Seismic Vulnerability Assessment

Phase 1

McCall Terminal
Portland, Oregon

for
McCall Oil & Chemical Corporation

May 31, 2024

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McCall Terminal
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1.0 Introduction

This report summarizes the preliminary results of seismic vulnerability assessments (Phase 1) performed to date for the McCall Terminal Facility, in Portland, Oregon. The seismic vulnerability assessments were requested by McCall Oil & Chemical Corporation (McCall) in response to the 81st Oregon Legislative Assembly passed Senate Bill 1567 that requires owners or operators of bulk oils and liquid 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) and Reid Middleton, Inc. (RM). 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 tanks, pipelines, piers and wharves (docks), buildings, and concrete containment walls.

McCall Terminal is located in the northwest portion of the City of Portland on the west bank of the Willamette River and southeast of the St. John's Bridge. The terminal site occupies approximately 19.4 acres and is bounded by Brenntag Pacific Inc., to the southwest, High Purity Products to the northwest, the McCall access road to the southeast, and the shoreline of the Willamette River to the north and east. The terminal is a petroleum bulk station that stores and distributes various petroleum products for industrial and commercial uses. The project 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 extensive and rendered the structural evaluation of individual infrastructure components moot. 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 McCall Terminal 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 objectives 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 American Society of Civil Engineers (ASCE) 7-16 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 terminal facility generally consists of a dock on the Willamette River (at river mile 7.8) for tankers and barges, a Tank Farm with aboveground storage tanks (ASTs), an Asphalt Plant with ASTs and a truck loading rack, one four-bay truck loading rack with vapor oxidation system, a railcar unloading rack, an oil/water separator, and complete office facilities. The Tank Farm ASTs are contained within an approximate 8 to 12-foot-high earthen berm capped in asphalt. The Asphalt Plant ASTs are contained within an approximate 3.2-foot-high concrete wall. The total tank capacity in the Tank Farm is 862,942 bbls with the capacity of the largest tank being 280,000 bbls approximately. The total tank capacity in the Asphalt Plant is 43,445 bbls with the capacity of the largest tank being 9,700 bbls approximately.

The facility also has a pier/dock on the Willamette River. The dock is used for loading and offloading petroleum products from tankers and barges. The dock was designed and built in 1974-1975. The original supports for the dock approach, pier head, and personnel walkways are creosote-treated timber framing and timber piles; however, several of those members have been replaced with steel elements during several repairs completed over the years. The dock approach surface, pier head surface, and personnel walkways themselves are constructed of reinforced concrete. The facility layout is depicted in Figure 2, Site Plan.

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 +39 feet in the North American Vertical Datum of 1988 (NAVD88). The earthen berm surrounding the Tank Farm is approximately 8 to 12 feet higher than the surrounding site grades.

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. 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 to 15 feet below ground surface (bgs). It generally consists of medium dense sand with varying amounts of silt.
- **Alluvium** was observed below the fill across the terminal and extended to approximately 75 to 85 feet bgs. Loose to medium dense sand with varying amounts of silt and very soft to medium stiff silt with varying amounts of sand (**ESU-2**) were observed at relatively shallower depths within the alluvium to approximately 15 to 34 feet bgs. A layer consisting of very soft to soft silt and clay with varying amounts of sand (**ESU-3**) was observed below ESU-2 to approximately 25 to 37 feet bgs. Organic silts were observed at the two borings located around the southwest corner of the site (B-1-99 and B-2-99). A layer consisting of medium stiff to stiff silt with varying amounts of sand and loose to medium dense

sand with varying amounts of silt (**ESU-4**) was observed underneath to approximately 40 to 50 feet bgs. Below ESU-4, silt with varying amounts of sand (**ESU-5**) was observed to approximately 70 to 80 feet bgs with soil consistency increasing with depth from soft to medium stiff. Medium stiff to hard silt with varying amounts of sand (**ESU-6**) was observed underneath to approximately 75 to 85 feet bgs.

- **CRB (ESU-7)** was observed below the alluvium and extended to the depths explored. The CRB generally consists of fresh, fine-grained, vesicular basalt with slight secondary mineralization within the vesicles.

The site plan shown in Figure 2 includes the approximate locations of the existing subsurface explorations that have been reviewed. The corresponding logs and the associated laboratory testing are provided in Appendix A.1.

2.3 GROUNDWATER CONDITIONS

Based on the pore pressure dissipation data reported from the existing subsurface explorations, the groundwater table was estimated to be on the order of 16 to 23 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 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, which 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. Two representative cross sections were developed across the terminal as shown in Figures 3 and 4 for use in the geotechnical assessment. The locations of the two cross sections are shown in Figure 2. Our interpretation of the subsurface conditions is also depicted in Figures 3 and 4. Due to the lack of subsurface information on the water side, in-land soil profiles were horizontally extended to the water side. Considering the water level in the Willamette River, a design groundwater table was assumed to be 13 feet bgs.

3.1.1 Seismic Design Parameters

As stated in Section 2.4, the site is a seismic Site Class F per ASCE 7-16 Section 20.3 due to the presence of potentially liquefiable soils on site; therefore, site response analysis is required to determine the seismic design parameters for this site.

A site-specific ground response analysis (GRA) was completed per ASCE 7-16 to develop the site-specific risk-targeted maximum-considered earthquake (MCR$_R$) horizontal response spectrum. Table 1 and Figure 5 present our recommended site-specific MCE$_R$ response spectrum. Please refer to Appendix A, Section A.4 for additional details regarding the development of the recommended site-specific MCE$_R$ response spectrum.

TABLE 1. RECOMMENDED SITE-SPECIFIC MCE$_R$ RESPONSE SPECTRUM

PERIOD (SEC)	5% DAMPED SPECTRAL ACCELERATION (G)
0.01	0.57
0.05	1.02
0.075	1.07
0.1	1.02
0.2	1.00
0.3	1.14
0.4	1.06
0.5	1.06
0.75	1.12
0.8	1.16
0.9	1.15
1	1.00
1.5	0.66
2	0.50
3	0.30
4	0.21
5	0.15
6	0.12
7	0.10
8	0.09
9	0.08
10	0.07
11	0.06
12	0.06
12.2	0.06

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 and design ground acceleration. 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. A non-linear effective stress site-specific response analysis (ESA) was performed to evaluate liquefaction potential of the site soils more rigorously by accounting for the effects of excess pore pressure generation during strong ground shaking. Please refer to Appendix A, Section A.4 for additional details regarding the ESA.

The maximum excess pore pressure ratio (Ru) was estimated from performing ESA. When Ru equals to 1.0, soils are considered fully liquefied; and a value of 0.8 indicates triggering of liquefaction that represents a factor of safety against liquefaction (FS_{liq}) of 1.0 per Boulanger et al. (1998). Per Oregon Department of Transportation (ODOT) Geotechnical Design Manual (GDM) 2023, liquefaction is conservatively predicted to occur when the FS_{liq} is less than 1.1; and a FS_{liq} of 1.1 or less also indicates the potential for liquefaction-induced ground movement. To account for this, we selected a Ru equal to 0.7 as our criterion to evaluate the liquefaction potential of the site soils and the corresponding liquefaction-induced settlement. The maximum Ru profiles estimated from our ESA are presented in Figure 6. As shown in Figure 6, ESU-1, ESU-2, and ESU-4 are susceptible to liquefaction under an MCE event; therefore, the depth of liquefaction was estimated along the bottom of ESU-4 (approximately 40 to 50 feet bgs).

Based on the estimated depth of liquefaction, the liquefaction-induced free-field ground settlement of the potentially liquefiable soils was estimated using the semi-empirical approaches (simplified procedures) proposed by Tokimatsu and Seed (1987); Ishihara and Yoshimine (1992); and Idriss and Boulanger (2008). Our analyses indicate that the site under existing conditions could experience a liquefaction-induced free-field ground settlement on the order of 6 to 15 inches under an MCE event with a magnitude (Mw) 9.0 and a site-specific geometric mean MCE peak ground acceleration (PGA_M) of 0.55g derived from our site-specific GRA.

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 soil 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 7 summarizes the earthquake-induced lateral ground deformations we estimated resulting from a Mw 9.0 earthquake event with MCE_R ground shaking intensity using simplified procedures. Under existing conditions (soils liquefy but shaking has stopped), flow failure is predicted to occur within approximately 400 feet behind riverbank. This area is highlighted in red in Figure 7. The blue area shown in Figure 7 extends from approximately 400 feet behind riverbank to the inland site boundary and represents the zone where flow failure is not predicted but significant lateral deformations could still result. Please refer to Appendix A, Section A.5 for additional details regarding the evaluation of the earthquake-induced lateral ground deformations.

The simplified procedures predict large lateral deformations due to a design earthquake. However, we do not believe these deformations are realistic. The simplified procedures for estimating lateral deformation, such as conventional slope stability analyses 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 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 models capture the significant dampening that occurs as the soils displace.

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

The Form 2 Checklist for tanks has been initiated by RM and is included in Appendix B.

3.2.1 Tank Characteristics

The site houses 31 steel storage tanks. The tanks are used for storage of bulk petroleum products including asphalt, bunker oil, high sulfur diesel, low sulfur diesel, biodiesel, and other products. The tanks vary in age with the oldest being built in the early 1950s and the newest built in 2020. Many of the facility's tanks were built in the 1970s. The tanks also vary in size with the smallest having a total capacity of 110 barrels (bbls) and the largest having a total capacity of 280,650 bbls. The maximum operating capacity of all combined tanks is 901,416 bbls. Eleven (11) of the tanks are located within a Tank Farm area. The Tank Farm is surrounded by an earthen containment berm that varies in height between 8 to 12 feet. The remaining 20 tanks are located within the Asphalt Plant area to the southwest of the Tank Farm. The Asphalt Plant is surrounded by an approximately 3.2-foot-hight concrete wall.

Most of the tanks on the site are ground supported steel tanks. Tanks 34 through 38 are slightly elevated on steel leg pedestals. Most of the tanks are also unanchored; however, some have conventional steel tank anchors, and the pedestals of Tanks 34 through 38 are also anchored. A list of the site's tanks and some relevant tank characteristics is presented in Table 2. 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.

TABLE 2. FACILITY TANK INFORMATION SUMMARY

LOCATION WITHIN FACILITY	TANK IDENTIFIER	YEAR BUILT	CONTENTS	NOMINAL HEIGHT (FEET)	DIAMETER (FEET)	MAX. OPERATING CAPACITY (BBLS)	ANCHORAGE TYPE
Tank Farm	1	1974	Asphalt	48	200	241,011	Unanchored
Tank Farm	2	1973	Asphalt	40	225	266,617	Unanchored
Tank Farm	4	1974	Asphalt	40	200	211,667	Unanchored
Tank Farm	5	1975	Bio Diesel	35	11.5	615	Unanchored
Tank Farm	6	1975	Bio Diesel	35	11.5	615	Unanchored
Tank Farm	7	1977	Diesel	48	100	60,137	Unanchored
Tank Farm	8	1979	Diesel	48	100	60,629	Unanchored
Tank Farm	9	1975	Bio Diesel	40	45	11,340	Unanchored
Tank Farm	10	1976	Bio Diesel	41.5	45	11,331	Unanchored
Tank Farm	11	1974	Oil Water Slop	26	11.5	486	Unanchored
Tank Farm	12	1974	Oil Water Slop	17	10	230	Unanchored
Asphalt Plant	15	1970 (Est.)	Asphalt Flux	29.5	11	500	Unanchored
Asphalt Plant	16	1970 (Est.)	Asphalt Flux	33	12	665	Unanchored
Asphalt Plant	18	1970 (Est.)	Anti-Strip	18	6	91	Unanchored
Asphalt Plant	19	1950 (Est.)	Asphalt	40	42.5	9870	Unanchored
Asphalt Plant	20	1950 (Est.)	Asphalt	40	42.5	10,115	Unanchored
Asphalt Plant	21	1950 (Est.)	Asphalt	40	42.5	10,115	Unanchored
Asphalt Plant	22	1950 (Est.)	Flux Oil	14.75	13	350	Anchored
Asphalt Plant	23	1954	Oil Water Slop	14.75	13	350	Anchored
Asphalt Plant	24	2000	Concentrate	40	20	2,240	Anchored
Asphalt Plant	25	2000	Concentrate	40	20	2,240	Anchored
Asphalt Plant	26	2000	Concentrate	40	20	2,240	Anchored
Asphalt Plant	27	2000	Concentrate	40	20	2,240	Anchored
Asphalt Plant	28	1950 (Est.)	Not Used	-	-	-	-

LOCATION WITHIN FACILITY	TANK IDENTIFIER	YEAR BUILT	CONTENTS	NOMINAL HEIGHT (FEET)	DIAMETER (FEET)	MAX. OPERATING CAPACITY (BBLS)	ANCHORAGE TYPE
Asphalt Plant	29	1970 (Est.)	Not Used	-	-	-	-
Asphalt Plant	33	2005	Polyphosphoric Acid	16	7.58	129	Anchored
Asphalt Plant	34	2020	Asphalt	Unk	13.5	Unk	Pedestal w/ Anchors
Asphalt Plant	35	2020	Asphalt	Unk	13.5	Unk	Pedestal w/ Anchors
Asphalt Plant	36	2020	Asphalt	Unk	12	Unk	Pedestal w/ Anchors
Asphalt Plant	37	2020	Asphalt	Unk	12	Unk	Pedestal w/ Anchors
Asphalt Plant	38	2020	Asphalt	Unk	12	Unk	Pedestal w/ Anchors

3.2.2 Tank Risk Categories Per OSSC, IBC, and ASCE 7

Every structure designed in accordance with the 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 McCall 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

3.2.3.1 OVERVIEW

Seismic design requirements for new building and non-building structures changed significantly throughout the 20th century. In general, seismic design requirements have become stricter 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 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.

3.2.3.2 SELECTED TANKS EVALUATED

At this time, a portion of the site's steel storage tanks have been selected for seismic evaluation. This is because many of the site's tanks were built at similar times and have similar or the same dimensions and structural design. As such, it should not be necessary to individually evaluate each tank as the results from tanks of similar vintage, design and construction can be extrapolated to each other. Tank dimensions and other characteristics are listed in Table 2. At this time, the following tanks have undergone seismic evaluation: Tank 1, 2, 4, 7, 8, 9, 10, 15, 19, 24 and 34.

3.2.3.3 TANK INFORMATION AVAILABLE FOR REVIEW

Few drawings and other records providing detail about the construction and properties of the site's existing steel storage tanks exist. The majority of the tank dimensions and properties used as part of this evaluation were obtained from existing API 653 inspection reports provided by McCall Companies. This section discusses information about the tanks that is lacking and additional information that must be collected in the future.

- **Foundation Information:** The existing API 653 inspection reports list some of the site's tank foundations as concrete ring wall, concrete pad, or gravel. However, dimensions and other foundation properties are not identified. It is implied that all the tanks have shallow foundations, or, in the case of "gravel," it is implied that there is no concrete foundation at all. The precise foundation dimensions and other properties of the concrete foundations are not known. Additional investigation should be undertaken to better understand and catalog the existing foundation configurations, dimensions, and properties.
- **Steel Material Properties:** The steel material properties such as ASTM designation, tensile yield strength and tensile ultimate strength are not known for the site's existing steel storage tanks. As such, conservative material property assumptions have been made where possible to accomplish the seismic evaluation. However, additional investigation should be undertaken to better understand and catalog the steel material properties of the existing tanks.
- **Interior Column Roof Supports:** The cross-section configuration, location and other information about interior column roof supports is unknown for most of the tanks. The tanks for which this information is unknown are Tank 1, 4, 9, 10, and 19. Additional investigation should be undertaken to better understand and catalog the configuration and properties of the interior column roof supports.
- **Tank Height and Shell Thickness:** Most of the tank heights and shell thicknesses are known from the existing API 653 inspection reports. However, this information is not known for Tank 34. Additional information must be gathered for Tank 34.

3.2.3.4 SOIL LIQUEFACTION AND LATERAL SPREADING POTENTIAL

GeoEngineers 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 and Appendix A 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 and lateral deformation. 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.

At this time, GeoEngineers has only used high-level, simplified, formulations 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 calculate 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 GeoEngineers conducts additional engineering work for the ground deformations, the ability of the tanks to accommodate the ground deformation will be evaluated in more detail.

3.2.3.5 TANK OVERTURNING

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. New tanks with low seismic overturning forces will have a low potential for uplift and may be permitted to be unanchored. However, new 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 stricter over time.

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.

Table 3 shows a summary of the overturning analyses for the evaluated tanks. Within the summary table, there is a column that indicates whether anchorage would be required if the tank were new. The information in this column is determined based on the procedures in API 650 Section E.6.2.1. For anchorage to not be required, it must be the case that the calculated anchorage ratio, J, is less than 1.54 and the shell compression requirements for self-anchored tanks are satisfied.

TABLE 3. TANK OVERTURNING SUMMARY TABLE

TANK IDENTIFIER	NOMINAL HEIGHT (FEET)	DIAMETER (FEET)	HEIGHT/DIAMETER ASPECT RATIO	ANCHORAGE TYPE	ANCHORAGE REQUIRED IF TANK WERE NEW?	ADDITIONAL INVESTIGATION WARRANTED FOR OVERTURNING?
1	48	200	0.24	Unanchored	No	No
2	40	225	0.18	Unanchored	No	No
4	40	200	0.20	Unanchored	No	No
7	48	100	0.48	Unanchored	No	No
8	48	100	0.48	Unanchored	No	No
9	40	45	0.89	Unanchored	Yes	Yes
10	41.5	45	0.92	Unanchored	Yes	Yes
15	29.5	11	2.68	Unanchored	Yes	Yes
19	40	42.5	0.94	Unanchored	Yes	Yes
24	40	20	2.00	Anchored	Yes	No, Already Anchored
34	Unknown	13.5	Unknown	Pedestal w/ Anchors	Yes	No, Already Anchored

3.2.4 Tank Structural Fragility Assessment

3.2.4.1 OVERVIEW

Typical building code documents include simplified seismic design criteria that allow for quick and efficient design of structures that are generally deemed to be safe. These procedures include seismic amplification and reduction factors that result in approximated design forces. This approach generally works for most structures and leads to infrastructure that is generally reliable. However, it does not reflect the true nature of how earthquakes affect structures. For example, it is not able to accurately predict the repercussions from cyclic shaking that causes plastic deformation of yielding elements within a structure as occurs in a real earthquake.

The typical simplified building code procedures are also not able to accurately identify damage states that are expected to occur given certain levels of earthquake shaking. As summarized in Section 1.1, the OAR 340-300 performance objective is to limit the potential for a product spill. As such, the building code procedures contained in ASCE 7 and API 650 are not sufficient on their own to assess the potential for a spill as these documents do not allow for this type of assessment.

An alternative approach is to use structural fragility analysis to estimate the type and amount of damage that might result from certain levels of shaking. Fragility analysis uses databases of how real-life structures have performed in past earthquakes and associates different levels of performance with demand parameters such as peak ground acceleration.

3.2.4.2 ANALYSIS APPROACH AND NEED FOR ADDITIONAL ANALYSIS

Reid Middleton discussed our team's tank fragility assessment approach with the Oregon DEQ in December 2023 and January 2024. It was indicated that this approach is likely to be acceptable, however, the Department is likely to require nonlinear finite element analysis of tank components in order to ensure that the tank fragility functions are sufficiently tailored to match the real tank properties.

At this time, it has not been possible to conduct detailed nonlinear finite element analysis of each tank. As noted elsewhere in this report, additional geotechnical investigation and analysis must be undertaken to more-accurately estimate the ground deformation due to soil liquefaction and lateral spreading. As such, the following sections of this report outline expected tank performance using default fragility equations contained in the FEMA HAZUS Earthquake Model Technical Manual (FEMA, 2020). However, additional analysis will likely be required to ensure that the fragility functions are sufficiently tailored to match the real tank properties.

However, it should be noted that the default fragility functions for ground supported steel tanks that are contained within HAZUS were developed in the 1990s and prior. Some studies since the 1990s have concluded that the default fragility functions in HAZUS are overly conservative and real-world ground supported steel tank performance might be better than could be predicted by HAZUS (O'Rourke, 2000). So, it may be the case that the default fragility functions are acceptable for conservatively estimating tank performance. Additional analysis is likely to confirm whether this is the case.

3.2.4.3 TANK FRAGILITY ANALYSIS RESULTS EXCLUDING LIQUEFACTION AND LATERAL SPREADING

Table 4 lists information related to the analyzed tanks and lists whether the tank is expected to leak in the design earthquake. The information is based on the results of the fragility analysis but excludes consideration for ground deformation. This could reflect either that liquefaction or lateral spreading does not occur or could reflect a site where liquefaction and lateral spreading have been mitigated. The fragility functions used for the tanks are those from the FEMA HAZUS Earthquake Model Technical Manual Section 8.1.6.4. By HAZUS definition, leaking of tank contents is expected to occur when a tank is in the "extensive" damage state or the "complete" damage state. Leaking of a tank is "expected" in the design earthquake when the probability of being in the extensive or complete damage state exceeds 50 percent. In this case, it means that it is more-likely-than-not that a tank will leak. Conversely, if this probability is less than 50 percent, then it is more likely that the tank will not leak.

TABLE 4. TANK FRAGILITY ANALYSIS RESULTS EXCLUDING LIQUEFACTION AND LATERAL SPREADING

LOCATION WITHIN FACILITY	TANK IDENTIFIER	HEIGHT/DIAMETER ASPECT RATIO	ANCHORAGE TYPE	LEAKING EXPECTED IN THE DESIGN EARTHQUAKE?
Tank Farm	1	0.24	Unanchored	No
Tank Farm	2	0.18	Unanchored	No
Tank Farm	4	0.20	Unanchored	No
Tank Farm	7	0.48	Unanchored	No
Tank Farm	8	0.48	Unanchored	No
Tank Farm	9	0.89	Unanchored	No
Tank Farm	10	0.92	Unanchored	No
Asphalt Plant	15	2.68	Unanchored	Additional Analysis Recommended
Asphalt Plant	19	0.94	Unanchored	No
Asphalt Plant	24	2.00	Anchored	No
Asphalt Plant	34	Unknown	Pedestal w/ Anchors	Additional Analysis Recommended

3.3 PIPELINES

3.3.1 Overview

The facility has a number of bulk petroleum pipes that cross the site. These pipes serve to transfer petroleum products between storage tanks, allow for the mixing of asphalt products and allow for the loading and unloading of vehicles for transporting petroleum products. The site can receive products via railcar, can load and unload tanker ships and barges via its pier/dock and can load and unload trucks. Some of the piping is buried beneath soil and some of the piping is above grade.

A majority of the length of piping exists within the boundaries of large secondary containment areas surrounding the Tank Farm and the Asphalt Plant. The only places where the piping is outside the boundaries of these areas is where the piping extends out to the pier/dock, where products are loaded on trucks or railcars, where piping extends between the Tank Farm and the Asphalt Plant and a small quantity of pipes that extend off the site to other nearby facilities. Otherwise, piping is within the boundaries of the large secondary containment areas.

Even in areas outside of the large secondary containment areas there are additional features such as curbs, catchments and drains that are intended to capture petroleum products in the event of their accidental release. For example, the railcar unloading area is equipped with a 560-bbl capacity containment area with a 6-inch-tall concrete containment curb, the four-bay truck loading rack is surrounded by an 8-inch-tall concrete containment curb with a capacity in excess of 2,800 bbls. Additional information about spill prevention, control and countermeasures can be found in the facility's Spill Prevention, Control and Countermeasure (SPCC) Plan. The most recent version of this plan was published in April 2023.

3.3.2 Piping Network Vulnerability Assessment Results

GeoEngineers identified the site as having potentially liquefiable soil. In addition, lateral spreading is noted as being a possibility across the site in a seismic event with larger lateral movements near the bank of the

Willamette River. Please see Section 3.1 and Appendix A for additional information. At this time, GeoEngineers has only used high-level, simplified, formulations 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 calculate the expected ground deformations in a seismic event.

The impact of lateral ground deformations and seismic shaking forces on the pipelines still need to be assessed. However, it should be noted that a majority of the length of the site's piping exists within the boundaries of the large secondary containment areas surrounding the Tank Farm and the Asphalt Plant. Also, most of the pipes across the site are 8-inch diameter and smaller (many of the pipes are insulated which makes them appear larger). This means that at any one time the total volume of product sitting idle within an individual pipe segment is relatively small in comparison to the total storage capacity of the facility. In addition, in the absence of soil liquefaction, any product that escapes a pipeline that is within a secondary containment area is likely to be contained by that system in a manner that is consistent with the facility's SPCC plan.

For the relatively short portions of pipes that are not within the boundaries of secondary containment areas, the site is sloped to drain to catch basins that lead to the storm drain system and ultimately to the oil/water separator. In the absence of soil liquefaction, it is likely that any product that escapes from one of these pipelines would drain to the catch basins and end up in the oil/water separator. The oil/water separator capacity is approximately 2,250 cubic feet or 16,831 gallons and has a design flow rate of 175 gallons per minute (Anchor QEA, 2023). For an 8-inch diameter pipe, the capacity of the oil/water separator equates to the amount of product that could be contained in more than a 1-mile-long length of pipe.

The Form 3 Checklist for pipes and pipeline system has been initiated by RM and is included in Appendix C.

3.4 PIERS AND WHARVES (DOCK)

3.4.1 Overview

The facility has a pier/dock on the Willamette River. The dock is used for loading and offloading petroleum products from tankers and barges (Norwest Engineering, 2019). The dock was designed and built in 1974-1975. The original supports for the dock approach, pier head, and personnel walkways are creosote-treated timber framing and timber piles; however, several members have been replaced with steel elements during repairs completed over the years. The dock approach surface, pier head surface, and personnel walkways themselves are constructed of reinforced concrete. Upstream and downstream mooring dolphins, breasting dolphins, and barge fender are supported by steel H-piles. Several pipes extend from the shore onto the pier/dock for the purposes of transferring petroleum products.

The pier/dock is approximately 930 feet long parallel to the shore. The approach trestle extending out from the shore to the service platform is approximately 193 feet long. The service platform in the center area of the pier/dock has plan dimensions of approximately 40 feet wide by 113 feet long. The top of riverbank elevation at the entrance to the approach trestle is approximately 28 feet above the water line. The mudline beneath the service platform is approximately 25-30 feet below the water line.

The pier/dock was originally designed to berth a "T-5 Class Tanker". The approach trestle and service platform have five product lines that extend out on them. These existing product lines are intended for conveying asphalt, diesel, oil, and storm water/slop. An above and below water dock inspection and condition assessment was conducted in 2018, and another is currently in progress.

3.4.2 Pier/Dock Vulnerability Assessment Results

GeoEngineers identified the site as having potentially liquefiable soil. In addition, lateral spreading is noted as being a possibility in a seismic event along the bank of the Willamette River. Please see Section 3.1 and Appendix A for additional information. At this time, GeoEngineers has only used high-level, simplified, formulations 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 calculate the expected ground deformations in a seismic event.

