO
30	Lift Sta 1 BLK I/O	Lift Station #1	SPCC	RIO
31				
32				
33				
34				

NO.	REVISION	BY / CHKD	DATE / APP'D

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PORTLAND

PLC PLOT PLAN
SHEET 3 OF 3

PORTLAND COUNTY, OREGON

SCALE
PROJECT NO
FILE NAME (aka) DOCUMENT NUMBER
PORT-IN-0045

Appendix I
Report Limitations and Guidelines for Use

Appendix I
Report Limitations and Guidelines for Use[1]

This appendix provides information to help you manage your risks with respect to the use of this report.

Geotechnical Services Are Performed for Specific Purposes, Persons and Projects

This report has been prepared for the exclusive use of Phillips 66 Pipeline LLC. This report may be made available to the Oregon DEQ's Fuel Tank Seismic Stability (FTSS) Program in order to perform their duties in accordance with Oregon Administrative Rules Chapter 340 Division 300 (OAR 340-300). This report should not be shared with others nor is it intended for use by others, and the information regarding the terminal contained herein is considered confidential and proprietary.

GeoEngineers structures our services to meet the specific needs of our clients. For example, a geotechnical or geologic study conducted for a civil engineer or architect may not fulfill the needs of a construction contractor or even another civil engineer or architect that are involved in the same project. Because each geotechnical or geologic study is unique, each geotechnical engineering or geologic report is unique, prepared solely for the specific client and project site. Our report is prepared for the exclusive use of our client. No other party may rely on the product of our services unless we agree in advance to such reliance in writing. This is to provide our firm with reasonable protection against open-ended liability claims by third parties with which there would otherwise be no contractual limits to their actions. Within the limitations of scope, schedule and budget, our services have been executed in accordance with our Agreement with the Client and generally accepted geotechnical practices in this area at the time this report was prepared. This report should not be applied for any purpose or project except the one originally contemplated.

A Geotechnical Engineering or Geologic Report is based on a Unique Set of Project-Specific Factors

This report has been prepared for the seismic vulnerability assessments (Phase 1) performed to date for the Phillips 66 Terminal and Lubricants facility in Portland, Oregon. GeoEngineers considered a number of unique, project-specific factors when establishing the scope of services for this project and report. Unless GeoEngineers specifically indicates otherwise, do not rely on this report if it was:

- Not prepared for you,
- Not prepared for your project,
- Not prepared for the specific site explored, or
- Completed before important project changes were made.

For example, changes that can affect the applicability of this report include those that affect:

[1] Developed based on material provided by GBA, Geoprofessional Business Association; www.geoprofessional.org.

- The function of the proposed structure;
- Elevation, configuration, location, orientation or weight of the proposed structure;
- Composition of the design team; or
- Project ownership.

If changes occur after the date of this report, GeoEngineers cannot be responsible for any consequences of such changes in relation to this report unless we have been given the opportunity to review our interpretations and recommendations. Based on that review, we can provide written modifications or confirmation, as appropriate.

Subsurface Conditions Can Change

This geotechnical or geologic report is based on conditions that existed at the time the study was performed. The findings and conclusions of this report may be affected by the passage of time, by manmade events such as construction on or adjacent to the site, or by natural events such as floods, earthquakes, slope instability or groundwater fluctuations. Always contact GeoEngineers before applying a report to determine if it remains applicable.

Most Geotechnical and Geologic Findings Are Professional Opinions

Our interpretations of subsurface conditions are based on field observations from widely spaced sampling locations at the site. Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. GeoEngineers reviewed field and laboratory data and then applied our professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ, sometimes significantly, from those indicated in this report. Our report, conclusions and interpretations should not be construed as a warranty of the subsurface conditions.

Geotechnical Engineering Report Recommendations Are Not Final

Do not over-rely on the preliminary construction recommendations included in this report. These recommendations are not final, because they were developed principally from GeoEngineers’ professional judgment and opinion. GeoEngineers’ recommendations can be finalized only by observing actual subsurface conditions revealed during construction. GeoEngineers cannot assume responsibility or liability for this report's recommendations if we do not perform construction observation.

Sufficient monitoring, testing and consultation by GeoEngineers should be provided during construction to confirm that the conditions encountered are consistent with those indicated by the explorations, to provide recommendations for design changes should the conditions revealed during the work differ from those anticipated, and to evaluate whether or not earthwork activities are completed in accordance with our recommendations. Retaining GeoEngineers for construction observation for this project is the most effective method of managing the risks associated with unanticipated conditions.

A Geotechnical Engineering or Geologic Report Could Be Subject to Misinterpretation

Misinterpretation of this report by other design team members can result in costly problems. You could lower that risk by having GeoEngineers confer with appropriate members of the design team after submitting the report. Also retain GeoEngineers to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering or geologic report. Reduce that risk by having GeoEngineers participate in pre-bid and preconstruction conferences, and by providing construction observation.

Do Not Redraw the Exploration Logs

Geotechnical engineers and geologists prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering or geologic report should never be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable but recognize that separating logs from the report can elevate risk.

Give Contractors a Complete Report and Guidance

Some owners and design professionals believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering or geologic report, but preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with GeoEngineers and/or to conduct additional study to obtain the specific types of information they need or prefer. A pre-bid conference can also be valuable. Be sure contractors have sufficient time to perform additional study. Only then might an owner be in a position to give contractors the best information available, while requiring them to at least share the financial responsibilities stemming from unanticipated conditions. Further, a contingency for unanticipated conditions should be included in your project budget and schedule.

Contractors Are Responsible for Site Safety on Their Own Construction Projects

Our geotechnical recommendations are not intended to direct the contractor’s procedures, methods, schedule or management of the work site. The contractor is solely responsible for job site safety and for managing construction operations to minimize risks to on-site personnel and to adjacent properties.

Geotechnical, Geologic and Environmental Reports Should Not Be Interchanged

The equipment, techniques and personnel used to perform an environmental study differ significantly from those used to perform a geotechnical or geologic study and vice versa. For that reason, a geotechnical engineering or geologic report does not usually relate to any environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. Similarly, environmental reports are not used to address geotechnical or geologic concerns regarding a specific project.