The approach trestle and service platform are primarily constructed of timber piles with wood lateral bracing, and the deck is constructed of reinforced concrete. Additional analysis of the structure is required once the magnitude of the ground deformation is more precisely understood.

Regardless, structural strengthening, improvement, or upgrade of the existing pier/dock may not be necessary to meet the OAR 340-300 performance objective. There could be a variety of approaches taken to mitigate the possibility of exceeding the performance objective.

The Form 4 Checklist for piers and wharves (docks) has been initiated by RM and is included in Appendix D.

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

As shown in Figure 2, an approximate 8 to 12-foot-high earthen berm capped in asphalt surrounds the Tank Farm ASTs, and an approximate 3.2-foot-high concrete wall surrounds the Asphalt Plant ASTs.

The earthen berm has a capacity of approximately 340,000 bbls, which is approximately 120 percent of the largest AST volume. The bermed area is graded to drain to four catch basins. The catch basins discharge to an oil/water separator. The berm outlet is controlled by a positive seal gate valve, which is normally locked and closed. Approximately three-fourths of the earthen berm is located within the potential flow failure zone as shown in Figure 7. The earthen berm would be subjected to the same deformations as the adjacent ground if lateral spreading were to occur.

The walled area has a containment capacity of approximately 10,554 bbls, which is approximately 109 percent of the largest AST volume. Runoff from the Asphalt Plant drains to a catch basin and into a drainage sump. The manually controlled sump pump discharges onto the ground in drainage area, where it would then flow into a catch basin before being routed to an oil/water separator. The concrete wall is located beyond the potential flow failure zone identified in Figure 7, but still within an area subject to potentially large lateral displacements during a design earthquake.

The Form 6 Checklist for berms and dikes has been initiated by RM and GeoEngineers and is included in Appendix F.

3.7 BUILDINGS AND BUILDING-LIKE STRUCTURES

The facility's site does not have any buildings that serve a product storage or product handling function. Therefore, the Form 7 Checklist for buildings and building-like structures (Appendix G) is not applicable for this facility to meet the OAR 340-300 performance objective.

3.8 FIRE DETECTION AND SUPPRESSION

The facility has a variety of fire protection systems and fire prevention processes. At this time, little information has been able to be gathered regarding the cataloging of fire detection and fire protection systems present at the terminal site. There is no existing database of all the existing equipment. Additional data gathering may be necessary in the next phase.

The Form 8 Checklist for fire detection and suppression has been initiated by RM and is included in Appendix H.

3.9 CONTROL SYSTEMS

The terminal has no centralized facility control system. All control equipment is operated manually, in its location, and only at the time it is needed. Most of the time control equipment sits idle and not in use. At this time, little information has been able to be gathered regarding the cataloging of all the control equipment present at the terminal site. There is no existing database of the existing equipment. Additional data gathering may be necessary in the next phase.

The Form 9 Checklist for control systems has been initiated by RM and is included in Appendix I.

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 soils samples for laboratory testing. The proposed additional subsurface characterization include geophysical surveys and geotechnical borings.

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

- **One (1)** two-dimensional (2D) multi-channel analysis of surface waves (MASW) testing to develop a 2D Vs profile across the site to capture site variability.
- **Two (2)** one-dimensional (1D) MASW or Refraction Microtremor (ReMi) sounding to capture the site-specific Vs to approximately 100 feet bgs. The results will be compared with the Vs measured

through the previous seismic CPTs and used to refine the design Vs profiles for use in our site-specific GRA.

- **Two (2)** 1D local microtremor array method (MAM) to capture the site-specific Vs to approximately 200 to 400 feet bgs. Based on our review of the existing subsurface information within the site vicinity, CRB was observed approximately 75 to 85 feet bgs; therefore, the engineering bedrock/firm ground typically with a Vs of 2,500 feet per second (ft/sec) was anticipated to be encountered within this depth range. The results will be used to refine the design Vs profiles for use in our site-specific GRA.

We propose to complete three (3) 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 mainly includes moisture content, percent fines, sieve analysis, and Atterberg Limit tests (plasticity characteristics). We also proposed to conduct up to nine (9) 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.

1D total and effective stress site-specific GRA will be refined to update 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.

2D numerical modeling will be completed on up to two (2) representative cross sections 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.

4.2 ADDITIONAL STRUCTURAL AND OPERATIONAL SYSTEM ASSESSMENTS

Upon the development of more reliable ground deformation estimates, detailed analyses, consistent with the methodologies described in the DEQ’s roadmap 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

4.3 PROPOSED SCHEDULE

The total duration for completing the remaining work may take up to 14 months upon receiving approval from Oregon DEQ to continue with the phased approach. Below summarized the proposed schedule:

- Complete additional geotechnical subsurface explorations and laboratory testing (Section 4.1.1) – 3 to 4 months upon approval.
- Complete additional geotechnical engineering analysis (Section 4.1.2) – 6 to 7 months upon approval.
- Complete structural and system assessments (Sections 4.2 through 4.9) – 11 to 12 months upon approval.
- Complete a final seismic vulnerability assessment report – 13 to 14 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 McCall 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 J, Report Limitations and Guidelines for Use, for additional information pertaining to the use of this report.

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Figures

Source(s):
- ESRI

Coordinate System: NAD 1983 UTM Zone 10N

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Vicinity Map

McCall Terminal Seismic Vulnerability Assessment
Portland, Oregon

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Figure 1

P:\26\26662001\GIS\26662001_Project\26662001_Project.aprx\2666200100_F01_VicinityMap Date Exported: 05/07/24 by ccabrera

Note(s):

Source(s):
- Aerial from Google Earth Pro, dated 6/14/2022.
- Lidarr from Metro dataset 2014.
- Bathymetry from Willamette River dataset 2003.

Coordinate System: NAD83 Oregon State Planes (Polyconic), North Zone, US Foot.

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Legend

B-1	⊕	Boring by Landau Associates, 2022
CPT-1	▲	Cone Penetration Test by Landau Associates, 2022
B-1-99	◎	Boring by PacRim Geotechnical Inc., 1999
PSI-CPT-1	▲	Cone Penetration Test by Intertek PSI, 2019
GP-1	▣	Geoprobe Test by Intertek PSI, 2019

A A' Cross Section Location

N

0 200
Feet

Site Plan

McCall Terminal Seismic Vulnerability Assessment
Portland, Washington

GeoEngineers

Figure 2

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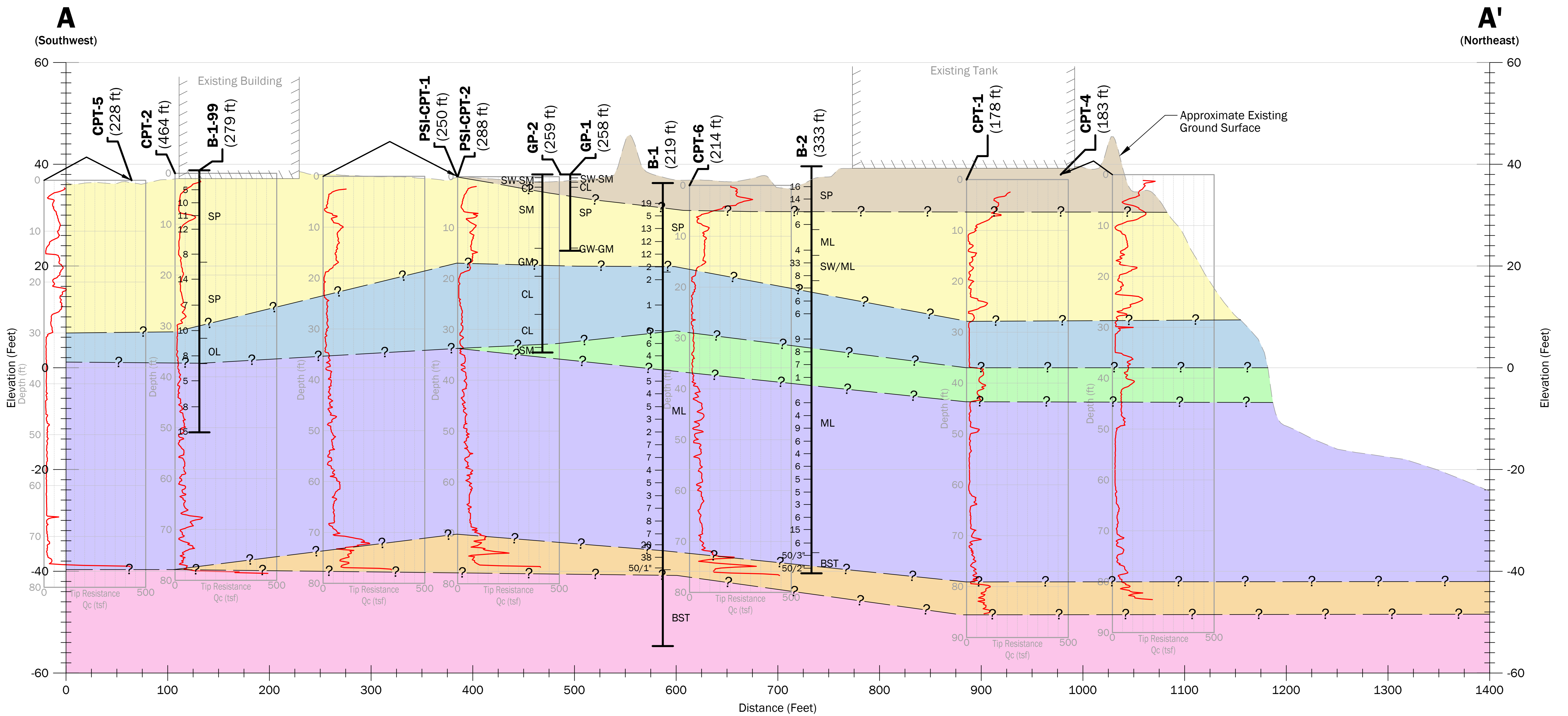

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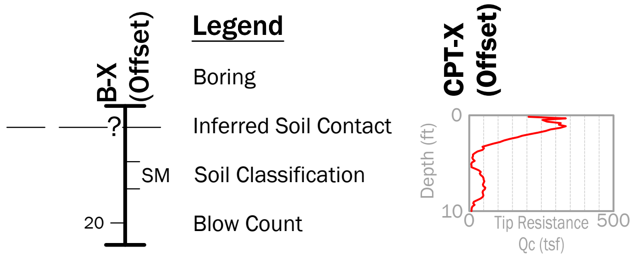
Note(s):

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

Legend



Boring
Inferred Soil Contact
Soil Classification
Blow Count

Cone Penetrometer Test (CPT)
CPT Tip Resistance in Tons/Square Foot (tsf)

🟫	Medium Dense SP (Fill)
🟨	Loose to Medium Dense SP/SP-SM/SM and Very Soft to M. Stiff ML (Fill/Alluvium)
🟦	Very Soft to Soft ML/CL (Alluvium)
🟩	Medium Stiff to Stiff ML and Loose SP-SM/SM (Alluvium)
🟪	Very Soft to Medium Stiff ML (Alluvium)
🟧	Medium Stiff to Hard Silt (Alluvium)
🟥	Columbia River Basalt (CRB)

0 100
Horizontal Feet
0 20
Vertical Feet
Vertical Exaggeration = 5X

Cross Section A-A'

McCall Terminal Seismic Vulnerability Assessment
Portland, Washington

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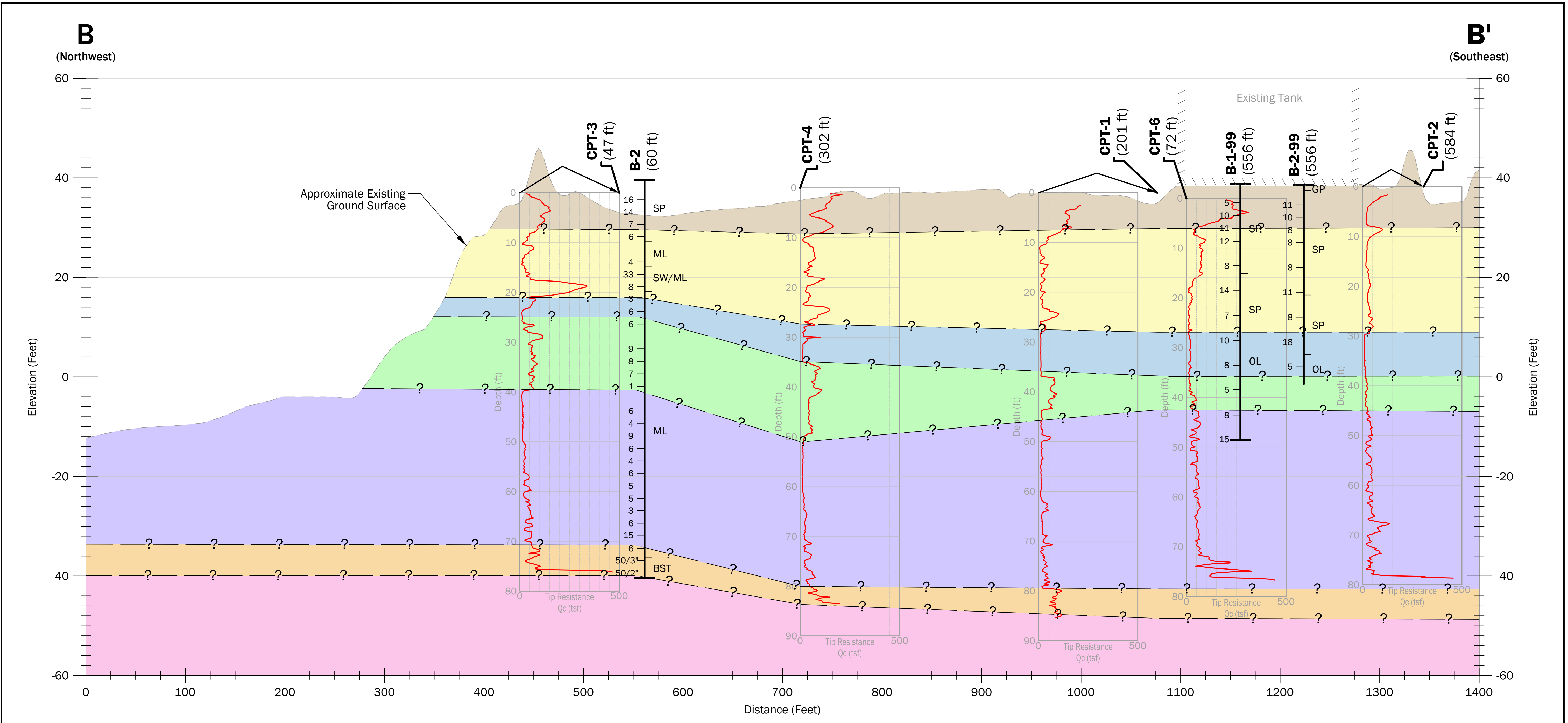
Figure 3



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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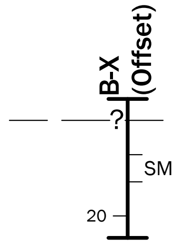
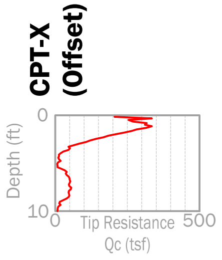


Boring

Inferred Soil Contact

Soil Classification

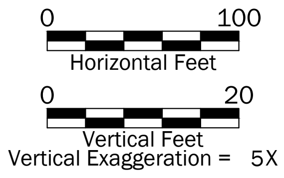
Blow Count



Cone Penetrometer Test (CPT)

CPT Tip Resistance in Tons/Square Foot (tsf)

🟫	Medium Dense SP (Fill)
🟨	Loose to Medium Dense SP/SP-SM/SM and Very Soft to M. Stiff ML (Fill/Alluvium)
🟦	Very Soft to Soft ML/CL (Alluvium)
🟩	Medium Stiff to Stiff ML and Loose SP-SM/SM (Alluvium)
🟪	Very Soft to Medium Stiff ML (Alluvium)
🟧	Medium Stiff to Hard Silt (Alluvium)
🟥	Columbia River Basalt (CRB)



Vertical Exaggeration = 5X

Cross Section B-B'

McCall Terminal Seismic Vulnerability Assessment
Portland, Washington

GeoEngineers

Figure 4

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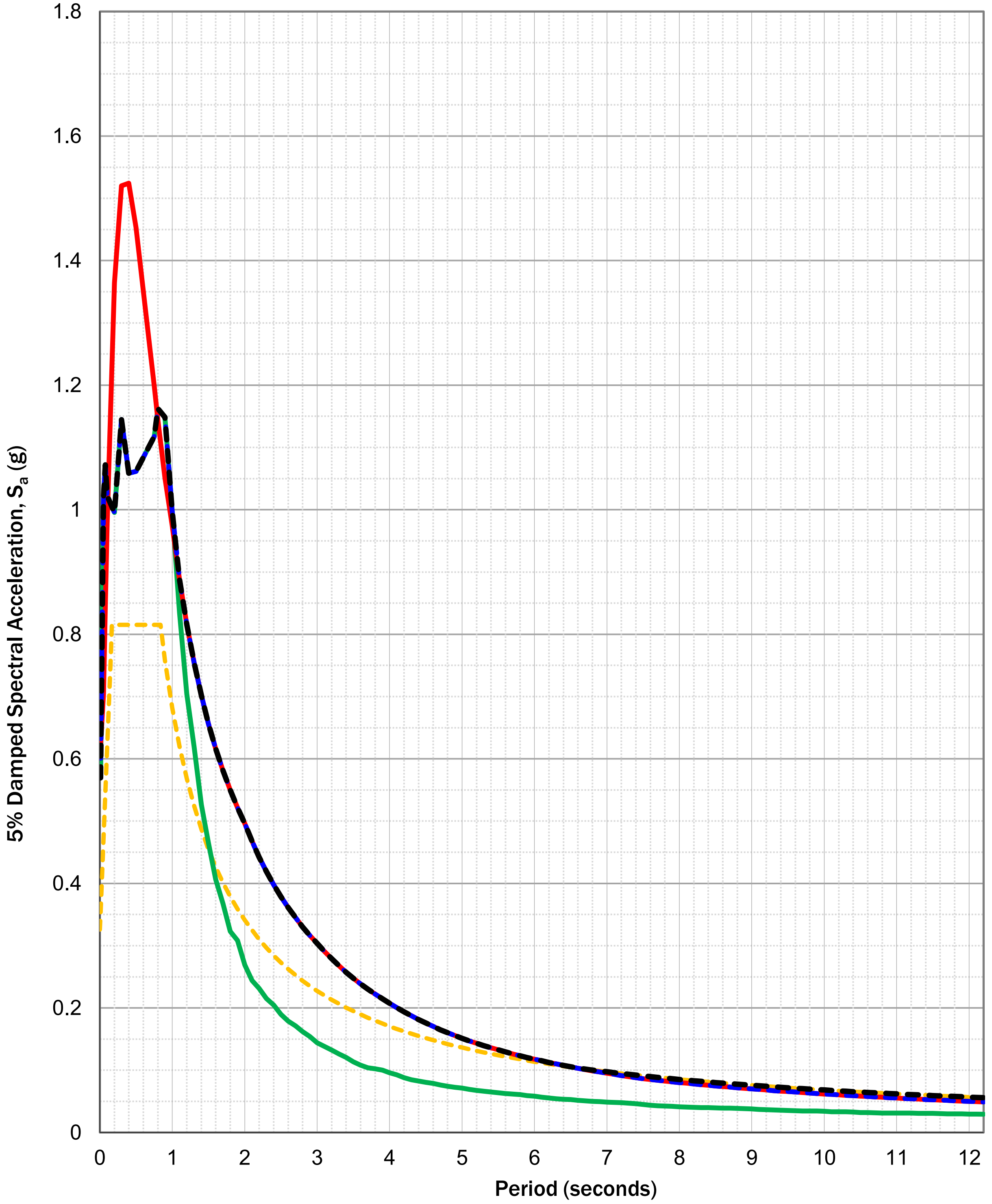

26662-001-00 Date:05/03/2024

Recommended Site-Specific MCE_R Response Spectrum

McCall Terminal Seismic Vulnerability Assessment
Portland, Oregon

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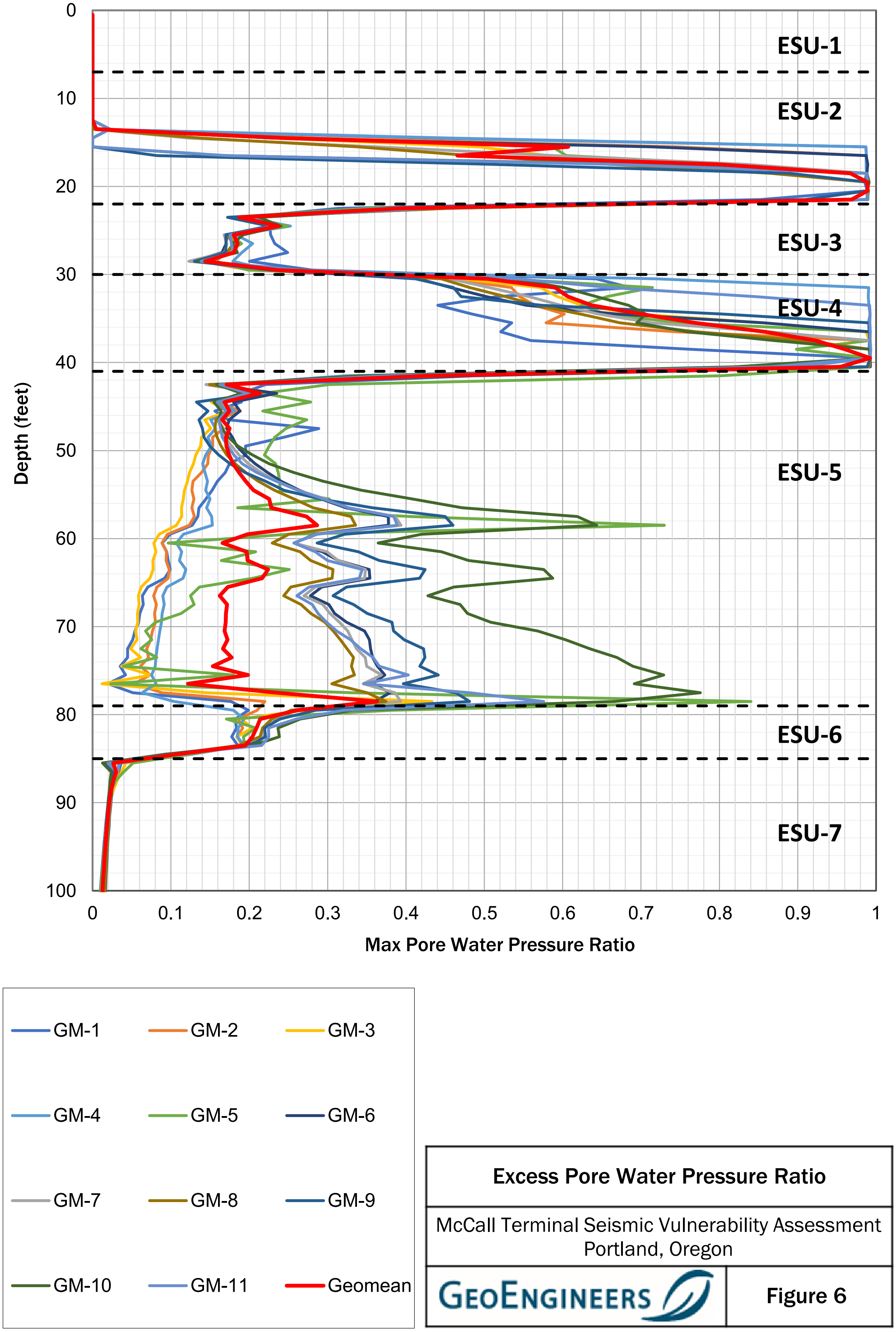
Figure 5



Excess Pore Water Pressure Ratio

McCall Terminal Seismic Vulnerability Assessment
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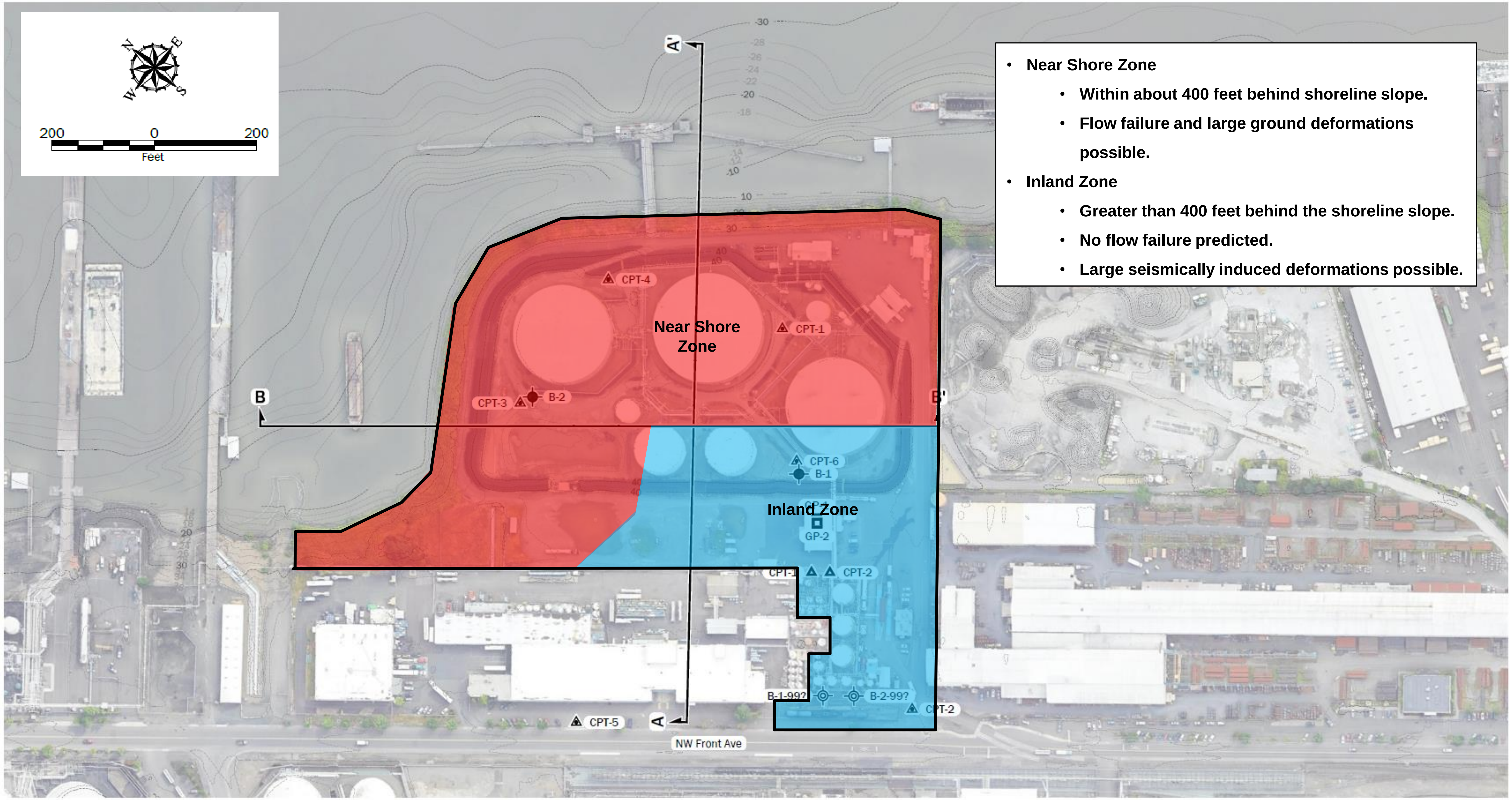
Figure 6

26662-001-00 Date Exported: 05/03/24



- **Near Shore Zone**
  - **Within about 400 feet behind shoreline slope.**
  - **Flow failure and large ground deformations possible.**
- **Inland Zone**
  - **Greater than 400 feet behind the shoreline slope.**
  - **No flow failure predicted.**
  - **Large seismically induced deformations possible.**

Notes:
1. The locations of all features shown are approximate.
2. This drawing is for information purposes. It is intended to assist in showing features discussed in an attached document. GeoEngineers, Inc. cannot guarantee the accuracy and content of electronic files. The master file is stored by GeoEngineers, Inc. and will serve as the official record of this communication.

Data Source: Aerial from Google Earth Pro dated 6/14/2022.
Lidarr from Metro dataset 2014.
Bathymetry from Willamette River dataset 2003.

Projection: NAD83 Oregon State Planes (Polyconic), North Zone, US Foot

Legend

B-1	⊕	Boring by Landau Associates, 2022
CPT-1	▲	Cone Penetration Test by Landau Associates, 2022
B-1-99	⊕	Boring by PacRim Geotechnical Inc., 1999
CPT-1	△	Cone Penetration Test by Intertek PSI, 2019
GP-1	☐	Geoprobe Test by Intertek PSI, 2019

A A' Cross Section Location

Earthquake-Induced Lateral Ground Deformations

McCall Terminal Seismic Vulnerability Assessment
Portland, Oregon

GEOENGINEERS

Figure 7

Appendices

Appendix A

Geotechnical Assessment

Appendix A – Table of Contents

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Appendices

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 Existing Subsurface Investigations

The logs of the existing subsurface investigations that include geotechnical borings and cone penetration tests (CPTs) are presented in this section. The associated laboratory testing results are also presented.

A.2 Historical Geotechnical Reports

The available historical geotechnical reports are presented in this section.

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 plan 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 A.1 and A.2 in this appendix 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 boreholes and Cone Penetration Tests (CPTs) are presented in Figure 2 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: Most of the existing borings and CPTs terminated at approximately 76 to 91 feet below ground surface (bgs). Two borings (B-1 and B-2) encountered the Columbia River Basalt (CRB) at approximately 75 feet bgs and terminated in this layer. CPTs terminated at practical refusal.

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

Response: All the existing borings and 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 boreholes and CPTs along the berms are more than 200 feet. We propose three 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 two-dimensional (2D) multi-channel analysis of surface waves (MASW) to develop 2D Vs profile across the site to capture site variability. The combination of additional geotechnical borings and geophysical survey should be adequate to capture and refine the subsurface conditions across the site.

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: CPT-3 and CPT-6 were compared to the adjacent borings, e.g., B-2 and B-1, respectively. The comparison was incorporated in our development of representative soil engineering properties.

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 Figures 3 and 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: Both simplified liquefaction analysis and effective stress site-specific response analysis (ESA) were used to evaluate the liquefaction potential of the site soils. Please refer to Appendix 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 was completed in this phase. We propose to perform 2D site-specific response analysis to refine the evaluation on the earthquake-induced ground deformations in the next phase.

c. *What ground motion parameters were used?*

Response: We used Mw 9.0 and PGA_M=0.55g from our site-specific ground motion analysis (GRA) in the simplified liquefaction analysis. Please refer to Appendix A.4 for detailed information and the ESA.

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 existing SPT borings and CPTs. The CPT-based correlations were completed using a commercial software, CPeT-IT, developed by GeoLogismiki. The SPT-based correlations were completed based on the following methods: Olsen and Stark (2002), Idriss and Boulanger (2007), Kramer (2008), and Idriss et al. (1998). Please refer to Table A-14 in Appendix A.5 for the developed residual shear strengths.

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

Response: We performed the ESA 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 first, as discussed in Appendix A.4. We then performed the ESA to refine the liquefaction evaluation, as discussed in Appendix 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 refined liquefaction evaluation per ESA. The seismically induced ground settlements were evaluated using semi-empirical approaches (simplified procedures). The seismically induced lateral movements were evaluated using a simplified approach by performing conventional slope stability analyses and Newmark sliding block analyses. Please refer to Sections 3.1.2 and 3.1.3 in the main body of the report and Appendix 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-14 in Appendix 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 Appendix 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 3.1.3 in the main body of this report and Appendix A.5. We estimated the liquefaction-induced settlement using the semi-empirical approaches proposed by Tokimatsu and Seed (1987); Ishihara and Yoshimine (1992); and Idriss and Boulanger (2008), 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: The seismically induced ground deformations evaluated using the simplified procedures are greater than 0.1 feet, as discussed in Sections 3.1.2 and 3.1.3 in the main body of this report and Appendix A.5. However, these simplified methods do not capture the softening and strain-hardening effects that occur as liquefied soils move, and excess pore water pressure is redistributed. Nor do models capture the significant damping that occurs as the soil displace. Therefore, to better understand the likely ground deformations, we propose numerical modeling using constitutive soil models based on detailed laboratory testing to be performed in the next phase.

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. Perform seismic hazard disaggregation for the MCE at the expected structural period(s) of interest and select a suite of horizontal seed ground-motion time histories representing the sources contributing to the total seismic hazard at the site.
4. Modify the time histories to approximately match the target rock outcrop MCE UHS.
5. Complete a site-specific ground response analysis (GRA) that includes total stress analysis (TSA) and effective stress analysis (ESA) to compute ground surface response spectra and corresponding site-specific soil amplification factors (AFs).
6. Develop maximum component adjustment (MCA) factors and risk coefficients.
7. Develop a site-specific probabilistic MCE_R response spectrum by scaling the rock outcrop MCE UHS by MCA factors, risk coefficients, and soil AFs.
8. Complete a deterministic seismic hazard analysis (DSHA) to develop a site-specific deterministic MCE_R response spectrum, as appropriate.
9. 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.
10. Evaluate the liquefaction susceptibility of the subsurface soils based on the ESA results.

SITE CLASS AND MAPPED SEISMIC PARAMETERS

Based on the two existing seismic CPTs (CPT-1 and CPT-6) (locations as shown in Figure 2 in the main body of the report), the time-average shear wave velocity (Vs) in the upper 100 feet (30 meters [m]) (Vs30) values were computed as 694 and 777 feet per second (ft/sec), respectively. Therefore, 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)	0.891
Long-period mapped MCE_R spectral response acceleration, S_1 (g)	0.405
Short-period site coefficient, F_a	1.14
Long-period site coefficient, F_v	2.11[2]
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)	0.85
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)	0.57
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.

[2] Determined based on a 35 percent relative contribution of the Cascadia Subduction Zone interface source to the total hazard for the MCE at T=1.0 second from the United States Geological Survey (USGS) Earthquake Hazard Toolbox and the National Seismic Hazard Model (NSHM) Conterminous U.S. 2023 (Beta) model per 2019 OSSC 1613.2.3.1 Modification.

SITE-SPECIFIC PROBABILISTIC SEISMIC HAZARD ANALYSIS

We completed a site-specific PSHA to develop the rock outcrop MCE UHS using the NSHM Hazard Tool with the Conterminous U.S. 2023 (Beta Version) at Latitude=45.5638˚N and Longitude=122.7352˚S. Table A-2 presents the developed site-specific probabilistic 5 percent damped rock outcrop MCE UHS. We defined rock outcrop conditions as the Vs=2,500 ft/sec (Site Class B/C boundary). The rock outcrop MCE UHS was used as the target for selecting and modifying input ground-motion time histories for use in the site-specific GRA.

TABLE A-2. ROCK OUTCROP MCE UHS, SITE CLASS B/C BOUNDARY

PERIOD (SEC)	5% DAMPED SPECTRAL ACCELERATION (G)
0.01	0.509
0.05	0.750
0.075	0.952
0.1	1.102
0.2	1.170
0.3	0.963
0.4	0.817
0.5	0.691
0.75	0.506
1	0.387
2	0.187
3	0.112
4	0.078
5	0.059
7.5	0.036
10	0.027
11	0.023[1]
12	0.019[1]
13	0.015[1]

Notes:

[1] Extrapolated linearly to cover the range of fundamental periods of the tanks in the terminal.

INPUT GROUND MOTION TIME HISTORY

Tables A-3 and A-4 summarize the fundamental periods of the tanks in the Tank Farm and Asphalt Plant that were selected for seismic evaluation. The impulsive mode is essentially the mode of vibration of steel tank itself and the weight of the fluid excited at the same frequency. The convective mode is the mode of vibration of the wave action of the fluid inside the tank. The fundamental periods were estimated by project structural engineer based on the review of the available structural information of the tanks.

TABLE A-3. TANK FARM FUNDAMENTAL PERIODS

TANK NO.	IMPULSIVE PERIOD (SECONDS)	CONVECTIVE PERIOD (SECONDS)
1	0.36	10.8
2	0.39	12.2
7	0.27	6.0
8	0.27	6.0
9	0.20	3.9
10	0.15	3.9

TABLE A-4. ASPHALT PLANT TANK FUNDAMENTAL PERIODS

TANK NO.	IMPULSIVE PERIOD (SECONDS)	CONVECTIVE PERIOD (SECONDS)
15	0.07	1.8
19	0.15	3.8
24	0.09	2.6

Input Ground Motion Time History Selection

To capture the large range of fundamental periods of the tanks, we performed site-specific seismic hazard disaggregation at the spectral periods of 0.15, 0.4, 4.0, 7.5, and 10.0 seconds to compute the percent contributions of various source-types to the total MCE hazard. Tables A-5 (A-5a and A-5b) and A-6 present the MCE source-type disaggregation results. In the time-history selection, we considered spectral shape, magnitude, arias intensity (AI), and significant duration (e.g., D575 and D595).

Per ASCE 7-16 Section 21.1.1, at least five horizontal ground motion time histories shall be used for site response analysis. We used a suite of eleven single-component horizontal ground motion time histories, which is sufficient to capture the primary sources that contribute to the seismic hazard at the site. Table A-7 presents the number of seed ground motion time histories selected to represent each source-type based on the source type disaggregation results and our judgement.

The crustal sources were subdivided between near-field and far-field sources based on the source-to-site distance for each discretely mapped fault. The sources with a distance less than 15 kilometers (km) were classified as near-field sources and were represented by the fault-normal component of pulse-like crustal ground motions. The far-field crustal sources were represented by far-field crustal ground motions. Table A-8 presents the ground motion time history suite characteristics. We retrieved the ground motions for shallow crustal sources from Next Generation Attenuation (NGA)-West 2 ground motion database (Ancheta et al. 2014) and the ground motions for subduction zone source types from the NGA Subduction ground motion database (Mazzoni 2022).

TABLE A-5a. PERCENT CONTRIBUTION (MCE, SITE CLASS D)

EARTHQUAKE SOURCE TYPE	T=0.15S	T=0.4S	T=4.0S	T=7.5S	T=10S
Crustal Faults with Distance less than 15 km (Near-Field)	33	43	34	18	9
Gridded Background and Other Crustal Faults (Far-Field)	17	12	5	5	4
Cascadia Subduction Zone Intraslab	15	8	0	0	0
Cascadia Subduction Zone Interface	26	28	47	61	68

TABLE A-5b. MAGNITUDE AND DISTANCE RANGE (MCE, SITE CLASS D)

EARTHQUAKE SOURCE TYPE	MAGNITUDE	DISTANCE (KM)
Crustal Faults with Distance less than 15 km (Near-Field)	6.6 to 7.0	2 to 15
Gridded Background and Other Crustal Faults (Far-Field)	6.0 to 7.4	15 to 62
Cascadia Subduction Zone Intraslab	6.9 to 7.0	54 to 124
Cascadia Subduction Zone Interface	8.4 to 9.1	74 to 158

TABLE A-6. RECOMMENDED INPUT GROUND MOTION TIME HISTORY DISTRIBUTION

EARTHQUAKE SOURCE TYPE	NUMBER OF RECORDS
Crustal Faults with Distance less than 15 km (Near-Field)	3
Gridded Background and Other Crustal Faults (Far-Field)	2
Cascadia Subduction Zone Intraslab	1
Cascadia Subduction Zone Interface	5

TABLE A-7. INPUT GROUND MOTION TIME HISTORIES

RECORD ID	RSN	EARTHQUAKE	SOURCE TYPE	MW	STATION	COMPONENT	DISTANCE (KM)
GM-1	184	Imperial Valley-06	Crustal	6.5	El Centro Differential Array	FN	5
GM-2	1402	ChiChi	Crustal	7.6	NST	FN	38
GM-3	1487	ChiChi	Crustal	7.6	TCU047	FN	35
GM-4	186	Imperial Valley-06	Crustal	6.5	Niland Fire Station	090	37
GM-5	736	Loma Prieta	Crustal	6.9	Apeel 9	227	41
GM-6	4000455	Tohoku	Interface	9.1	Kuzumaki	W	87
GM-7	4001145	Tohoku	Interface	9.1	HANNOH	EW	133
GM-8	4000839	Tohoku	Interface	9.1	Tatebayashi	NS	117
GM-9	6001811	2844986	Interface	8.8	MET	NS	122
GM-10	6002259	Coastal Chile	Interface	8.3	VA03	NN	117
GM-11	4032480	Hokkaido East	Intraslab	8.3	47420	NS	130

Input Ground Motion Time History Modification

We modified the seed ground motion time histories via spectral matching to match the target rock outcrop MCE UHS for spectral periods up to 12.2 seconds. We performed the spectral matching using RSPMatch09 (Fouad et al. 2012) based on the improved spectral matching approach proposed by Al Atik et al. (2010). We post processed the input ground motions with a Butterworth low pass filter to remove the frequencies greater than 10 Hertz (Hz). Figures A-1 and A-2 present the as-recorded and spectrally matched response spectra for the MCE input ground motion time history suite. Figures A-3 through A-13 present the associated acceleration, velocity, and displacement time histories.

SITE-SPECIFIC GROUND RESPONSE ANALYSIS

We completed the site-specific GRA using the explicit finite difference program, Fast Lagrangian Analysis of Continua (FLAC) Version 8.0 (Itasca 2016). FLAC models continuum materials, such as soil in one or two dimensions, and is capable of explicitly modeling stress-strain behavior during earthquake shaking.

One-Dimensional Soil Model

The soils encountered at the site generally consists of seven engineering soil units as presented in Section 2.2 in the main body of this report. We developed a best-estimate (BE) Vs profile as shown in Figure A-14. The upper 80 feet of Vs profile was developed based on the Vs measurements from the two existing seismic CPTs (CPT-1 and CPT-6). The Vs profile below 80 feet was developed based on the Vs measurements from two nearby sites: DOGAMI-13_64 per Bilderback et al. (2008) and WA-DNR-08_207 per Madin and Burns (2013) and the distributions of Vs with depth (average and ±1 standard deviation) for the CRB per Roe and Madin (2013). The developed BE Vs profile gradually increases with depth below 80 feet, which exhibits a similar trend with the two nearby Vs measurements. The Vs profile terminated at depth of 225 feet bgs with Vs=2,500 ft/sec.

We developed representative soil parameters of each engineering soil unit based on correlations with the existing standard penetration test (SPT) borings, CPTs, and the associated laboratory testing results. For the CPTs that have adjacent SPT borings (e.g., CPT-3 and CPT-6), we compared the correlation results and incorporated the comparison in the development of the soil parameters. More details of the developed representative soil parameters were presented in the following sections.

Total Stress Analysis

We used the GeoIndex MRD model proposed by Roblee and Chiou (2004) via the sigmoidal-3 model within FLAC to simulate the nonlinear dynamic behavior of fill and alluvium (ESU-1 through ESU-6); and used the Peninsular Range (PR) (Silva et al. 1997) curves for CRB (ESU-7) in TSA. Table A-8 presents the input soil parameters for the FLAC TSA model.

The GeoIndex MRD model is an adaptation of the depth- and soil-type dependent Darendeli (2001) model that incorporates more recent laboratory testing completed at larger confining stresses and strain levels, while aiming to accommodate the sensibilities of routine practice. This is accomplished by using three discrete soil groups based on plasticity index (PI) and grain size (percent fines, P200), rather than continuous functions of PI and confining stress, to account for the variation in properties with depth. The soil types are defined broadly so that specification can be applied to stratigraphic layers classified based on simple index tests or estimated base on visual classification or correlations. This feature is attractive for sites that have complex, interbedded stratigraphic profiles such as this one. As shown in Table A-8, we used the 1-PCA (primarily coarse-grained soil with P200≤30% for all PI values) and the 2-FML (primarily coarse-grained soil with P200>30% and PI<15) MRD curves to characterize the nonlinear dynamic response of ESU-1 through ESU-6. We assigned the 1-PCA and 2-FML models based on laboratory tests results for fines contents, Atterberg Limits, and correlated fines contents from the CPTs.

The PR curves are a subset of the EPRI (1993) curves and reflect a more linear response than the EPRI curves observed during the 1994 Northridge, 1971 San Fernando, and 1987 Whittier Narrows earthquakes.

The base of the 1D soil model consists of an elastic half-space located at a depth where the rock outcrop condition is encountered. We discretized the soil model to be capable of transmitting frequencies up to 25 Hz. The Elastic constitutive model was applied to all ESUs in TSA.

TABLE A-8. FLAC TSA INPUT SOIL PARAMETERS

ESU	SOIL DESCRIPTION	DEPTH RANGE (FEET)	UNIT WEIGHT (PCF[1])	MRD MODEL
1	Medium Dense SP (Fill)	0-7	120	1- PCA
2	Loose to Medium Dense SP/SP-SM/SM and Very Soft to Medium Stiff ML (Fill/Alluvium)	7-22	115	2-FML
3	Very Soft to Soft ML/CL (Alluvium)	22-30	110	
4	Medium Stiff to Stiff ML and Loose SP-SM/SM (Alluvium)	30-41	115	
5	Very Soft to Medium Stiff ML (Alluvium)	41-79	115	
6	Medium Stiff to Hard Silt (Alluvium)	79-85	120	
7	Columbia River Basalt (CRB)	85-221	130	PR (50-1000')

Notes:

[1] pcf – pound per cubic foot.

Effective Stress Analysis

The main purpose of performing ESA is to explicitly evaluate the liquefaction potential of the subsurface soils at the site. To accomplish this, we used the soil constitutive models that are capable of modeling stress-strain responses and excess pore water pressure generation during dynamic loading. These models were applied to the ESUs (ESU-2 through ESU-6) determined to be potentially liquefiable based on semi-empirical liquefaction correlations (more details discussed in the following section). For the sand-like ESUs located below the design groundwater table, we used the PM4Sand (Boulanger and Ziotopoulou 2017) model. For the clay-like ESUs, we used the PM4Silt (Boulanger and Ziotopoulou 2018) model. PM4Sand and PM4Silt build on the framework of the stress-ratio controlled, critical state compatible, bounding surface plasticity model for sand. PM4Sand approximates the undrained cyclic and monotonic responses of sands and non-plastic silts, whereas PM4Silt is intended for low plasticity silts and clays. PM4Sand and PM4Silt incorporate nonlinear hysteric soil response directly; therefore, separate MRD models are not necessary.

We used the PM4Sand constitutive model for the potentially liquefiable sand-like ESUs (e.g., ESU-2 and ESU-4). PM4Sand is a bounding surface plasticity model that computes the volumetric response of the soil using a flow rule that is a function of the current stress ratio. Importantly, PM4Sand can simulate excess pore water pressure generation under dynamic loading conditions. The PM4Sand model was calibrated at the equation level to approximate the trends observed across a set of experimentally- and case history-based liquefaction correlations.

The primary input parameters for the PM4Sand model, in addition to the typical soil parameters derived previously (e.g., unit weight, Poisson's ratio, and hydraulic conductivity), are the relative density (D_R), shear modulus coefficient (G_o), and the contraction rate parameter (h_{p0}). The relative density is best considered an "apparent relative density," rather than a strict measure of relative density from laboratory testing.

The input value of D_R influences model response like any other input parameter and can be adjusted as part of the calibration process. The shear modulus coefficient, G_o, controls the elastic shear modulus as follows:

$$G = G_o p_A (p/p_A)^{0.5}$$

where G_o was derived from the idealized Vs profile normalized to 1 atmosphere (V_{s1}). The contraction rate parameter, h_{p0}, adjusts the contraction rate and therefore the cyclic resistance ratio (CRR).

We used the PM4Silt constitutive model to model the dynamic behavior of potentially liquefiable clay-like ESUs (ESU-3, ESU-5, and ESU-6). The PM4Silt model builds on the framework of PM4Sand to approximate undrained monotonic and cyclic loading responses of low-plasticity silts and clays. The primary input parameters for the PM4Silt model include undrained shear strength S_u (or undrained strength ratio S_u/σ'_v), which can be estimated by in situ testing, laboratory testing of "undisturbed" field samples, and/or empirical correlations, G_0 and h_{po}.

We calibrated the PM4Sand and PM4Silt models following the general procedures provided by Boulanger and Ziotopoulou (2017) and Boulanger and Ziotopoulou (2018), respectively. We calibrated h_{po} by matching the CRR values from direct simple shear simulation with the $CRR_{M=7.5}$ values computed from the SPT-based liquefaction triggering correlation. We assume for PM4Sand that $CRR_{M=7.5}$ is approximately equal to the CRR corresponding to 15 uniform loading cycles causing a maximum excess pore water pressure ratio (R_u) of 0.98. We used a similar approach for the PM4Silt calibration, where $CRR_{M=7.5}$ is approximately equal to the CRR corresponding to 30 uniform loading cycles causing a peak shear strain of 3% in direct simple shear loading. The key model parameters used in the ESA are presented in Table A-9.

For the ESUs determined to be non-liquefiable (e.g., located above the design groundwater table and ESU-7), we used the conventional Morh-Coulomb (MC) constitutive model and included the MRD models from the TSA to approximate nonlinear behavior during dynamic loading. The inputs for the MC model are the same as the Elastic model used in the TSA, except the MC model additionally includes soil strength (friction angle), as presented in Table A-9.

TABLE A-9. FLAC ESA INPUT SOIL PARAMETERS

ESU	Depth Range (feet)	Constitutive Model	Unit Weight (pcf)	Friction Angle (degrees)	S_u/σ'_v	V_{s1} (ft/sec)	Poisson's Ratio	h_{p0}	Hydraulic Conductivity (ft/sec)	MRD Model
1	0-7	MC	120	35	-	-	0.3	-	3.34E-05	1- PCA
2	7-13	MC	115	33	-	-	0.3	-	3.18E-07	2-FML
	13-16	PM4Sand		-	-	653	0.3	0.47		-
	16-22	PM4Sand		-	-	538	0.3	0.53		-
3	22-30	PM4Silt	110	-	0.24	-	0.3	9.2	2.94E-09	-
4	30-41	PM4Sand	115	-	-	639	0.3	0.48	5.94E-08	-
5	41-65	PM4Silt	115	-	0.23	-	0.3	9.0	3.22E-09	-
	65-79			-		-	0.3	8.6		-
6	79-85	PM4Silt	120	-	0.42	-	0.3	72	3.77E-08	-
7	85-221	Elastic	130	-	-	-	-	-	1.00E-05	PR (50-1000')

Dynamic Loading Conditions

The development of the input ground motion time histories for use in the site-specific GRA is presented in the above sections. FLAC uses an input stress time history to prevent absorption by the compliant base dashpots. We converted the acceleration time histories to equivalent shear stress time histories using the following equation:

$$\sigma_s = -2\rho V_s v_s$$

where σ_s is the shear stress, ρ is the soil mass density, and v_s is the shear component of particle velocity at the boundary (i.e., the input velocity time history).

Site-Specific Soil Amplification Factors

Figures A-15 and A-16 present the individual and geomean surface response spectra computed from TSA and ESA, respectively. Figures A-17 and A-18 present the corresponding soil AFs, which we computed as the geomean of the ratio of the ground surface response spectra to the input rock outcrop MCE UHS. Figure A-19 presents the comparison of the soil AFs from TSA and ESA. The soil AFs from ESA are generally lower than the soil AFs from TSA at the periods less than 5.7 seconds but become relatively higher at longer periods. The recommended soil AFs that were taken as the envelop of the soil AFs from TSA and ESA.

MAXIMUM COMPONENT ADJUSTMENT FACTORS AND RISK COEFFICIENTS

Per ASCE 7-16, the MCE$_R$ ground motions are to be taken in the direction of maximum horizontal response. MCA factors convert the geometric mean spectral ordinates to spectral ordinates that correspond to the direction of maximum horizontal response. We used the MCA factors provided by Shahi and Baker (2014).

Risk coefficients convert the probabilistic MCE ground motions (i.e., 2 percent probability of exceedance in 50 years) to MCE$_R$ ground motions, which correspond to a 1 percent probability of collapse in 50 years. Risk coefficients were calculated per ASCE 7-16 Section 21.2.1.2. We computed the risk coefficients based on the Site Class D seismic hazard curves. Table A-10 below presents the MCA factors and the site-specific risk coefficients.

TABLE A-10. MAXIMUM COMPONENT ADJUSTMENT FACTORS AND RISK COEFFICIENTS

PERIOD (SEC)	MAXIMUM COMPONENT ADJUSTMENT FACTOR	RISK COEFFICIENT
0.01	1.19[1]	0.88
0.05	1.19	0.88
0.075	1.19	0.88
0.1	1.19	0.88
0.2	1.21	0.88
0.3	1.22	0.88
0.4	1.23	0.88
0.5	1.23	0.88
0.75	1.24	0.88
1	1.24	0.87
2	1.24	0.87
3	1.25	0.86
4	1.26	0.86
5	1.26	0.88
7.5	1.28	0.90
10	1.29	0.93

SITE-SPECIFIC MCE$_R$ RESPONSE SPECTRUM

We computed the site-specific probabilistic MCE$_R$ horizontal response spectrum per ASCE 7-16 Section 21.2.1 by scaling the rock outcrop MCE UHS by the recommended site-specific soil AFs, MCA factors, and risk coefficients.

Figure A-20 presents the site-specific probabilistic MCE$_R$ response spectrum. Based on the exception under Section 21.2.2 in the ASCE 7-16 Supplement 1, the deterministic ground motion response spectrum need not be calculated when the largest spectral response acceleration of the probabilistic ground motion response spectrum developed per ASCE 7-16 Section 21.2.1 is less than 1.2F$_a$. This F$_a$ was determined as 1.0 for the Site Class D with a value of S$_s$ taken as 1.5 per the ASCE 7-16 Supplement 1 Section 21.2.2. Per Figure A-20, the largest spectral response acceleration of the site-specific probabilistic MCE$_R$ response spectrum is 1.16g, which is less than 1.2F$_a$. Therefore, the deterministic ground motion response spectrum was not calculated for comparison with the site-specific probabilistic MCE$_R$ response spectrum.

The fundamental period of the 1D soil model used in the site-specific GRA is approximately 0.6 seconds. Considering that 1D GRA may not adequately capture the amplification behavior at the periods greater than 1.5 to 2 times the fundamental period of the site model (T$_{col}$), we compared the spectral accelerations computed from the Vs30-based empirical approach with the spectral accelerations from our site-specific GRA at the periods greater than 1.5T$_{col}$=0.9 seconds. We computed a Vs30-based MCE response spectrum using the NSHM Hazard Tool with the Conterminous U.S. 2023 (Beta Version) based on Vs30=763 feet/sec calculated from the BE Vs profile. The Vs30-based MCE response spectrum was scaled by applying the MCA factors and risk coefficients to compute the Vs30-based MCE_R response spectrum, as shown in Figure A-20. The Vs30-based spectral accelerations are lower than the site-specific GRA spectral accelerations at the periods less than 1.1 seconds and become higher at longer periods. Therefore, we enveloped the Vs30-based spectral accelerations and the site-specific GRA spectral accelerations at the periods greater than 0.9 seconds. The recommended site-specific MCE_R response spectrum is the greater of enveloped MCE_R spectral accelerations and the ASCE 7-16 Section 21.3 Site Class D minimum. As shown in Figure A-20, the recommended site-specific MCE_R response spectrum is controlled by the ASCE 7-16 Section 21.3 Site Class D minimum at the periods greater than 6 seconds. Table A-11 presents the recommended site-specific MCE_R response spectrum.

TABLE A-11. RECOMMENDED SITE-SPECIFIC MCE_R RESPONSE SPECTRUM

PERIOD (SEC)	5% DAMPED SPECTRAL ACCELERATION (G)
0.01	0.57
0.05	1.02
0.075	1.07
0.1	1.02
0.2	1.00
0.3	1.14
0.4	1.06
0.5	1.06
0.75	1.12
0.8	1.16
0.9	1.15
1	1.00
2	0.50
3	0.30
4	0.21
5	0.15
6	0.12
7	0.10
8	0.09
9	0.08
10	0.07
11	0.06
12	0.06
12.2	0.06

Site-Specific Design Acceleration Parameters

Per ASCE 7-16 Section 21.4, the site-specific S_{DS} is taken as 90 percent of the maximum design Sa between T=0.2 to 5.0 sec in Table A-11. The site-specific S_{D1} is taken as the maximum value of the product, T×Sa, in Table A-11 between T=1.0 to 5.0 sec for Vs30≤1,200 ft/sec. Table A-12 presents the site-specific S_{DS} and S_{D1} values computed per ASCE 7-16 Section 21.4.

TABLE A-12. SITE-SPECIFIC DESIGN ACCELERATION PARAMETERS

PARAMETER	VALUE
S_{DS} (g)	0.70
S_{D1} (g)	0.67

Recommended Site-Specific PGA_M

We computed the geometric mean MCE peak ground acceleration (PGA_M) by scaling the rock outcrop PGA by the recommended site-specific PGA soil AF. Table A-13 presents the site-specific and code-based PGA values. Per ASCE 7-16 Section 21.5.3, the site-specific PGA shall not be less than 80 percent of the code based PGA_M. The site-specific PGA_M is higher than 80 percent of the Site Class D PGA_M; therefore, the site-specific PGA_M controls.

TABLE A-13. RECOMMENDED SITE-SPECIFIC PGA_M

PARAMETER	VALUE
Mapped MCE_G peak ground acceleration, PGA (g)	0.403
PGA Site Coefficient, F_{PGA} (Site Class D)	1.197
ASCE 7-16 PGA_M (g)	0.482
80% ASCE 7-16 PGA_M (g) [code minimum]	0.386
Site-specific Rock Outcrop PGA (g)	0.509
Site-Specific PGA Soil AF	1.07
Recommended Site-specific PGA_M (g)	0.55

LIQUEFACTION EVALUATION

Prior to the ESA, we performed simplified liquefaction analysis to evaluate the liquefaction potential of the site soils under an MCE event using semi-empirical methods (Youd, et al. 2001 and Idriss and Boulanger 2008) based on the existing SPT borings and CPTs. The simplified liquefaction analysis was performed with a Mw 9.0 to conservatively capture the CSZ interface event and a site-specific PGA_M of 0.55g as presented in Table A-13. Based on the simplified liquefaction analysis, ESU-2 through ESU-6 are susceptible to liquefaction. Therefore, in the ESA, we used the soil constitutive models that are capable of modeling stress-strain responses and excess pore water pressure generation during dynamic loading (e.g., PM4Sand and PM4Silt) to simulate these potentially liquefiable soils.

We evaluated liquefaction triggering by considering a stress-based definition of liquefaction based on the excess pore pressure ratio (Ru) computed for each soil zone during dynamic loading. As stated in Section 3.1.2 in the main body of this report, we selected Ru equal to 0.7 as our criterion to evaluate the liquefaction potential per Boulanger et al. (1998) and Oregon Department of transportation (ODOT) Geotechnical Design Manual (GDM) 2023. The Ru=0.7 is equivalent to a factor of safety against liquefaction (FS_{liq}) of 1.1. Per ODOT GDM 2023, liquefaction is conservatively predicted to occur when the FS_{liq} is less than 1.1; and a FS_{liq} of 1.1 or less also indicates the potential for liquefaction-induced ground movement.

Figure A-21 presents the individual and geomean maximum Ru versus depth obtained from the ESA. From Figure A-21, the maximum Ru is generally lower than 0.7 below the ESU-4 at depths greater than 40 feet bgs approximately. Some occasional local peaks reaching approximately 0.72 to 0.84 were observed at a depth of 58 feet and between 75 to 80 feet bgs, approximately. These local peaks only occurred from GM-5 and GM-10. Given only two out of the eleven ground motion time histories with these occasional local peaks, we concluded the likelihood of liquefaction below the ESU-4 is low. Therefore, liquefaction depth was considered along the bottom of the ESU-4. Refined constitutive soil models may provide greater insight into potentially liquefiable soil layers.

A.5 Earthquake-Induced Lateral Ground Deformation

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, 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-14. Under the post-earthquake condition, residual strengths were used in the liquefiable soils; 80 percent of static strengths were used in the soils above groundwater table; and full static strengths were used in the soils (non-liquefiable) below liquefaction depth. Under the seismic condition, we considered two scenarios assuming liquefaction occurs during or after earthquakes. For the case that we assumed liquefaction occurs at the end of earthquakes, 80 percent of static strengths were used in the soils above liquefaction depth. For the case that we assumed liquefaction occurs during earthquakes, we used residual strengths in the liquefiable soils. For the silty soil layers (e.g., ESU-5 and ESU-6), undrained shear strength was used under seismic and post-earthquake conditions. It is important to note the soil strength parameters used for pre and post liquefaction conditions will likely change with additional subsurface exploration and more direct laboratory testing of the soils’ dynamic properties.

TABLE A-14. SOIL PROPERTIES IN SLOPE/W ANALYSIS

ESU	UNIT WEIGTH (PCF³)	FRICTION ANGLE (DEG²)	COHESION (PSF²)	UNDRAINED SHEAR STRENGTH (PSF²)	RESIDUAL FRICTION ANGLE¹ (DEG²)
1	120	35	-	-	-
2	115	33	-	-	4
3	110	28	-	-	4
4	115	32	-	-	6
5	115	28	-	900	-
6	120	34	-	2,000	-
7	130	40	400	-	-

Notes:

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

[2] deg = degree; psf = pounds per square foot

Figure A-22 presents the slope stability analysis results for the post-earthquake condition. Under the post-seismic condition, within the area that is approximately 400 feet behind the riverbank, the factor of safety (FOS) was estimated to be less than 1.1. It indicated a flow failure within approximately 400 feet behind riverbank. This is likely a conservative assessment as the simplified model does not incorporate strain hardening that often occurs as excess porewater pressures redistribute.

Figures A-23 and A-24 present the slope stability analysis results for the two seismic conditions. The seismic conditions were evaluated for the areas that were not anticipated to experience flow failure (areas greater than 400 feet away from the riverbank). Given relatively consistent subsurface conditions across the site, the earthquake-induced lateral ground deformation generally decreases with the distance from free face. Therefore, we performed the slope stability analysis at a distance of approximately 900 feet away from the riverbank which covered the majority of the terminal site. In the case that assumed liquefaction occurs at the end of earthquakes, the minimum yield acceleration was estimated as 0.145g. The yield acceleration is the horizontal seismic coefficient that results in a FOS of 1.0 computed from the slope stability analysis under seismic condition. The corresponding earthquake-induced lateral ground deformation was estimated to be on the order of 24 inches using the simplified displacement approach developed by Bray and Travasarou (2007) and Bray et al. (2018). A Mw 9.0 was used in the simplified displacement approach to conservatively capture the influence from the CSZ interface earthquake events. The recommended site-specific MCE_R response spectrum (Table A-11) was used as the ground shaking intensities. In the case that assumed liquefaction occurs and strong shaking is still occurring, the minimum yield acceleration was estimated as 0.110g. The corresponding earthquake-induced lateral ground deformation was estimated on the order of 36 inches. Again, it is important to note these are likely conservative assessments because the simplified slope stability model does not incorporate the strain hardening that often occurs as excess porewater pressures redistribute. In addition, the Newmark-type analyses assume the soil behaves as a rigid block. In actuality the soils will deform considerably during shaking, dampening energy from the shaking and thus reducing the overall permanent lateral displacement.

Based on the simplified analyses described above, we developed a map that divides the terminal site into two groups (as shown in Figure 7 in the main body of the report), that include:

- Within approximately 400 feet of the riverbank (the Near Shore Zone), that was anticipated to experience flow failure based on the simplified analyses.
- For the areas more than 400 feet beyond the riverbank (the Inland Zone), “Inland Zone” where flow failure was not predicted but large seismically induced ground deformations are still possible based on the simplified analyses.

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.

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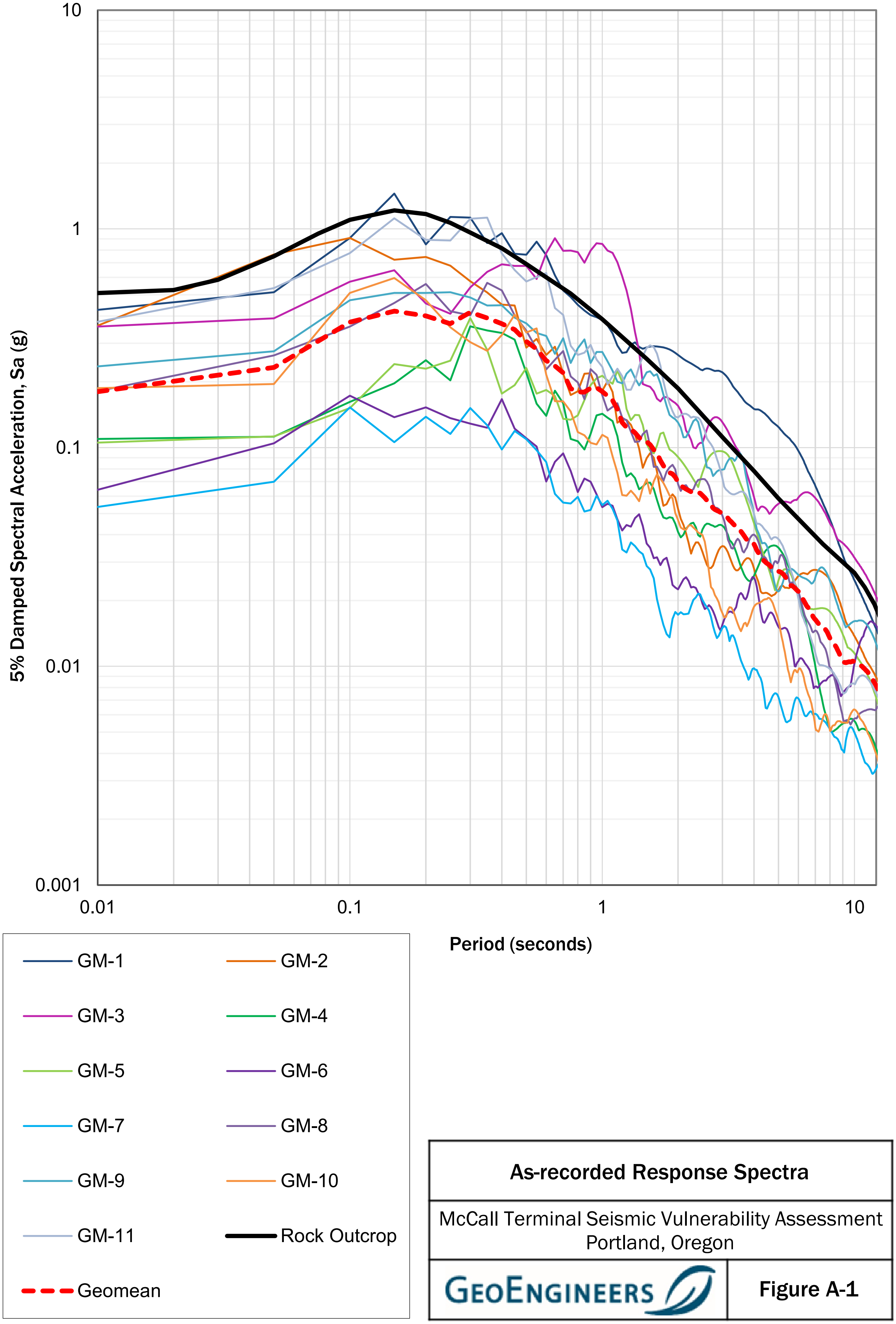
Figures

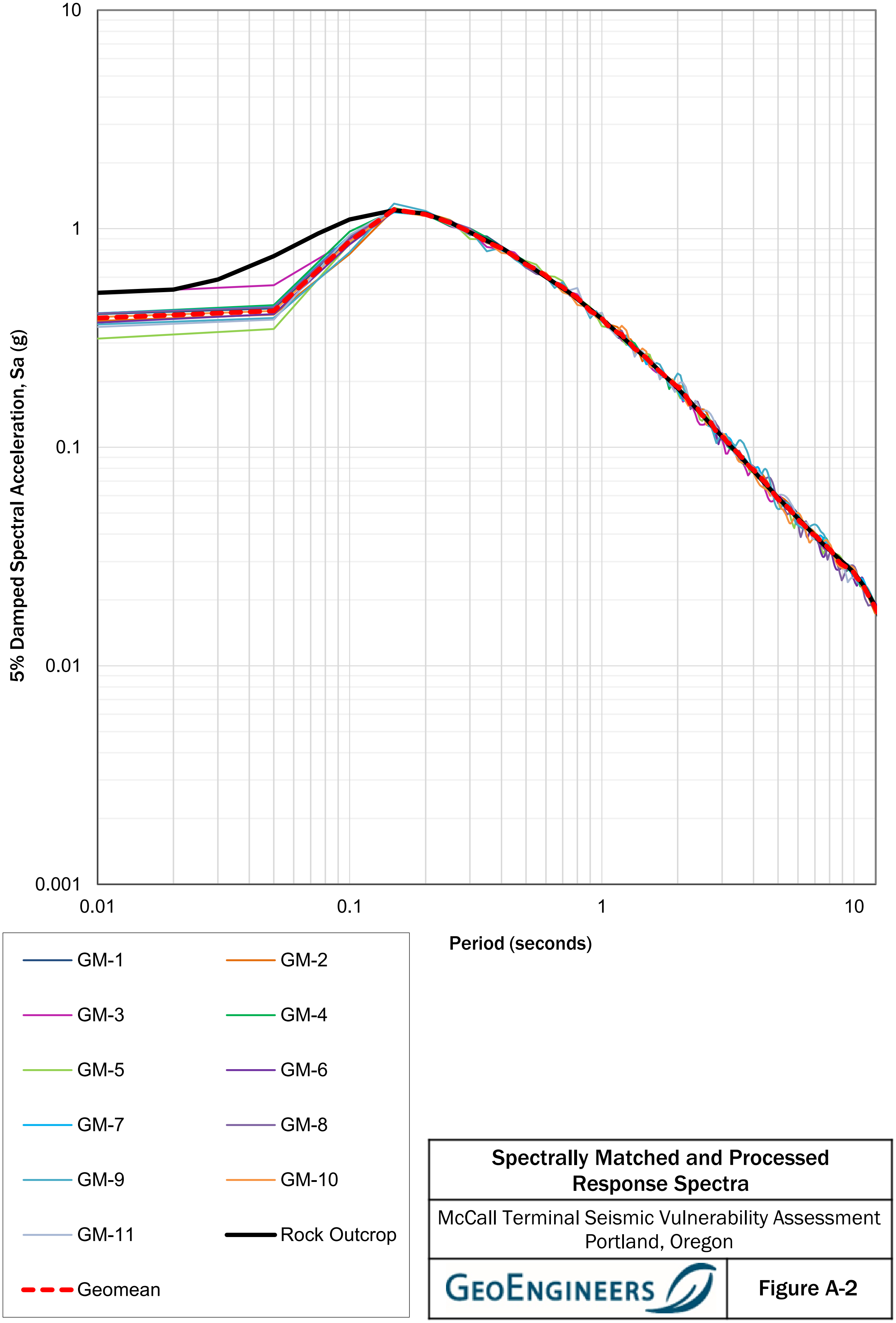


As-recorded Response Spectra

McCall Terminal Seismic Vulnerability Assessment
Portland, Oregon

GeoEngineers

Figure A-1



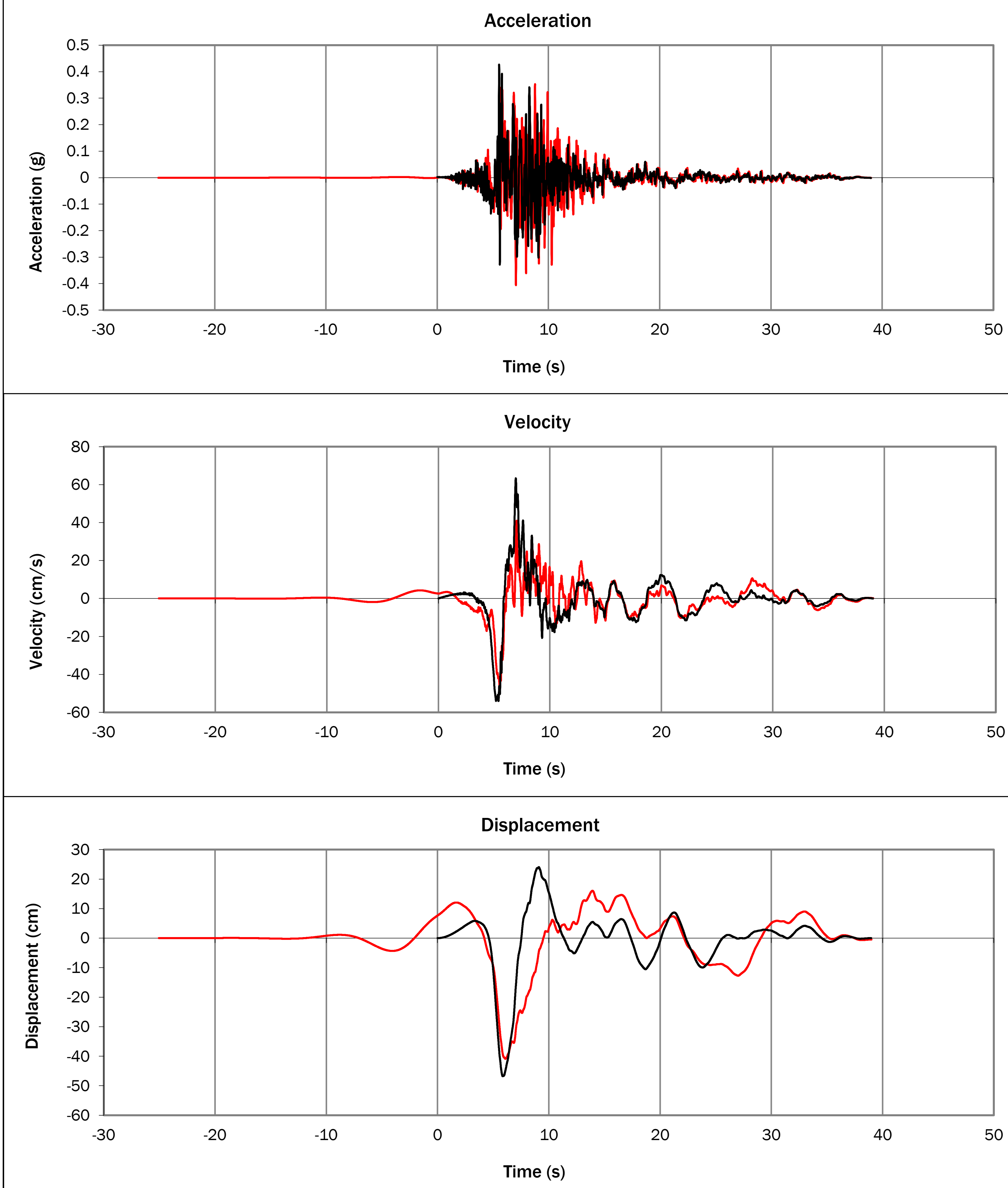
26662-001-00 Date:05/03/2024

Spectrally Matched and Processed Response Spectra

McCall Terminal Seismic Vulnerability Assessment
Portland, Oregon

GeoEngineers

Figure A-2



Legend

── As-recorded

── Spectral Matched

Acceleration, Velocity & Displacement Time Series
GM-1 - Imperial Valley 1979 - El Centro Differential Array (FN)

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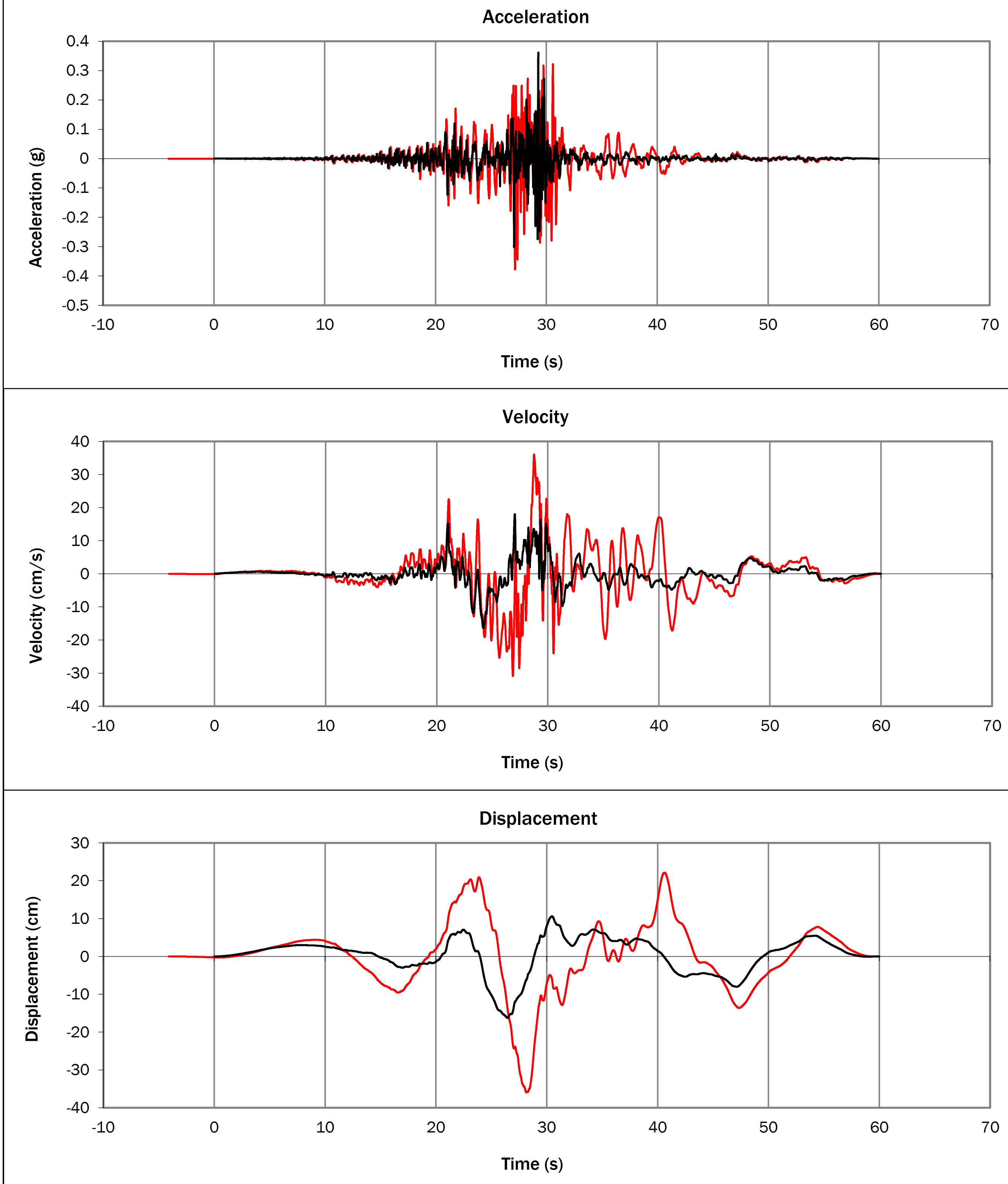
Figure A-3



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— As-recorded

— Spectral Matched

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GM-2 - Chi-Chi 1999 - NST (FN)

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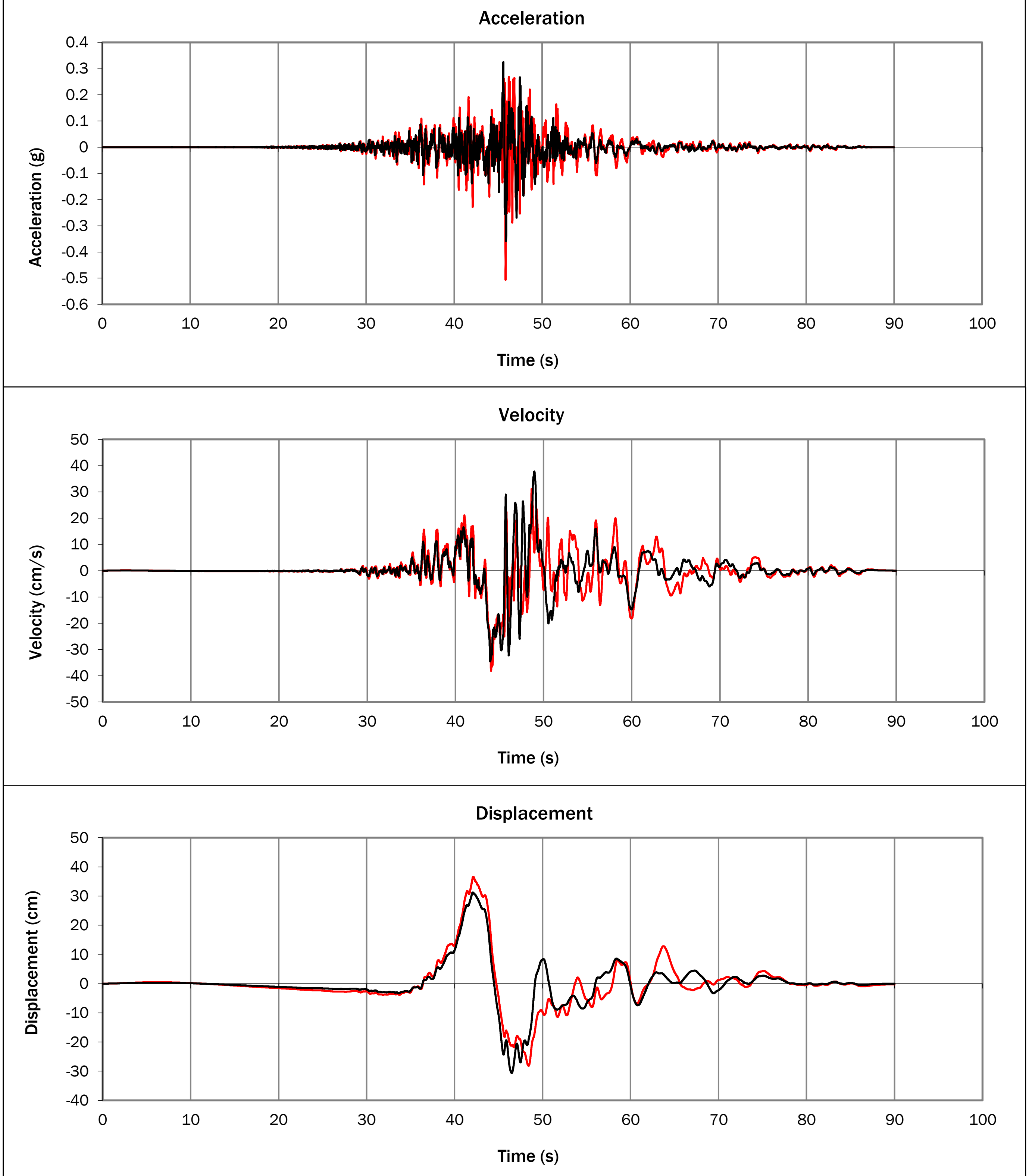
Figure A-4



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——— As-recorded

——— Spectral Matched

Acceleration, Velocity & Displacement Time Series
GM-3 - Chi-Chi 1999 - TCU047 (FN)

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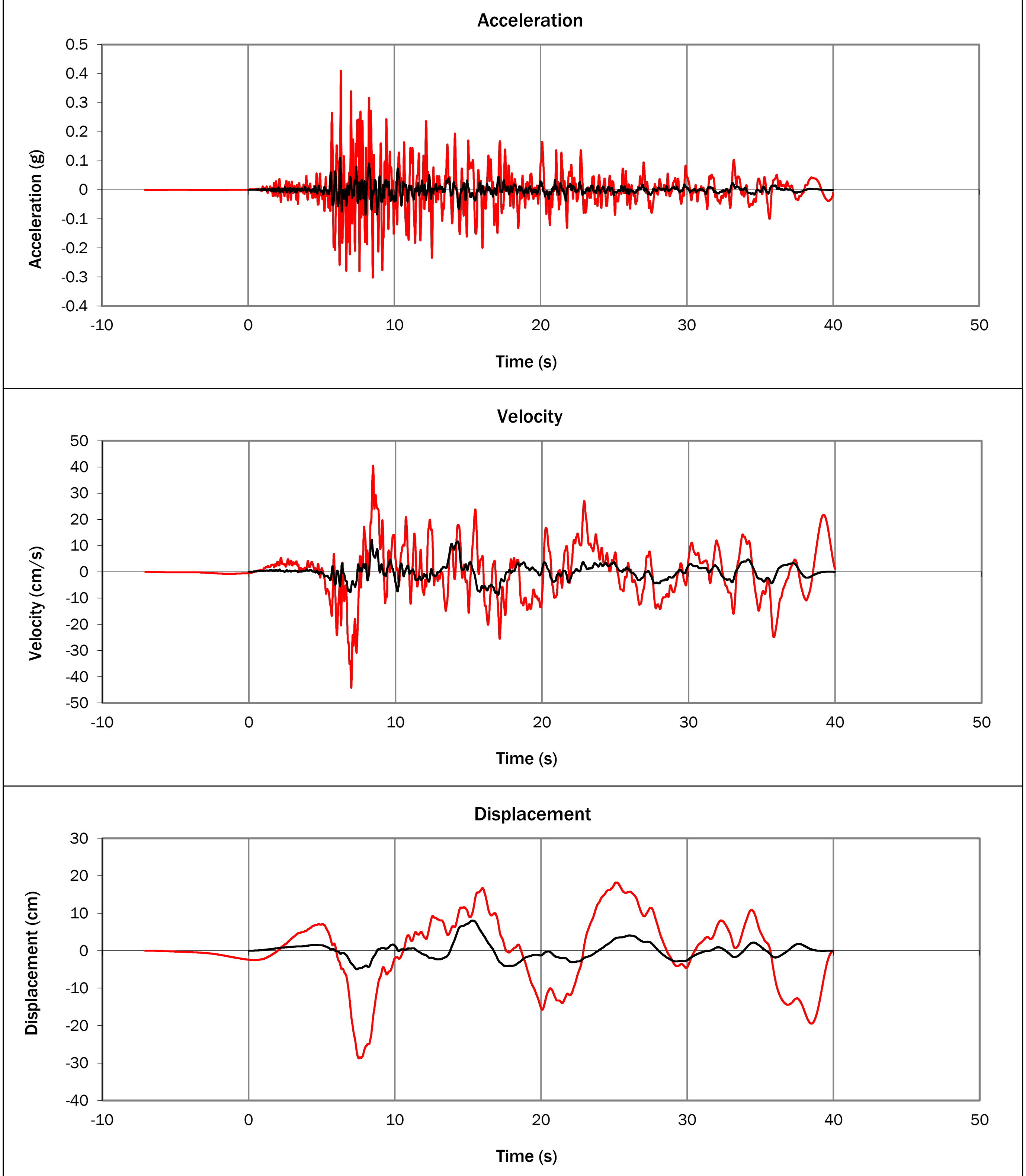
Figure A-5



Legend

——— As-recorded

——— Scaled

Acceleration, Velocity & Displacement Time Series
GM-4 - Imperial Valley 1979 - Niland Fire Station (090)

McCall Terminal Seismic Vulnerability Assessment
Portland, Oregon

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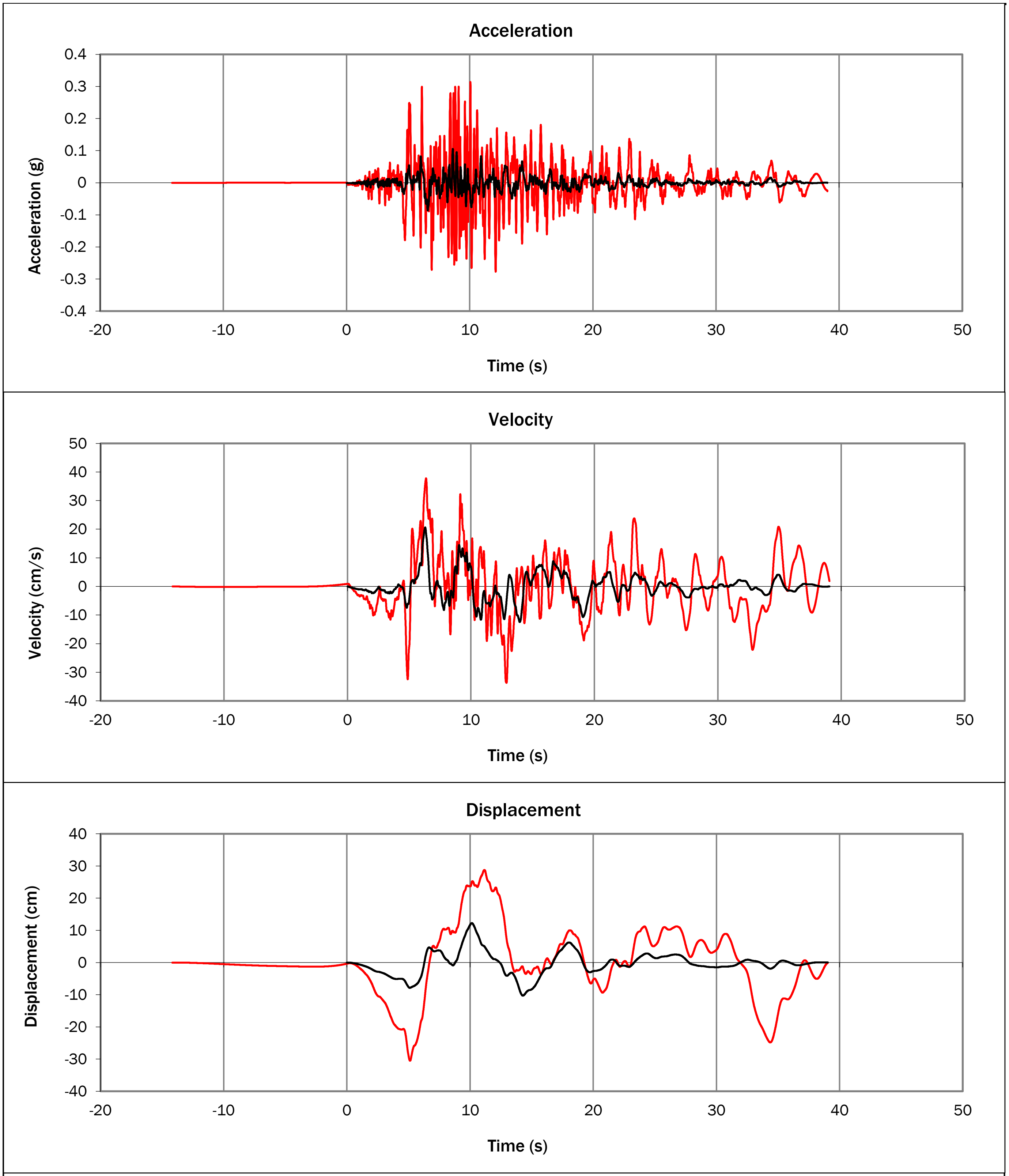
Figure A-6



Legend

— As-recorded

— Scaled

Acceleration, Velocity & Displacement Time Series
GM-5 - Loma Prieta 1989 - Apeel 9 (227)

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Portland, Oregon

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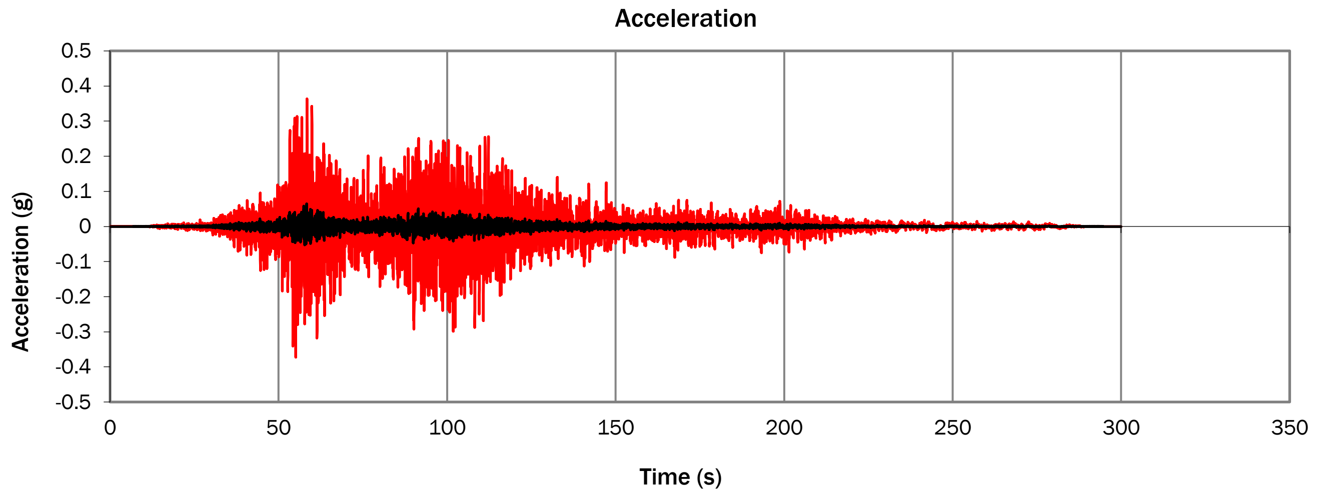
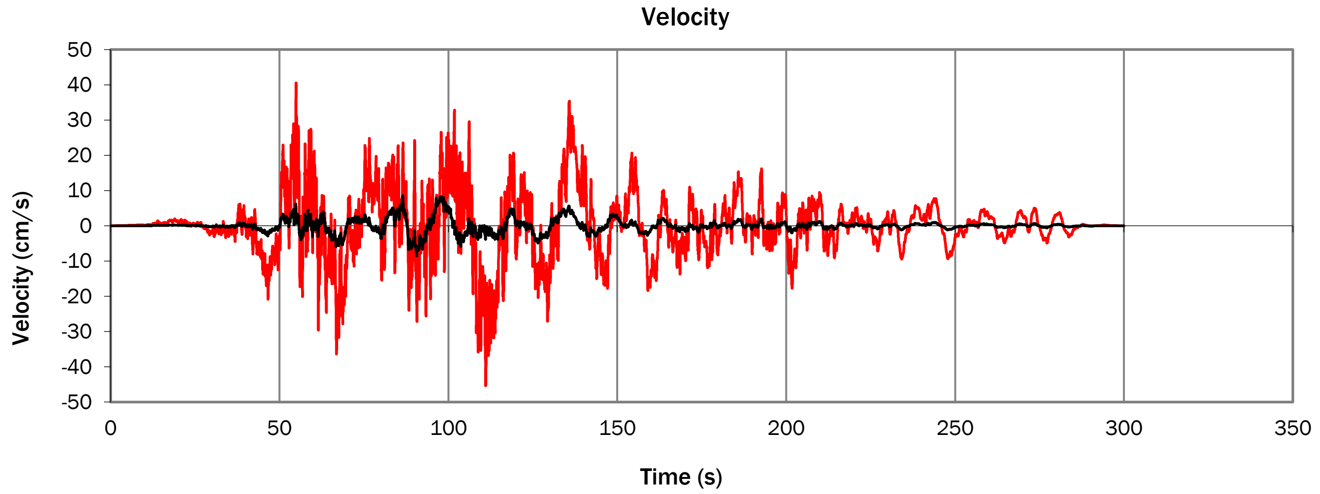
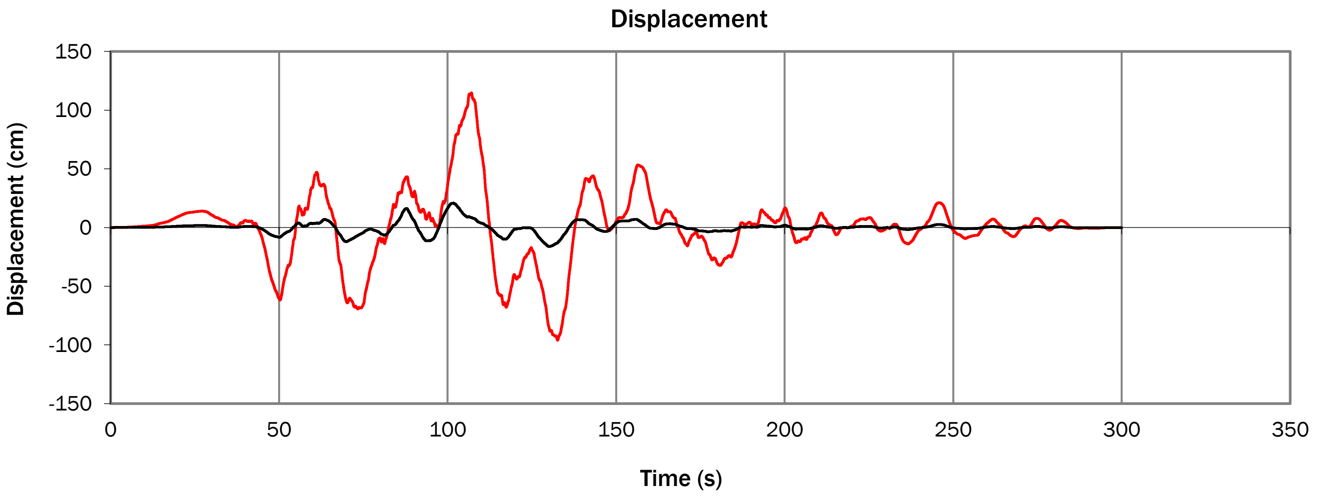
Figure A-7







Legend

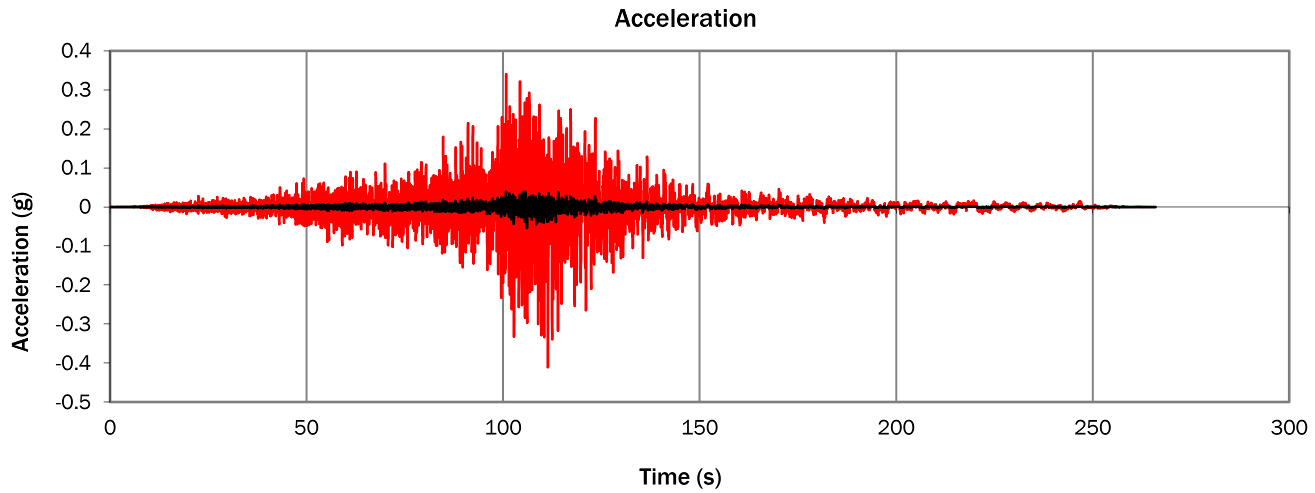
—— As-recorded
—— Scaled

Acceleration, Velocity & Displacement Time Series
GM-6 - Tohoku 2011 - Kuzumaki (W)

McCall Terminal Seismic Vulnerability Assessment
Portland, Oregon

Figure A-8

Acceleration



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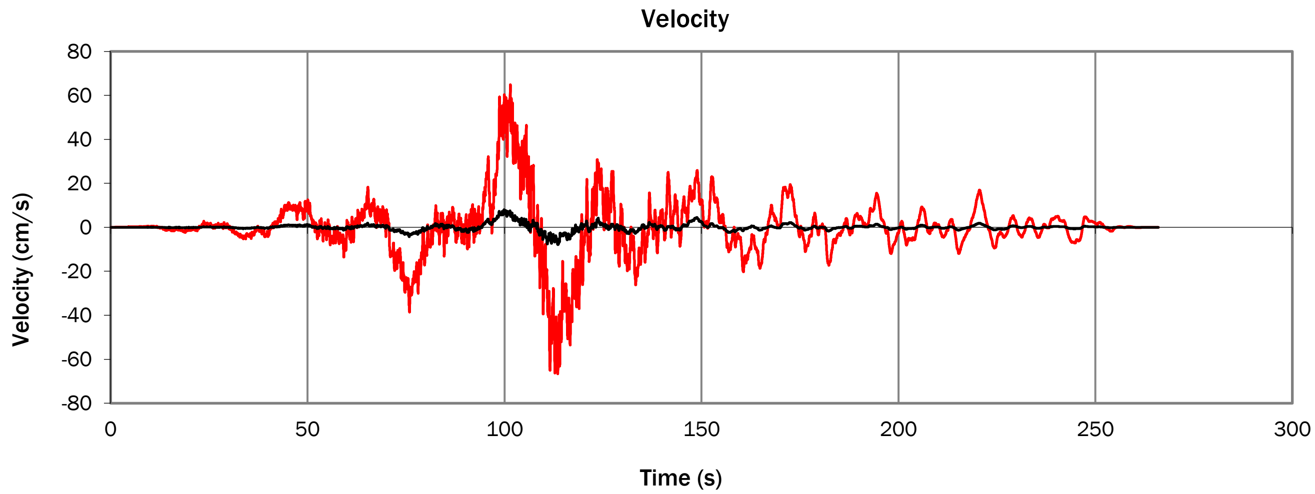
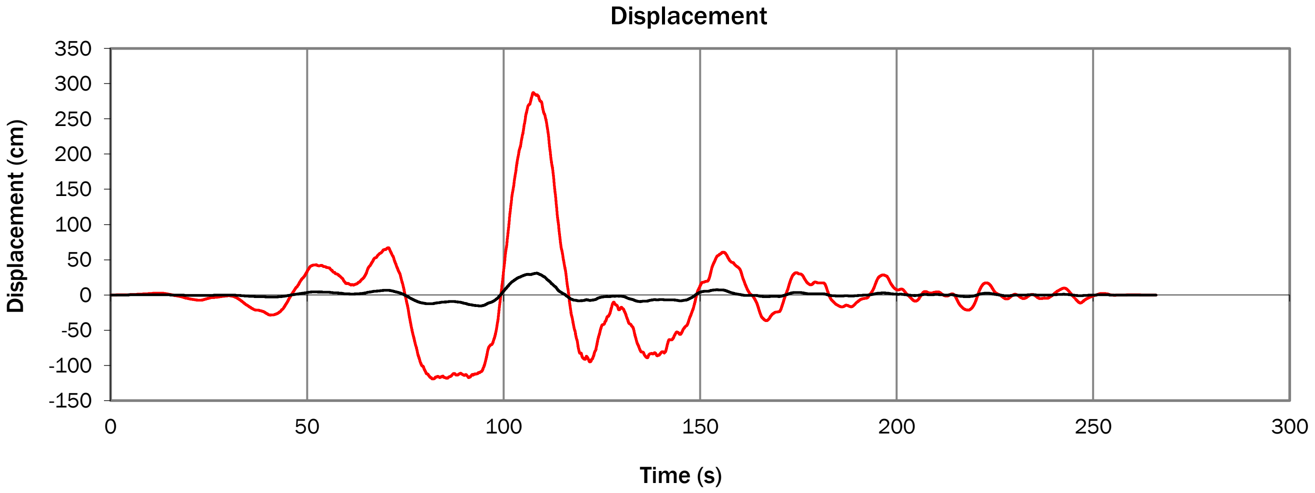


Displacement



Legend

— As-recorded
— Scaled

Acceleration, Velocity & Displacement Time Series
GM-7 - Tohoku 2011 - HANNOH (EW)

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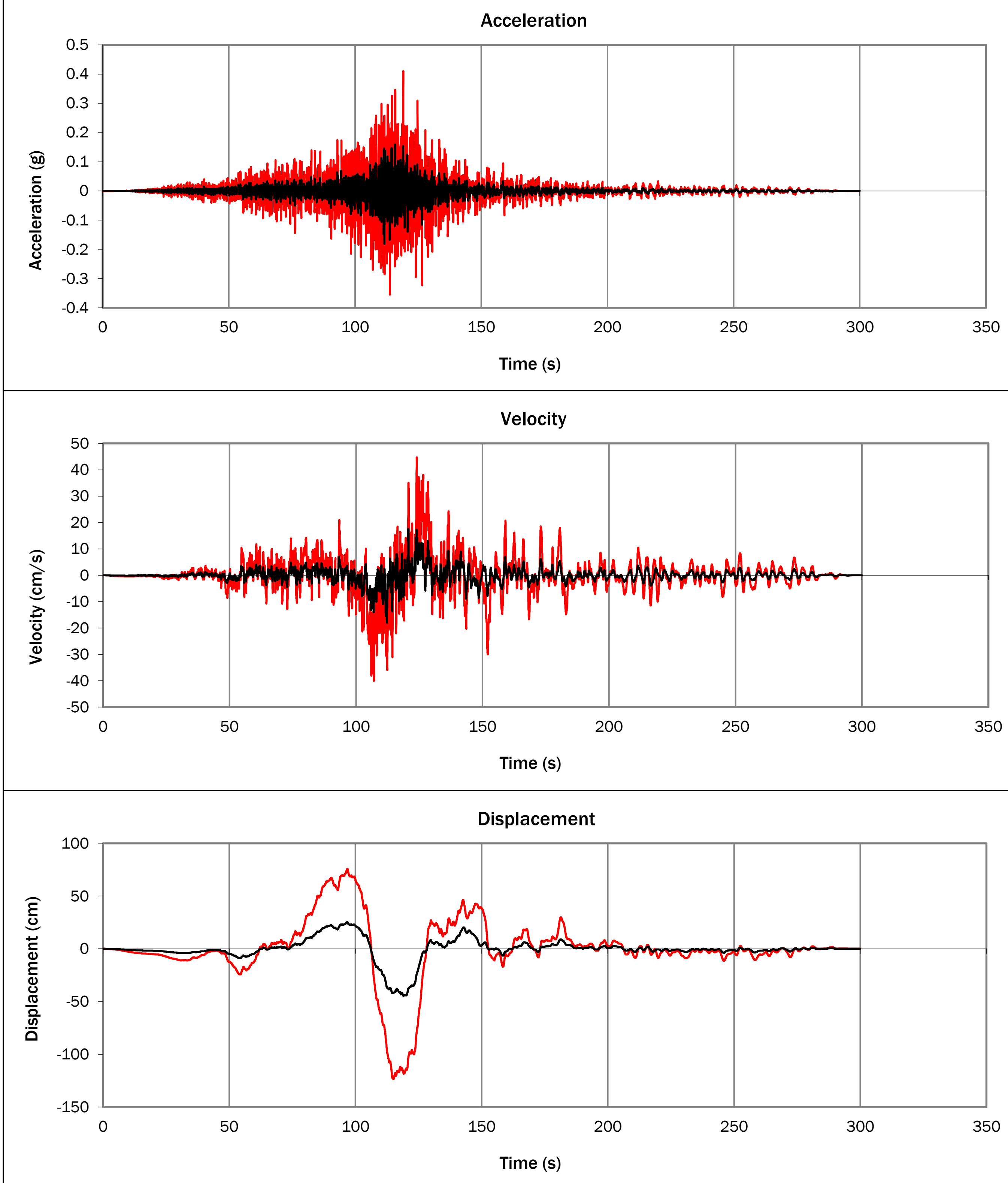
Figure A-9



Legend

— As-recorded

— Scaled

Acceleration, Velocity & Displacement Time Series
GM-8 - Tohoku 2011 - Tatebayashi (NS)

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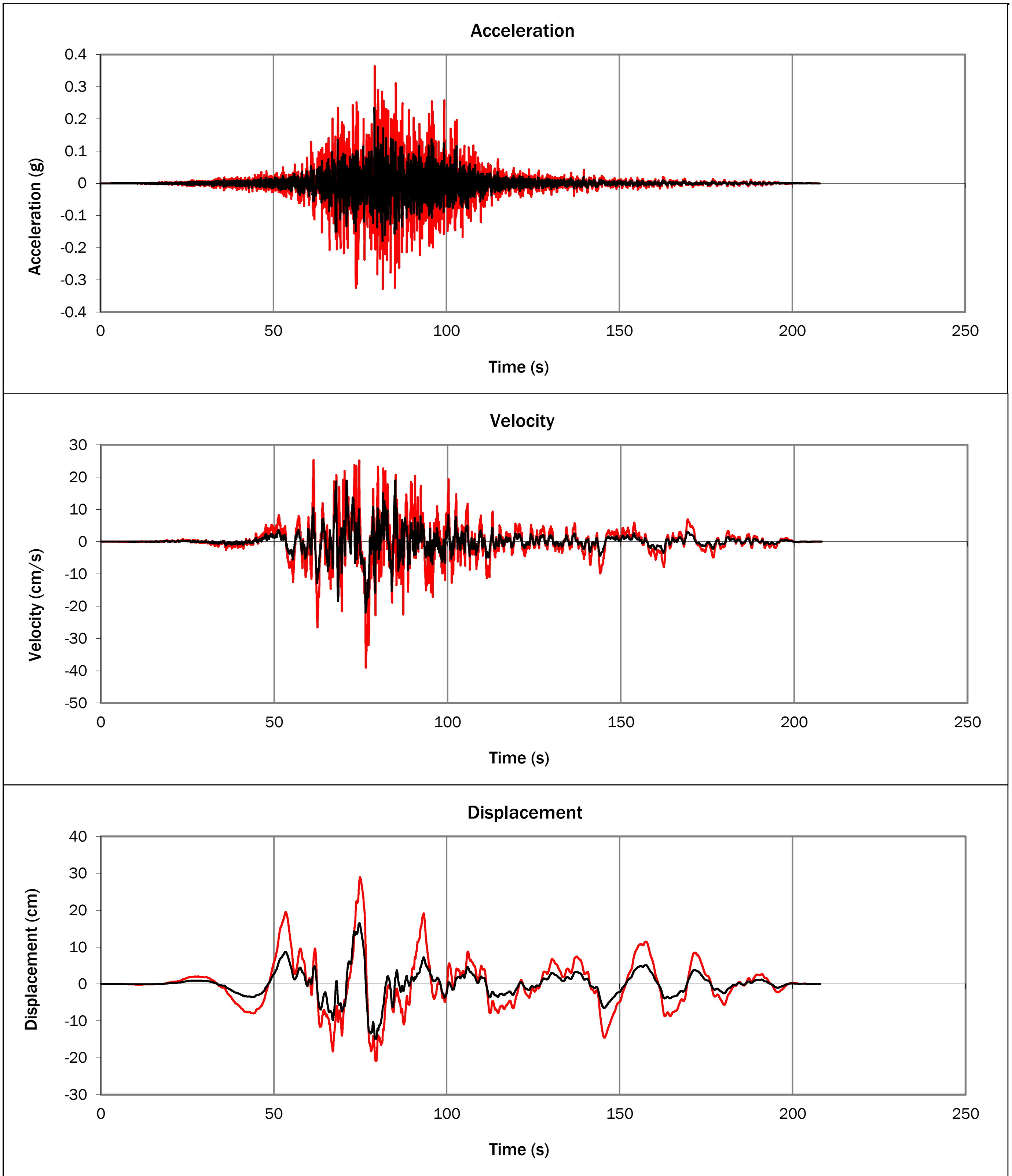
Figure A-10



Legend

— As-recorded

— Scaled

Acceleration, Velocity & Displacement Time Series
GM-9 - 2844986 2010 - MET (NS)

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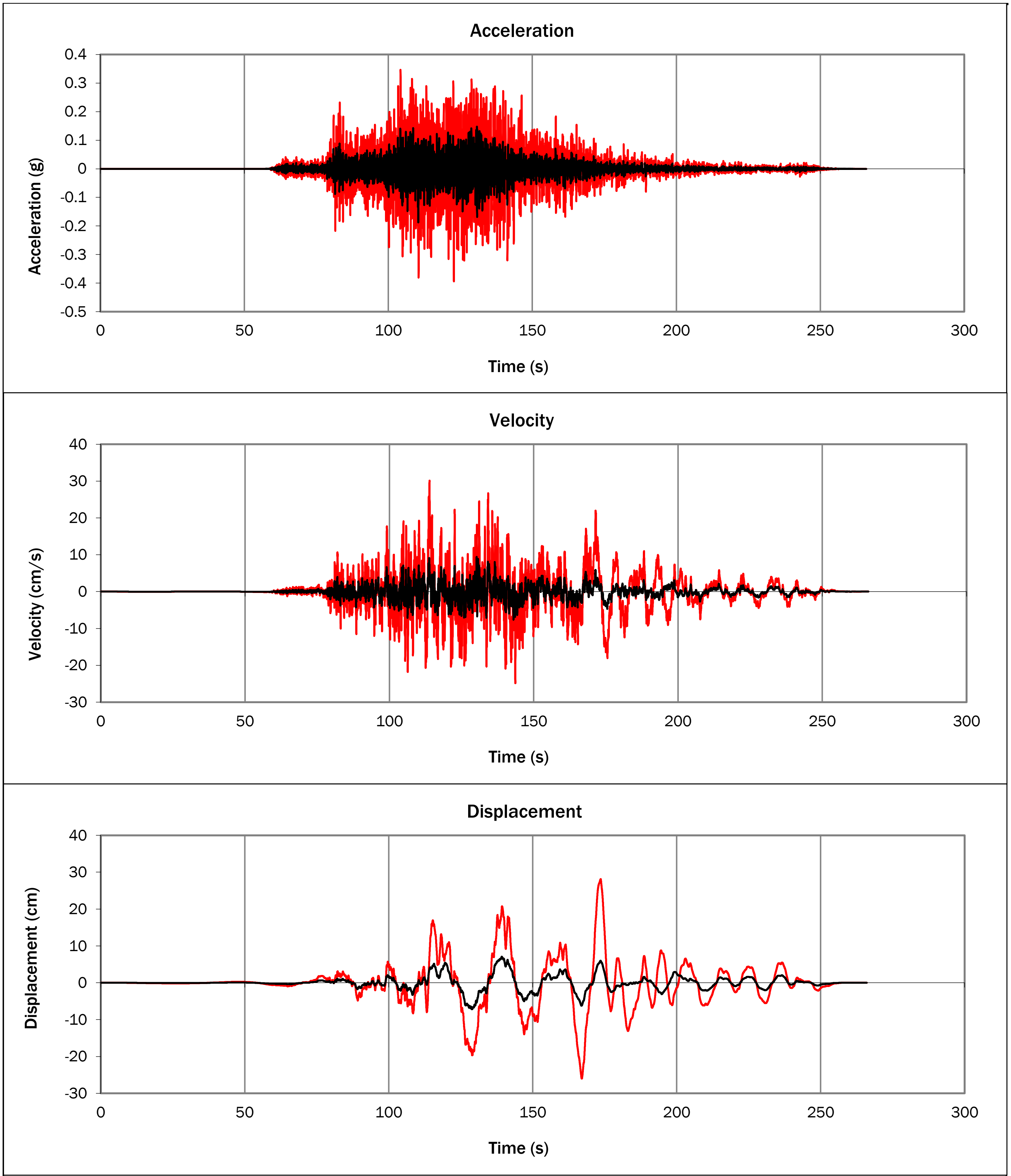
Figure A-11



Legend

—— As-recorded
—— Scaled

Acceleration, Velocity & Displacement Time Series
GM-10 - Coastal Chile 2015 - VA03 (NN)

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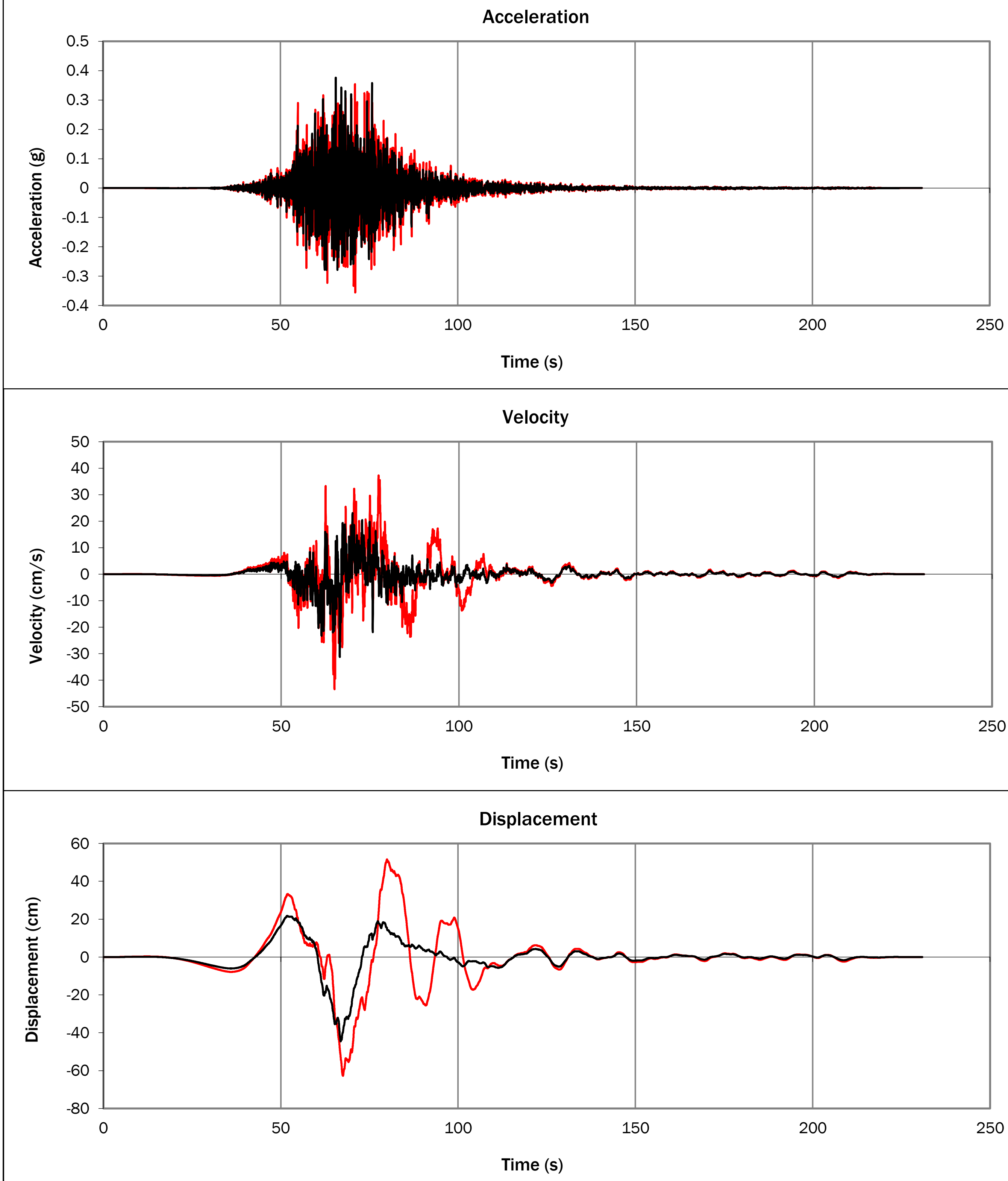
Figure A-12



Legend

—— As-recorded

—— Scaled

Acceleration, Velocity & Displacement Time Series
GM-11 - Hokkaido East 1994 - 47420 (NS)

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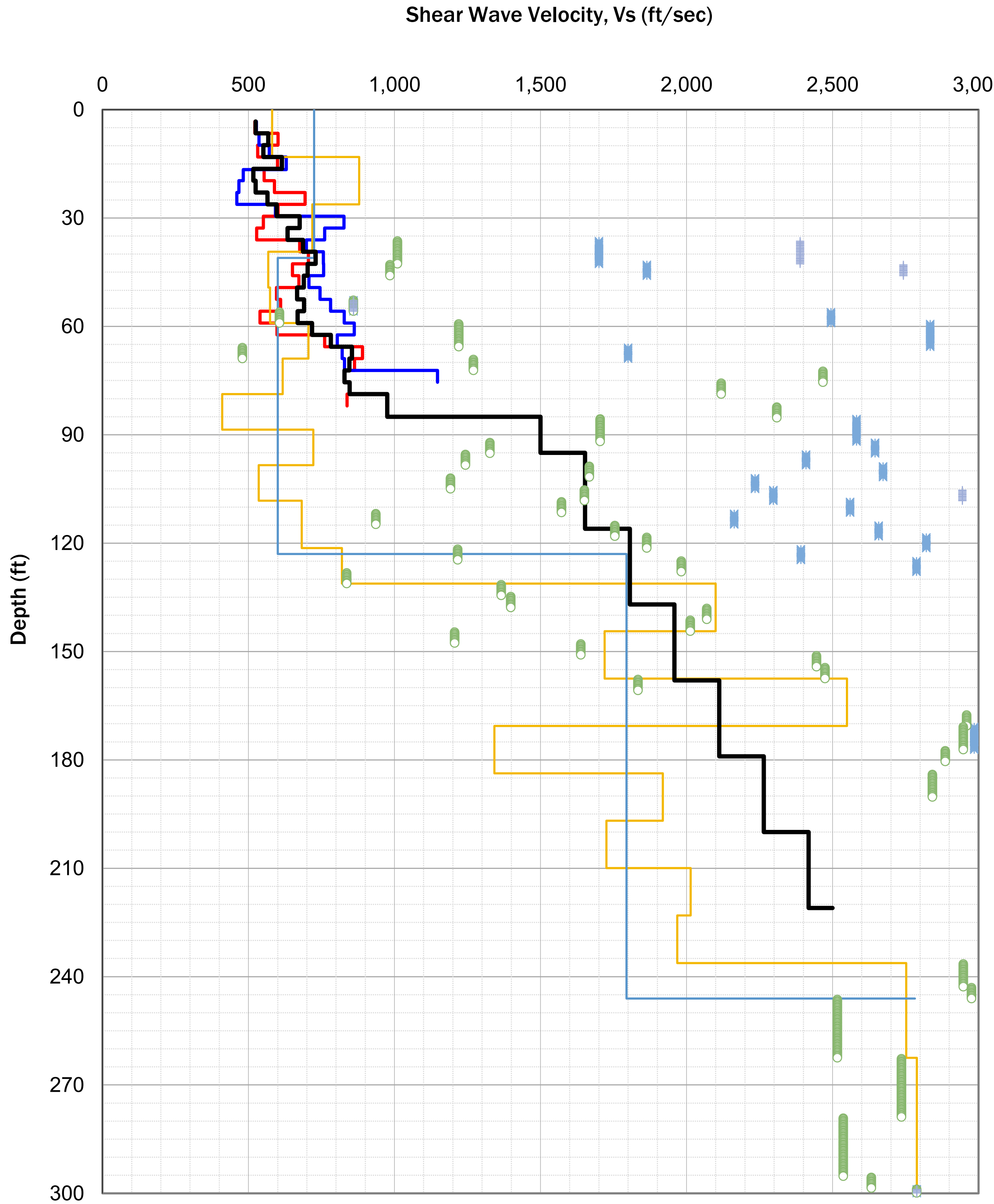
Figure A-13



- CPT-1 Vs Testing
- CPT-6 Vs Testing
- DOGAMI - CRB (Ave.)
- DOGAMI - CRB (-Std)
- DOGAMI - CRB (+Std)
- DOGAMI-13_64
- WA-DNR-08_207
- Best-Estimate

Shear Wave Velocity Profiles

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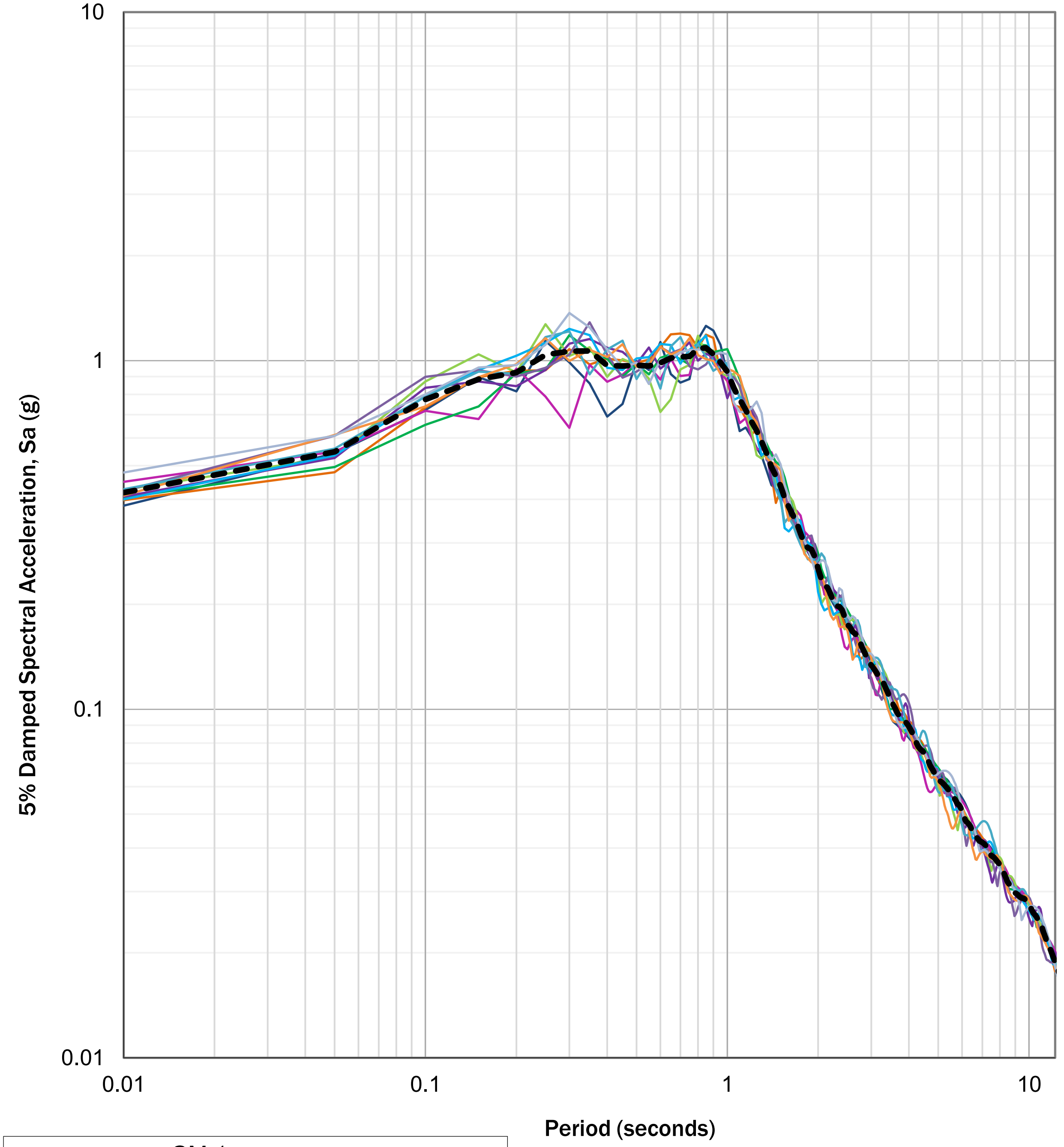
Figure A-14



MCE Surface Response Spectra
(Total Stress)

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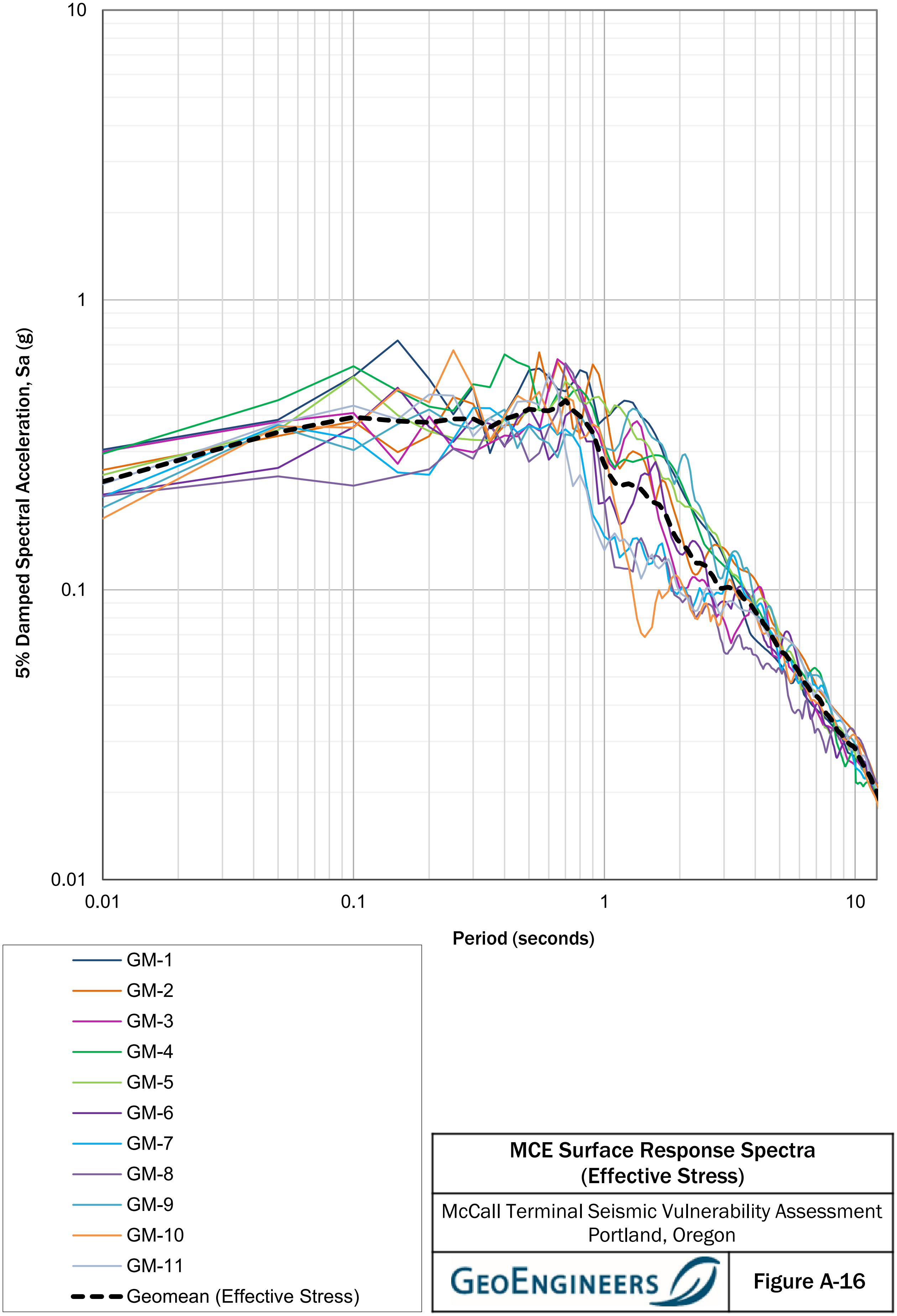
Figure A-15



MCE Surface Response Spectra (Effective Stress)

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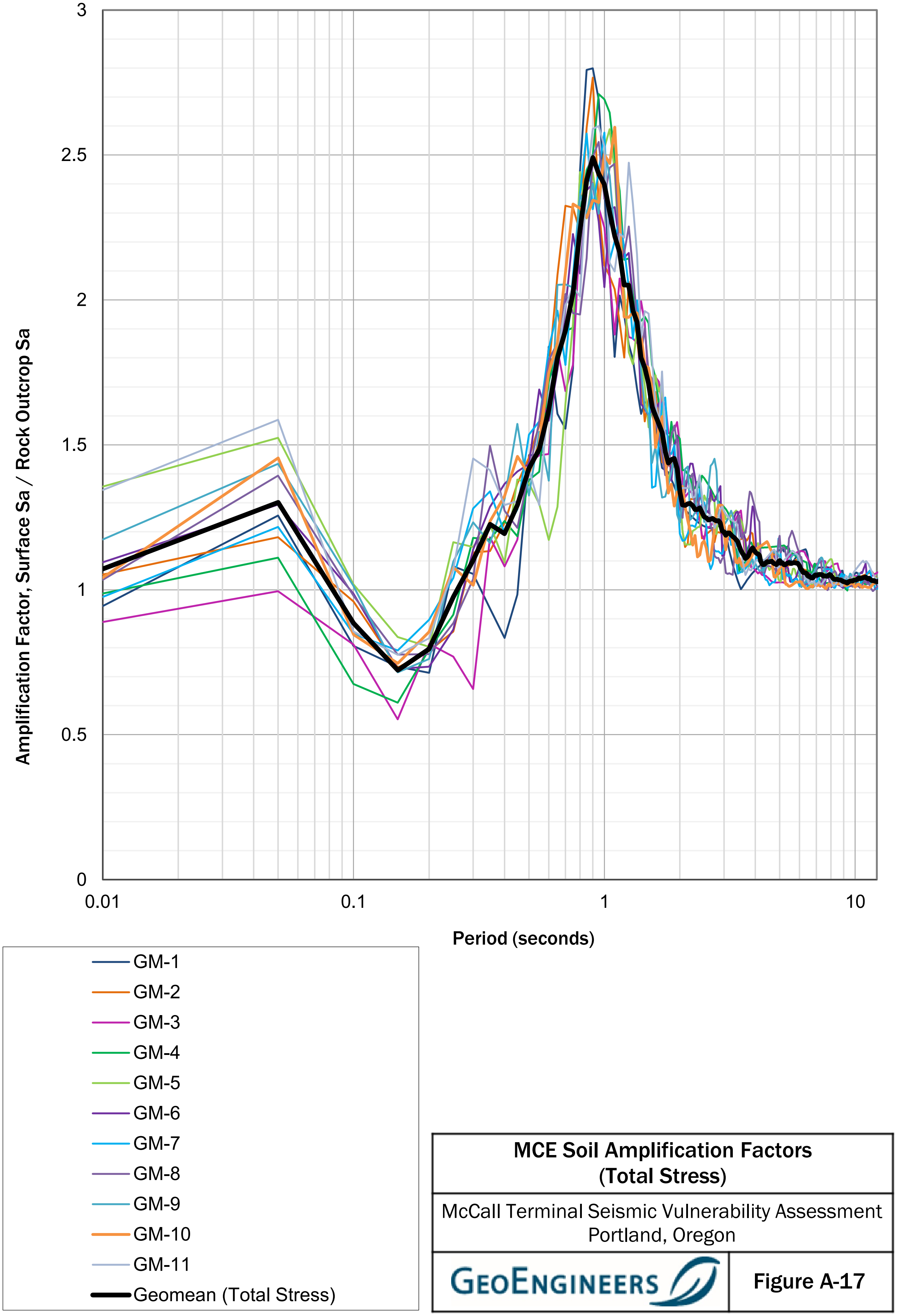
Figure A-16



MCE Soil Amplification Factors (Total Stress)

McCall Terminal Seismic Vulnerability Assessment
Portland, Oregon

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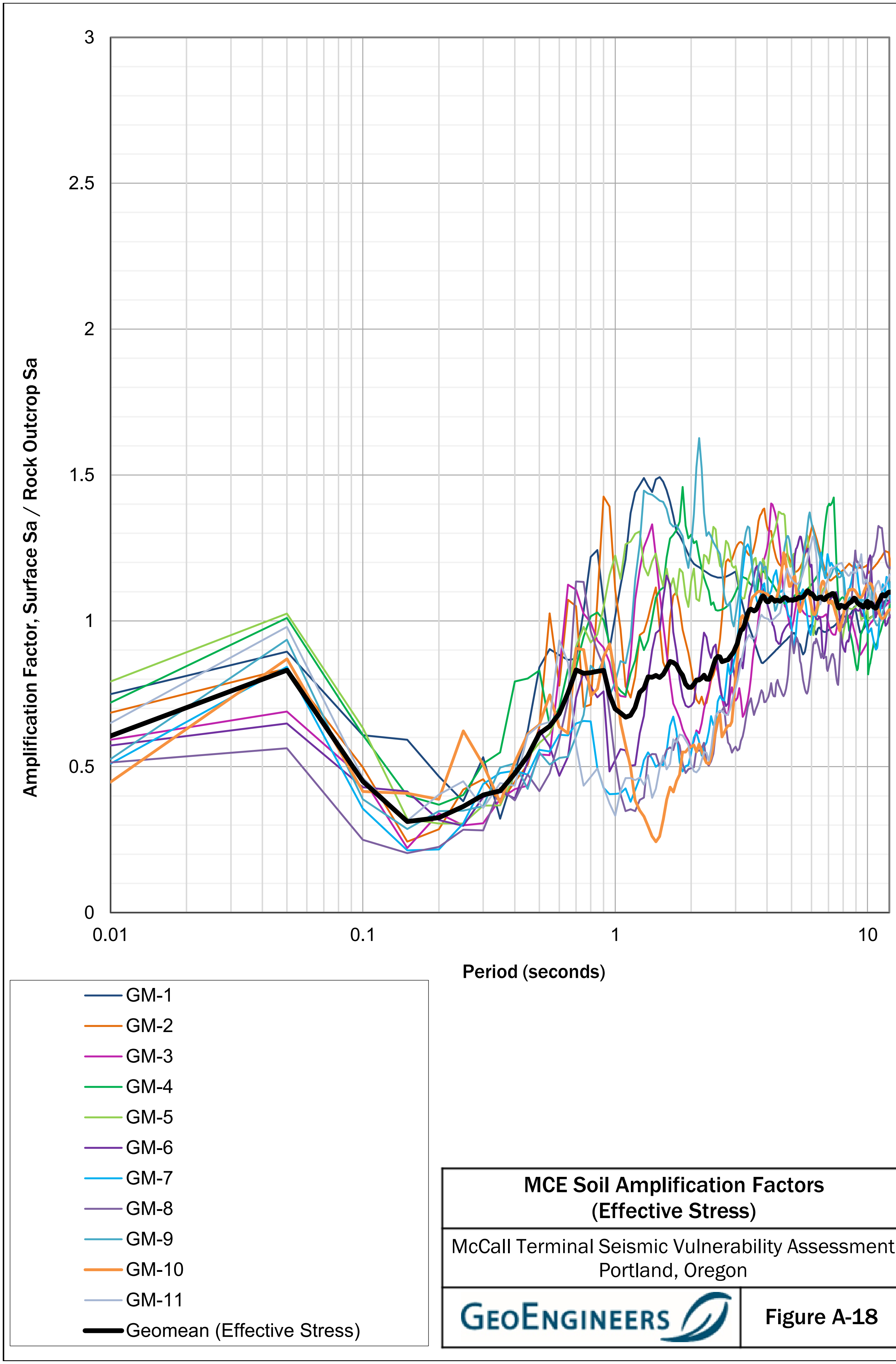
Figure A-17



26662-001-00 Date:05/03/2024

MCE Soil Amplification Factors (Effective Stress)

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Portland, Oregon

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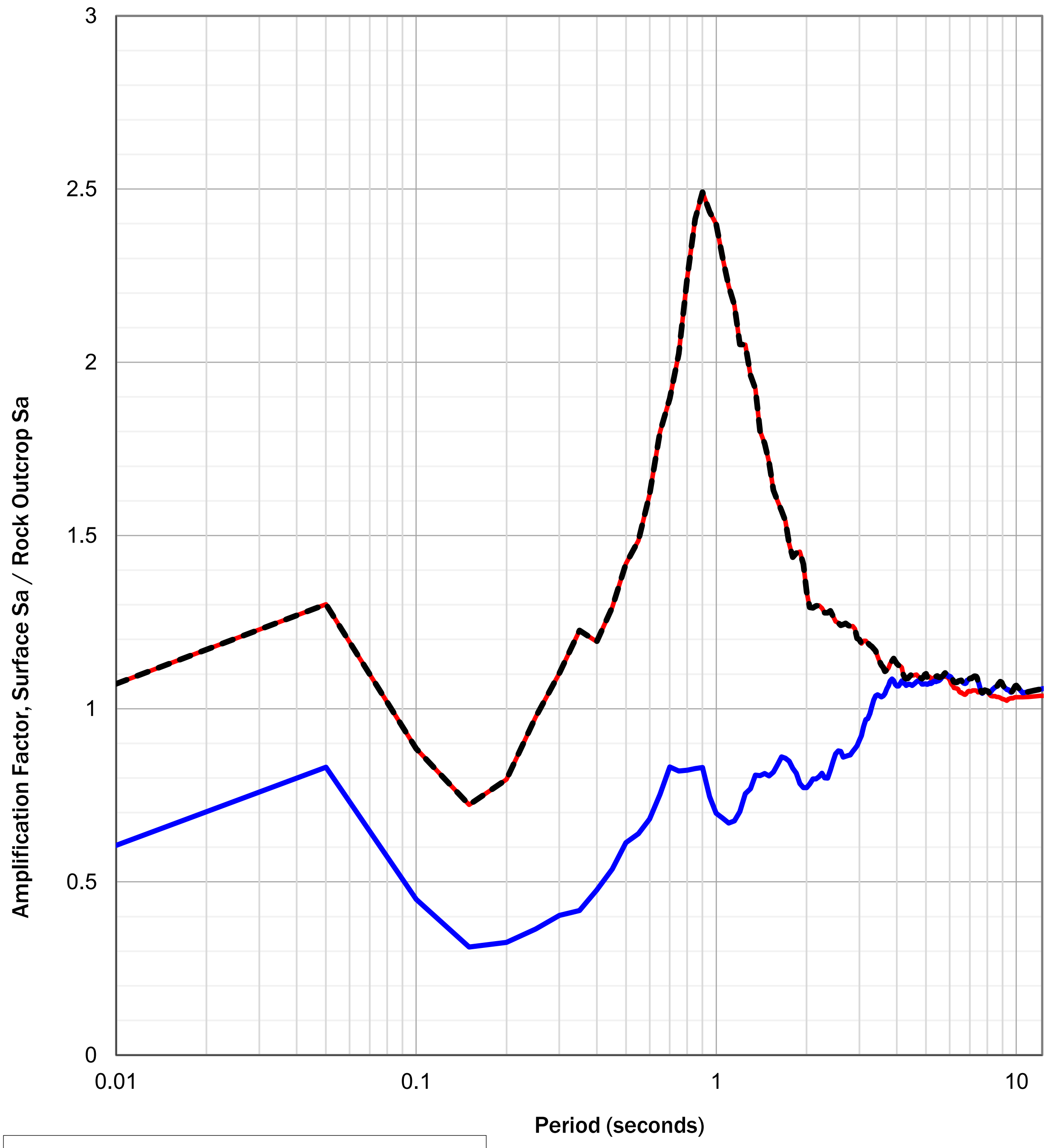
Figure A-18



Total Stress Analysis

Effective Stress Analysis

Recommended Soil AFs

MCE Soil Amplification Factors

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Portland, Oregon

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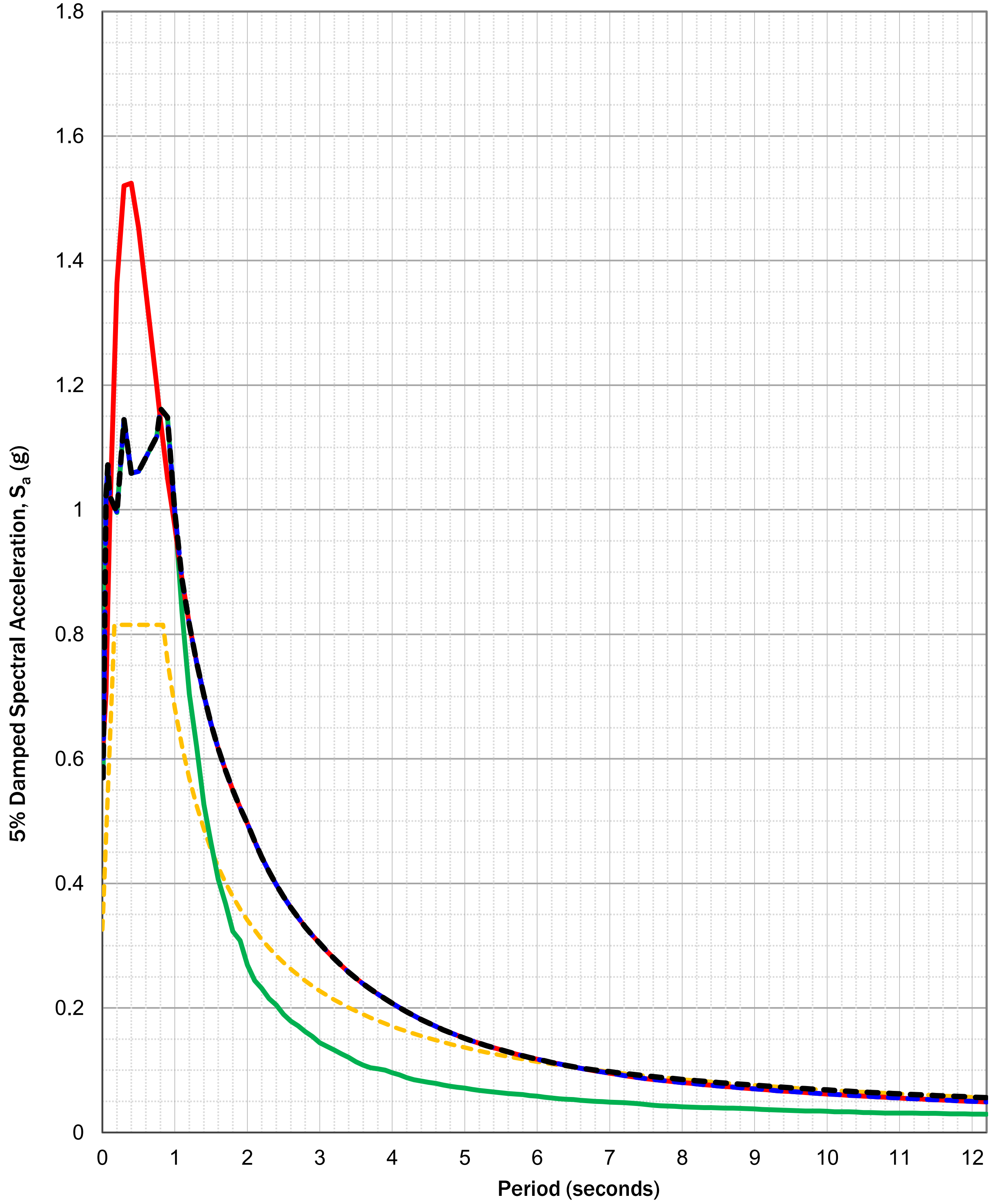
Figure A-19



Recommended Site-Specific MCE_R Response Spectrum

McCall Terminal Seismic Vulnerability Assessment
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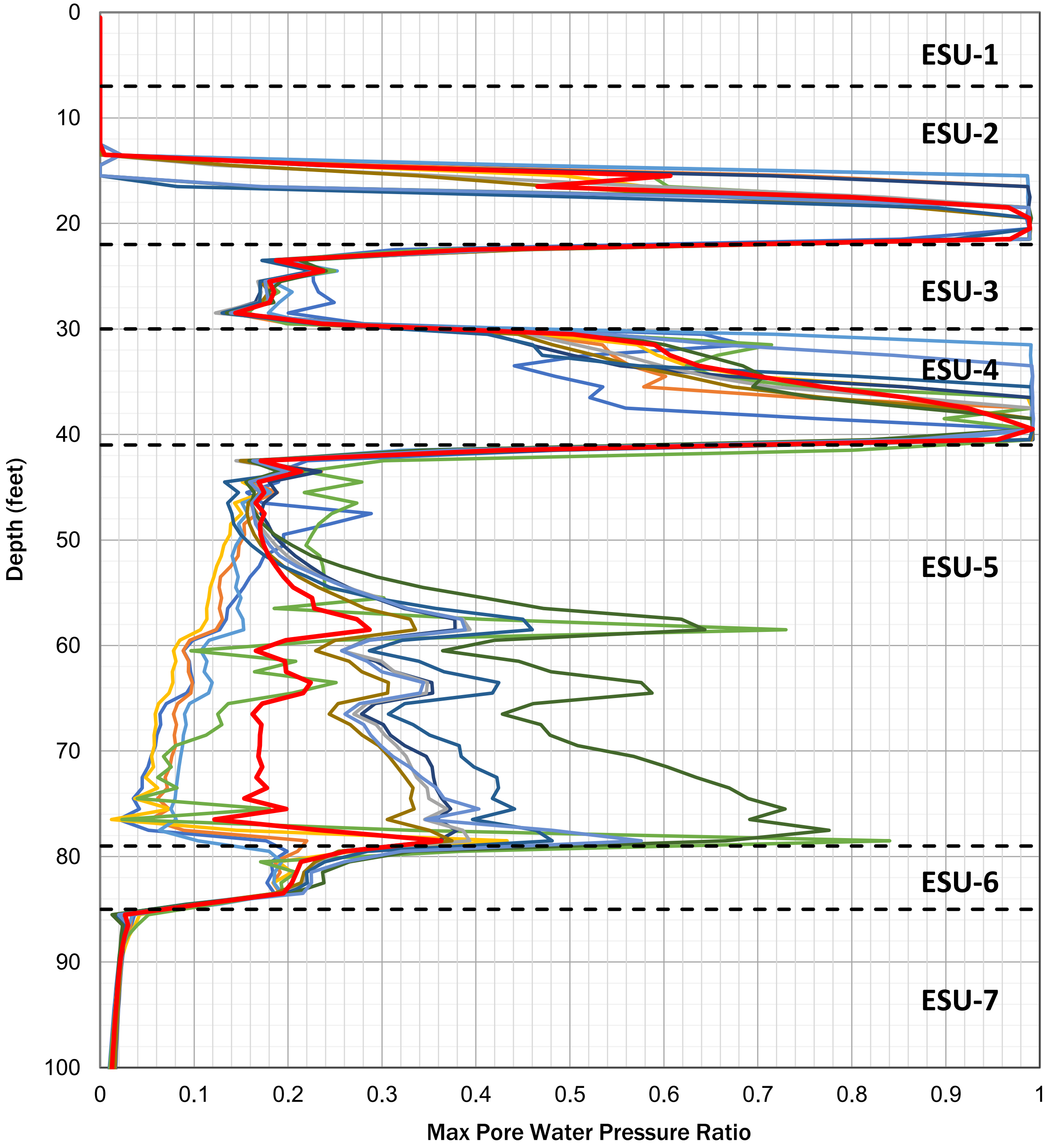
Figure A-20



26662-001-00 Date Exported: 05/03/24

Excess Pore Water Pressure Ratio

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Portland, Oregon

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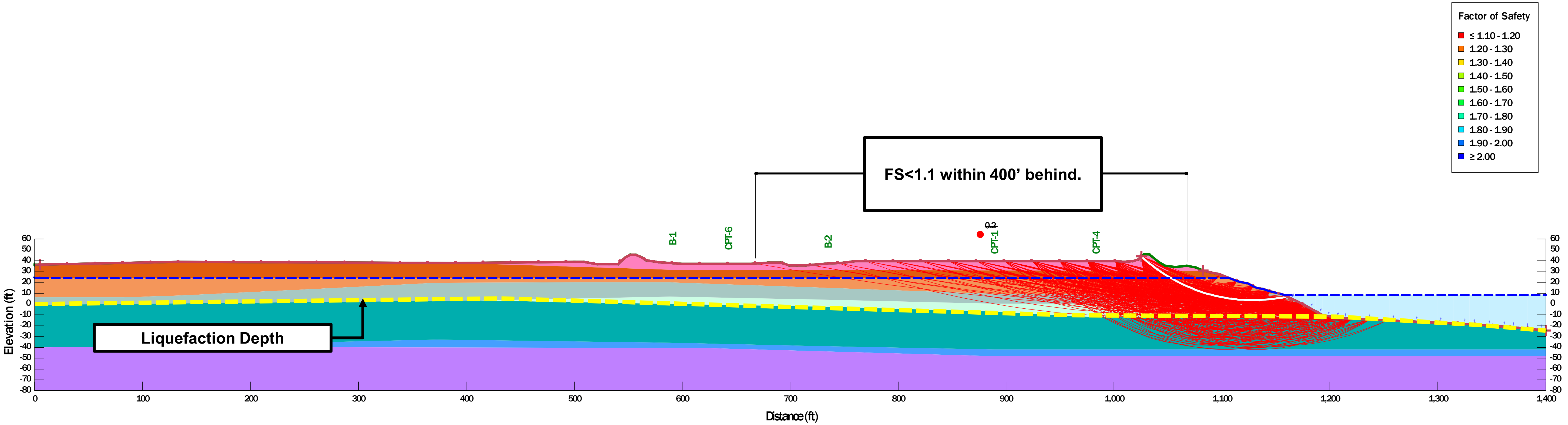
Figure A-21

Post-Earthquake Condition:

- **Liquefaction in ESU-1 through ESU-4.**
- **Flow failure within 400’ behind the riverbank.**



Slope Stability Analysis under Post-Earthquake Condition

McCall Terminal Seismic Vulnerability Assessment
Portland, Oregon

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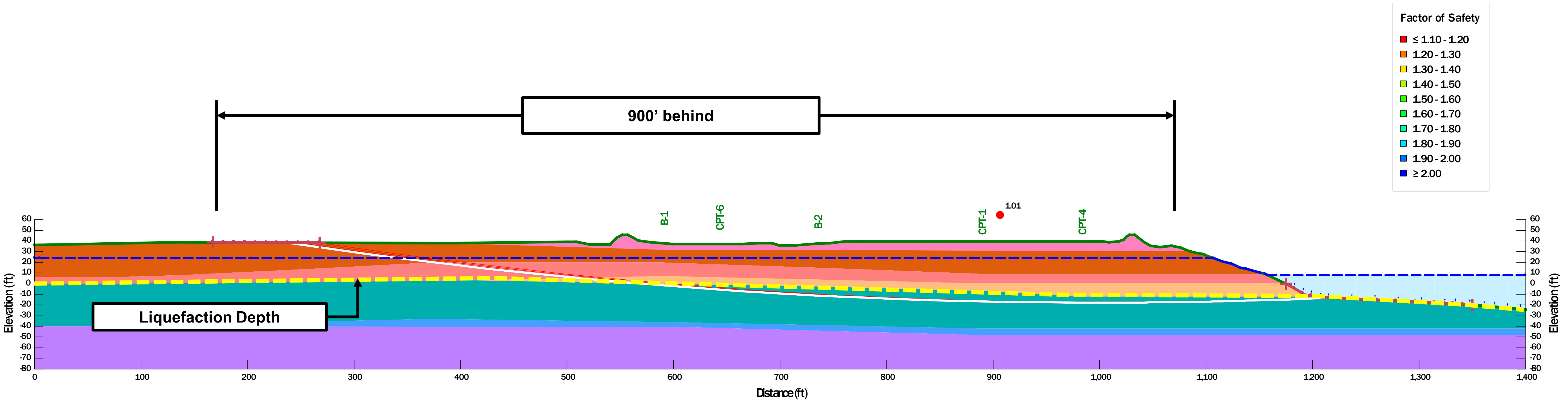
Figure A-22

Seismic Condition:

- **Liquefaction in ESU-1 through ESU-4 assumed occur at the end of earthquakes.**
- **Yield acceleration = 0.145g approximately.**
- **Estimated lateral ground deformation = 24 inches approximately.**



Slope Stability Analysis under Seismic Condition (Liquefaction Occurs at the End of Earthquakes)

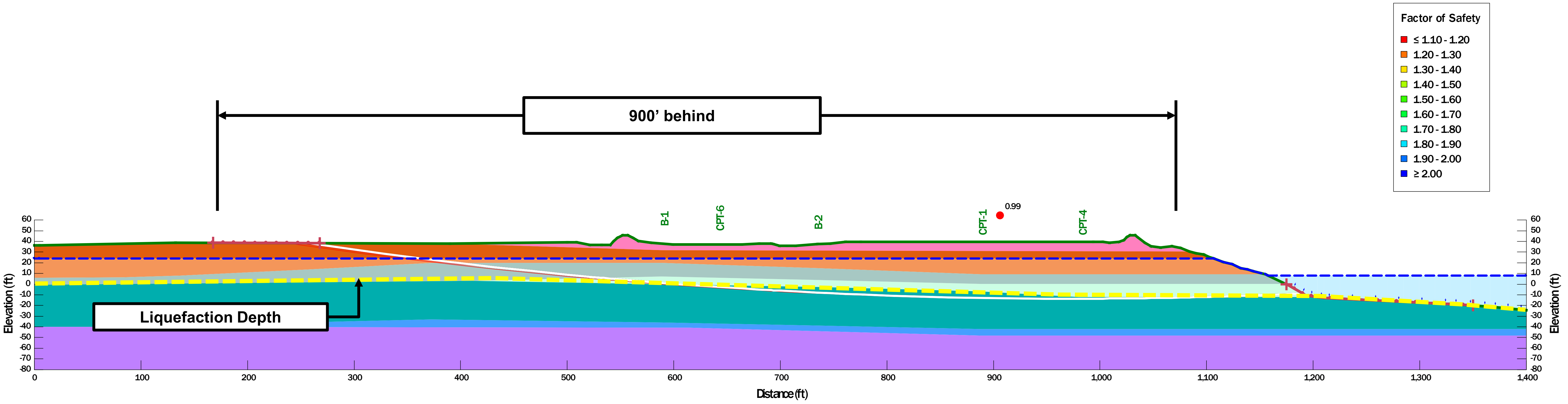
McCall Terminal Seismic Vulnerability Assessment
Portland, Oregon

GeoEngineers

Figure A-23

Seismic Condition:

- **Liquefaction in ESU-1 through ESU-4 assumed occur during earthquakes.**
- **Yield acceleration = 0.145g approximately.**
- **Estimated lateral ground deformation = 24 inches approximately.**



Slope Stability Analysis under Seismic Condition
(Liquefaction Occurs during Earthquakes)

McCall Terminal Seismic Vulnerability Assessment
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Figure A-24

Appendices

Appendix A.1

Previous Subsurface Exploration Logs and Laboratory Testing Results

APPENDIX A

FIELD EXPLORATION PROGRAM

Subsurface conditions for the project area were explored by drilling two borings using mud rotary drilling techniques. The two borings (B-1-99 and B-2-99) were drilled at selected locations on November 30, 1999, to a depth of 51.5 feet below ground surface. The borings were completed using a truck-mounted, modified Mobile B-61 drill rig owned and operated by Geo-Tech Explorations of Tualatin, Oregon under subcontract to PacRim Geotechnical Inc. The drill rig was equipped with a 2 7/8 inch outside diameter (O.D.) tricone drill bit with 2¾-inch O.D. drill rods.

The approximate locations of the explorations are shown on Figure 2 in the main body of the report. Subsurface information from the borings is shown on the summary boring logs included in this appendix as Figures A-1 and A-2. Figure A-3 is a key to symbols and descriptions used on the boring logs.

Soil samples were obtained from all of the borings at 2.5 feet intervals to a depth of 10 feet and 5-foot intervals thereafter. Samples were obtained using a 2-inch outside diameter Standard Penetration Test (SPT) sampler. A relatively undisturbed sample of the organic silt, encountered in boring B-2-99 was obtained at a depth of 40 to 42 feet with a Shelby tube. The sampler was driven into the soil a distance of 18 inches using a 140-pound hammer freely falling a distance of 30 inches. The hammer was operated using a rope and cathead system. Recorded blows for each 6 inches of sampler penetration are shown on the boring logs. The blow counts provide a qualitative measure of the relative density of cohesionless soil, or the relative consistency of fine-grained soils. Representative portions of the samples were placed in moisture tight plastic jars and transported to our laboratory for further observation and testing.

The explorations were located in the field by measurement from existing features shown on site plans. The approximate ground surface elevations at the exploration locations were also determined based on the available topographic maps. The locations and elevations of the explorations should be considered accurate only to the degree implied by the methods used.

PacRim personnel were present throughout the field exploration program to observe the borings, assist in sampling, and to prepare descriptive logs of the explorations. Soils were classified in general accordance with ASTM D-2488, *Standard Practice for Description and Identification of Soils (Visual-Manual Procedure)*. The exploration logs in this appendix represent our interpretation of subsurface conditions observed in the field, and the results of laboratory testing.

Other Tests	Sample No.	Dry Density (pcf)	Moisture Content (%)	Blows/6 inches	Sample	Depth (feet)	USCS	Graphic Symbol	DESCRIPTION	Elevation (feet)	Groundwater	Well Detail
									Loose, brown, fine SAND; dry. (ARTIFICIAL FILL)			
	1		19	4 2 3	◨							
						5						
	2			5 4 6	◨							
									Becomes medium dense.			
	3		16	5 6 5	◨							
							SP					
						10						
	4			6 6 6	◨							
											ATD ▼	
						15			Becomes loose and wet, trace silt.			
	5		37	5 4 4	◨							
									Medium dense, gray, fine SAND; wet. (ALLUVIUM)			
						20						
	6		31	6 6 8	◨							
						25			Becomes loose.			
	7			4 4 3	◨		SP					
						30			Scattered seams of gray organic silt with organic debris.			
FC	8		33	4 4 6	◨							
									Medium stiff, dark brown, organic SILT, trace clay; moist.			
						35						
	9		100	2 3 5	◨		OL					
									Interbedded, loose to medium stiff, dark gray, clayey SILT and fine sandy SILT to silty fine SAND; moist to wet; scattered organic debris and fine sand			

Surface Elevation:
Date Completed: 11/30/99
Logged By: KJL
Equipment: Mobile B-59
Drilling Method: Mud Rotary
Hammer System: 140# Rope and Cathead

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LOG OF BORING B-1

SHEET 1 OF 2

Project No. 097-001

FIGURE A-1

Other Tests	Sample No.	Dry Density (pcf)	Moisture Content (%)	Blows/6 inches	Sample	Depth (feet)	USCS	Graphic Symbol	DESCRIPTION	Elevation (feet)	Groundwater	Well Detail
	10		87	2 2 3					layers.			
	11			3 3 5		45			Mottled dark brown and green-gray, grading geen gray at 46 feet.			
	12			5 5 10		50						
									Borehole backfilled with soil cuttings and bentonite chips.			
						55						
						60						
						65						
						70						
						75						

Date Completed: 11/30/99
Logged By: KJL
Equipment: Mobile B-59
Drilling Method: Mud Rotary
Hammer System: 140# Rope and Cathead

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LOG OF BORING B-1

SHEET 2 OF 2

Project No. 097-001

FIGURE A-1

Other Tests	Sample No.	Dry Density (pcf)	Moisture Content (%)	Blows/6 inches	Sample	Depth (feet)	USCS	Graphic Symbol	DESCRIPTION	Elevation (feet)	Groundwater	Well Detail
							GP		Loose, brown to black, sandy GRAVEL; moist. (ARTIFICIAL FILL)			
									Medium dense, brown, fine SAND; dry to moist.			
	1		27	5 4 7								
						5						
	2			4 4 6								
	3		28	4 4 4					Becomes loose.			
						10						
	4			4 4 4			SP					
											ATD ▼	
						15			Becomes wet, trace silt, scattered dark gray silt partings.			
MA	5		38	5 3 5								
						20						
	6			7 6 5								
									Loose, gray, fine SAND; wet; scattered organic debris and dark gray silt seams with organics. (ALLUVIUM)			
						25						
MA	7		34	4 3 5								
							SP		Becomes medium dense.			
						30						
	8		30	8 8 10								
									Medium stiff, brown, organic SILT with fine sand; moist to wet; abundant organic debris.			
						35						
FC, Cons	9		103	2 2 3								
							OL					

Surface Elevation:
Date Completed: 11/30/99
Logged By: KJL
Equipment: Mobile B-59
Drilling Method: Mud Rotary
Hammer System: 140# Rope and Cathead

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LOG OF BORING B-2

SHEET 1 OF 2

Project No. 097-001

FIGURE A-2

RELATIVE DENSITY OR CONSISTENCY VERSUS SPT N-VALUE

COHESIONLESS SOILS			COHESIVE SOILS		
Density	N (blows/ft)	Approximate Relative Density (%)	Consistency	N (blows/ft)	Approximate Undrained Shear Strength (psf)
Very Loose	0 to 4	0 - 15	Very Soft	0 to 2	<250
Loose	4 to 10	15 - 35	Soft	2 to 4	250 - 500
Medium Dense	10 to 30	35 - 65	Medium Stiff	4 to 8	500 - 1000
Dense	30 to 50	65 - 85	Stiff	8 to 15	1000 - 2000
Very Dense	over 50	85 - 100	Very Stiff	15 to 30	2000 - 4000
			Hard	over 30	>4000

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			GROUP DESCRIPTIONS	
Coarse Grained Soils More than 50% Retained on No. 200 Sieve Size	**Gravel and Gravelly Soils** More than 50% of Coarse Fraction Retained on No. 4 Sieve	Clean Gravel (little or no fines)	GW	Well-graded GRAVEL
			GP	Poorly-graded GRAVEL
		Gravel with Fines (appreciable amount of fines)	GM	Silty GRAVEL
			GC	Clayey GRAVEL
	Sand and Sandy Soils 50% or More of Coarse Fraction Passing No. 4 Sieve	Clean Sand (little or no fines)	SW	Well-graded SAND
			SP	Poorly-graded SAND
		Sand with Fines (appreciable amount of fines)	SM	Silty SAND
			SC	Clayey SAND
Fine Grained Soils 50% or More Passing No. 200 Sieve Size	**Silt and Clay**	Liquid Limit Less than 50%	ML	SILT
			CL	Lean CLAY
			OL	Organic SILTor CLAY
	Silt and Clay	Liquid Limit 50% or More	MH	Elastic SILT
			CH	Fat CLAY
			OH	Organic SILTor CLAY
Highly Organic Soils			PT	PEAT

DESCRIPTORS FOR SOIL STRATA AND STRUCTURE

General Thickness or Spacing	Parting:	less than 1/16 in.	Structure	Pocket:	Erratic, discontinuous deposit of limited extent	General Attitude	Near horizontal:	0 to 10 deg.
	Seam:	1/16 to 1/2 in.					Low angle:	10 to 45 deg.
	Layer:	1/2 to 12 in.		Lens:	Lenticular deposit		High angle:	45 to 80 deg.
	Stratum:	greater than 12 in.		Varved:	Alternating seams of silt and clay		Near vertical:	80 to 90 deg.
	Scattered:	less than 1 per ft.		Laminated:	Alternating seams			
	Numerous:	more than 1 per ft.		Interbedded:	Alternating layers			

Notes:

1. Sample descriptions in this report are based on visual field and laboratory observations, which include density/consistency, moisture condition, grain size, and plasticity estimates, and should not be construed to imply field nor laboratory testing unless presented herein. Visual-manual classification methods of ASTM D 2488 were used as an identification guide. Where laboratory data are available, soil classifications are in general accordance with ASTM D 2487.

2. Solid lines between soil unit descriptions indicate change in interpreted geologic unit. Dashed lines indicate stratigraphic change within the unit.

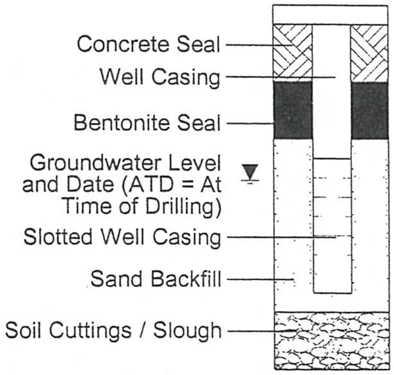
LABORATORY TEST SYMBOLS

AL	Atterberg Limits
FC	Fines Content
GSD	Grain Size Distribution
MC	Moisture Content
MD	Moisture Content/Dry Density
Comp	Compaction Test (Proctor)
SG	Specific Gravity
CBR	California Bearing Ratio
RM	Resilient Modulus
Perm	Permeability
TXP	Triaxial Permeability
Cons	Consolidation
VS	Vane Shear
DS	Direct Shear
UC	Unconfined Compression
TXS	Triaxial Compression
UU	Unconsolidated, Undrained
CU	Consolidated, Undrained
CD	Consolidated, Drained

SAMPLE TYPE SYMBOLS

◧	Std. Penetration Test (2.0" OD)
☒	Ring Sampler (3.25" OD)
⧗	California Sampler (3.0" OD)
■	Undisturbed Tube Sample
G	Grab Sample
^ ^	Core Run
▯	Non-standard Penetration Test (with split spoon sampler)

GROUNDWATER WELL COMPLETIONS



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KEY TO EXPLORATION LOGS

Project No. 097-001

FIGURE A-3

APPENDIX B

LABORATORY TESTING

PacRim personnel, and our subcontract Laboratory – Rosa Environmental and Geotechnical Labs, performed laboratory soil tests in general accordance with appropriate ASTM test methods. We tested selected soil samples to determine moisture content, grain size distribution, fines content, Atterberg Limits, and consolidation characteristics. The test procedures and test results are discussed below.

MOISTURE CONTENT

Laboratory tests were conducted to determine the moisture content of selected soil samples, in general accordance with ASTM D-2216. Test results are indicated at the sampled intervals on the appropriate boring logs in Appendix A.

GRAIN SIZE DISTRIBUTION

Grain size distribution was determined for selected samples in general accordance with ASTM D-422. Results of grain size analyses are plotted on Figure B-1.

FINES CONTENT

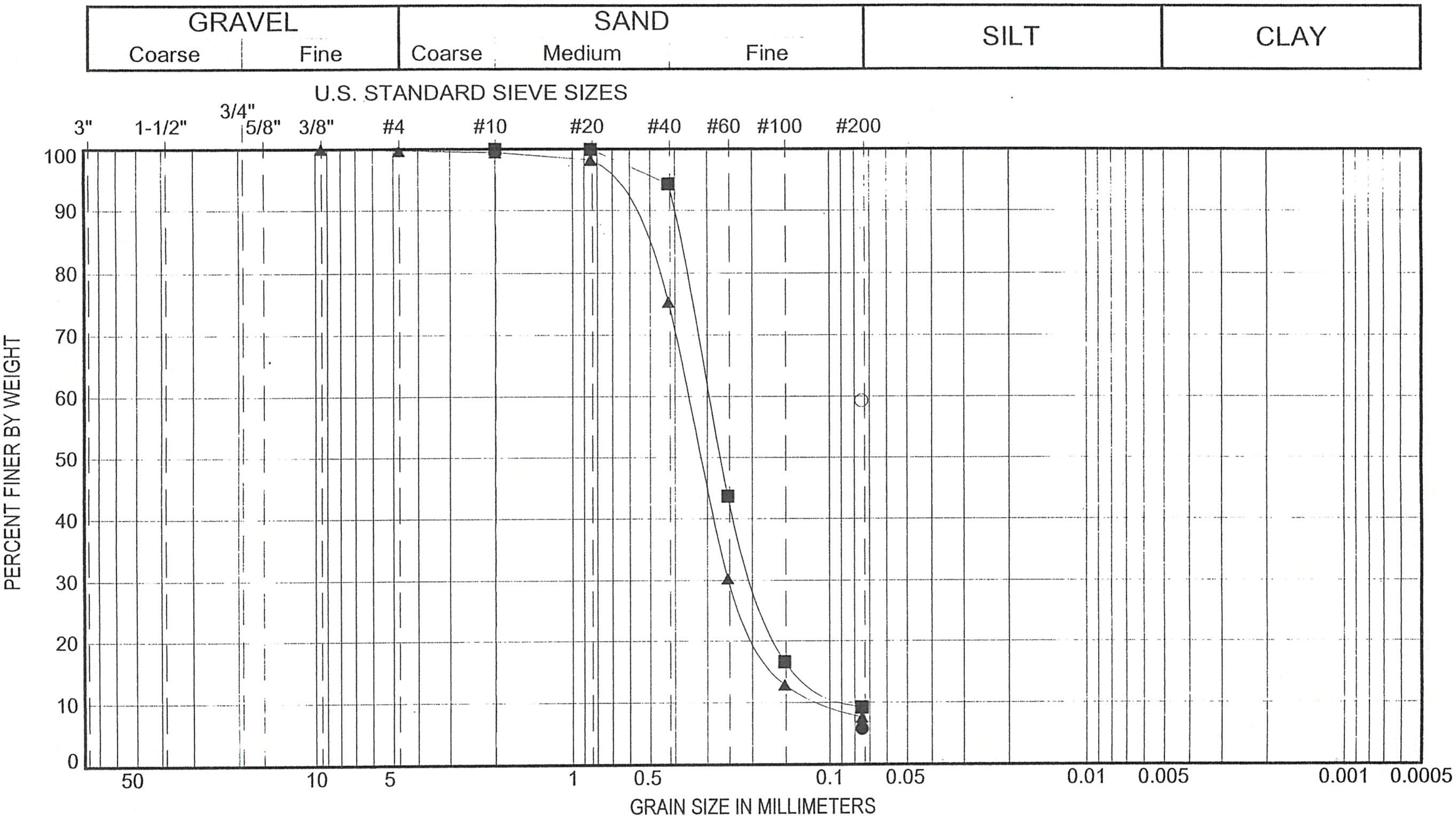
Selected soil samples were washed through the U.S. Standard No. 200 sieve in general accordance with ASTM D 1140 to determine the percentage of fines. The test results are plotted on the Grain Size Distribution plot Figure B-1.

ATTERBERG LIMITS

Atterberg Limits indices were determined for sample S-11in Boring B-2-99. The test was performed in general accordance with ASTM D-4318. The results are shown on Figure B-2.

CONSOLIDATION TEST

Consolidation characteristics were determined for sample S-10 in Boring B-2-99. The test was performed in general accordance with ASTM D-2435. The test results are shown on Figure B-3.



SYMBOL	SAMPLE		DEPTH (ft)	CLASSIFICATION	% MC	LL	PL	PI	% Gravel	% Sand	% Fines
●	B-1	8	30.0 - 31.5	Gray, fine SAND with silt (SP)	33						6
■	B-2	5	15.0 - 16.5	Brown, fine SAND with silt (SP-SM)	38				0	91	9
▲	B-2	7	25.0 - 26.5	Black, fine to medium SAND with organics and silt (SP-SM)	34				0	92	8
○	B-2	9	35.0 - 36.5	Brown, orgainc SILT with fine sand (OL)	103						59

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GRAIN SIZE ANALYSIS TEST RESULTS

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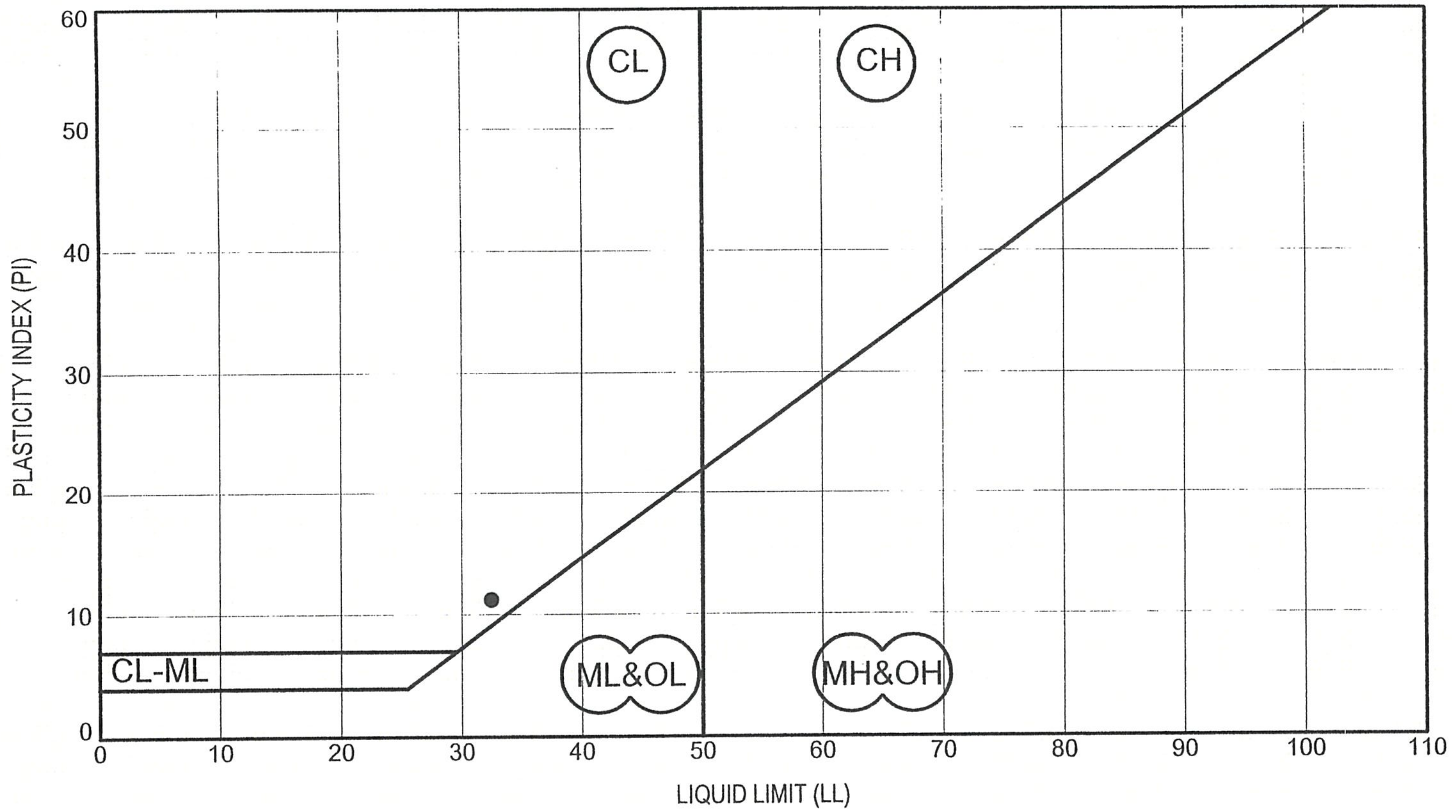
FIGURE 1



SYMBOL	SAMPLE		DEPTH (ft)	CLASSIFICATION	% MC	LL	PL	PI	% Fines
●	B-2	11	42.0 - 43.5	Gray, silty CLAY (CL)		32	21	11	

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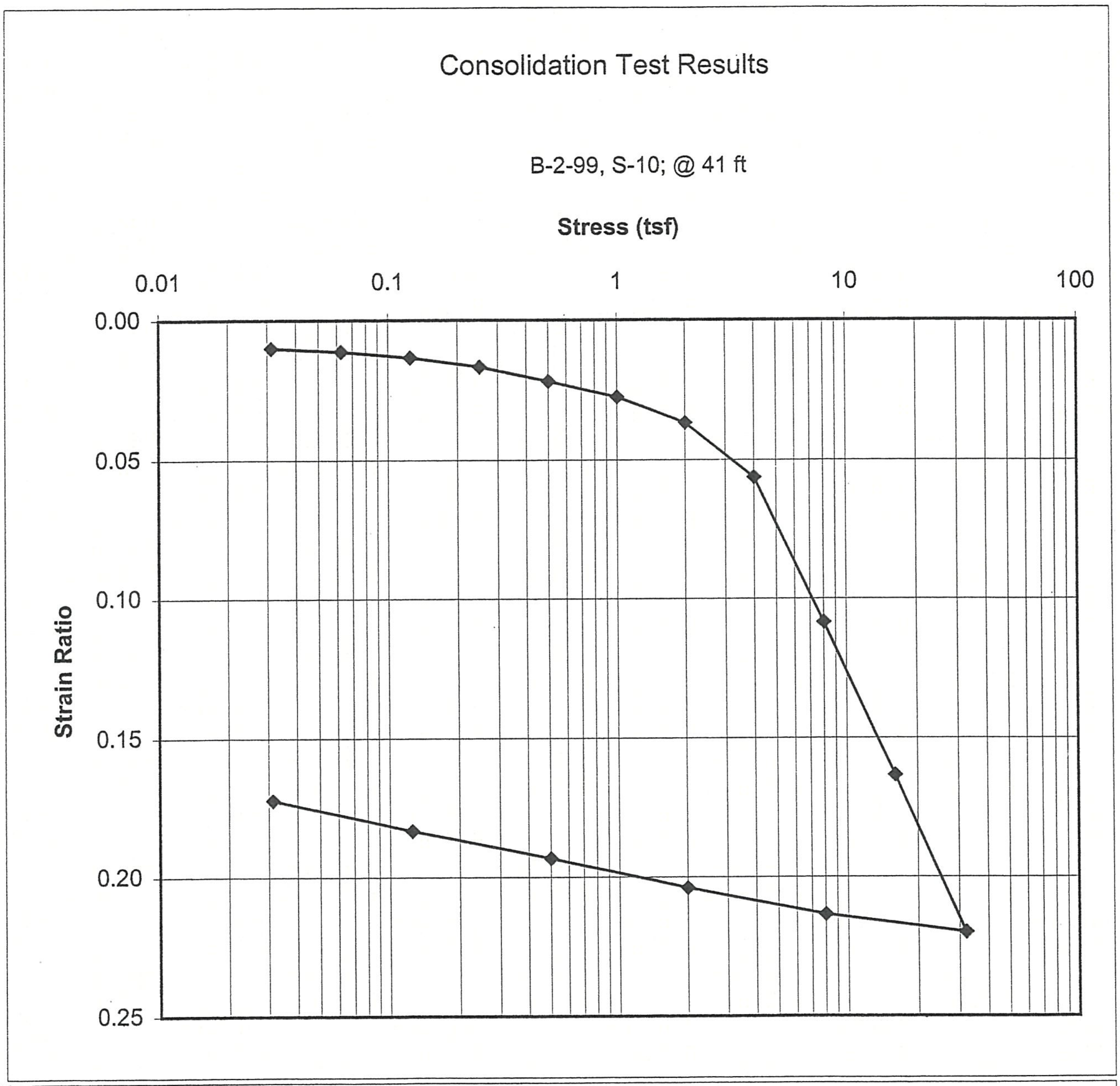
ATTERBERG LIMITS TEST RESULTS

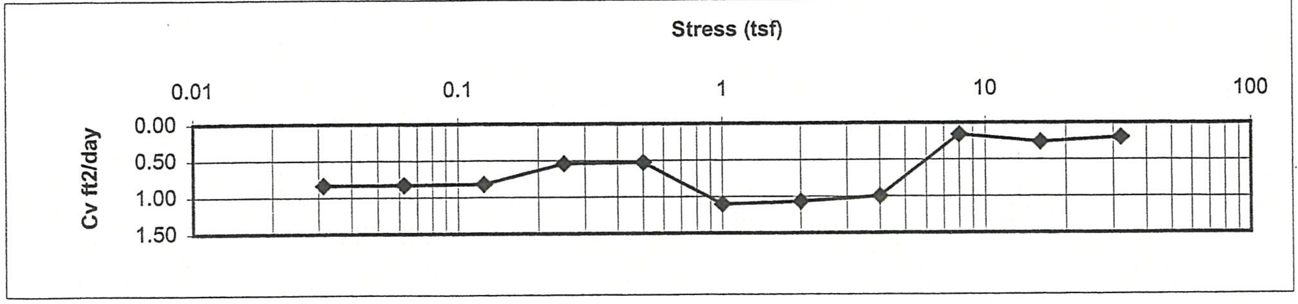
Project No. 097-001

FIGURE 1

Rosa Environmental and Geotechnical Laboratory, LLC

PacRim Geotechnical, Inc.
097-001





APPENDIX A
FIELD EXPLORATIONS

Site subsurface conditions were explored on August 10 and 11, 2022. The exploration program included six cone penetration test (CPT) soundings (CPT-1 through CPT-6) and two geotechnical borings (B-1 and B-2) at the approximate locations shown on Figure 2. CPT-1 was advanced 85.3 feet (ft) below ground surface (bgs), CPT-2 78.6 ft bgs, CPT-3 47 ft bgs, CPT-3 76 ft bgs, CPT-4 83.5 ft bgs, and CPT-6 76.6 ft bgs.

B-1 and B-2 were advanced 91 ft bgs and 80 ft bgs respectively. The exploration locations were sited by measuring from existing infrastructure. Ground surface elevations at the exploration locations were not determined.

The CPT soundings were advanced by Oregon Geotechnical Explorations of Keizer, Oregon, subcontracted by Landau Associates, Inc. (Landau). The geotechnical borings were advanced by Holt Services and subcontracted by Landau.

The field exploration program was coordinated and monitored by Landau personnel, who also obtained representative soil and core samples, maintained a detailed record of the subsurface soil and groundwater conditions observed, and described both the soil and rock encountered by visual and textural examination. Each representative soil type was described using the soil classification system shown on Figure A1, in general accordance with ASTM International (ASTM) standard test method D2488, *Standard Recommended Practice for Description of Soils (Visual-Manual Procedure).* Landau personnel collected 15 ft of core run to determine the rock quality designation (RQD) of the core.

Samples collected in this manner were taken to Landau's soils laboratory for further examination and testing. A discussion of laboratory test procedures and the laboratory test results are included in Attachment 2.

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

Summary boring logs are provided on Figures A2 and A3. These logs represent Landau's interpretation of the subsurface conditions identified during the field exploration program. The stratigraphic contacts shown on the summary logs represent the approximate boundaries between soil types; actual transitions may be more gradual. The soil and groundwater conditions depicted are for the specific dates and locations reported and are not necessarily representative of other locations and dates.

Summary logs of the CPT soundings are included at the end of Appendix A.

Soil Classification System

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

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

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

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

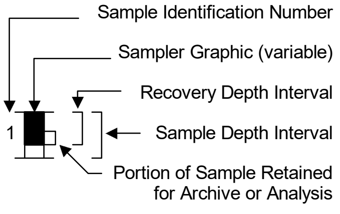
4. Soil density or consistency descriptions are based on judgement using a combination of sampler penetration blow counts, drilling or excavating conditions, field tests, and laboratory tests, as appropriate.

Drilling and Sampling Key

SAMPLER TYPE & METHOD

Graphic	Code	Description
☒	a	3.25-in OD, 2.42-in ID Split Spoon
☒	b	2.00-in OD, 1.50-in ID Split Spoon
◧	c	Shelby Tube
G	d	Grab Sample
◘	e	Single-Tube Core Barrel
◘	f	Double-Tube Core Barrel
☒	g	2.50-in OD, 2.00-in ID WSDOT
☒	h	3.00-in OD, 2.37-in ID Mod. Calif.
☒	i	Other - See text if applicable
	1	300-lb Hammer, 30-inch Drop
	2	140-lb Hammer, 30-inch Drop
	3	Pushed Sample
	4	Vibrocore (Rotosonic/Geoprobe)
	5	Other - See text if applicable
	6	Piston Extraction

SAMPLE NUMBER & INTERVAL



Field and Lab Test Data

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

Groundwater

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

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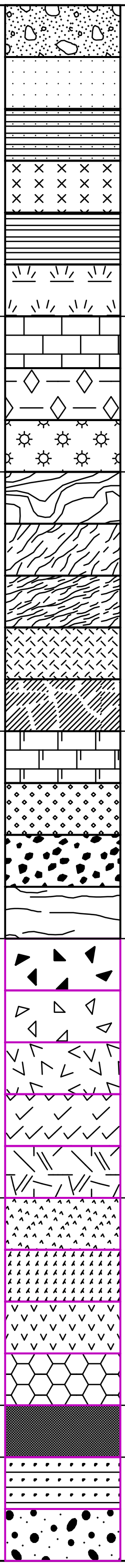
Portland, Oregon

Soil Classification
System and Key

Figure
A1.1

Rock Classification System

Primary Rock Types



Sedimentary	Clastic	Conglomerate
		Sandstone/ Sedimentary Quartzite
		Siltstone/Graywacke
		Claystone/Mudstone
		Shale
		Coal
	Crystalline	Limestone/Dolomite
		Gypsum/Halite/ Anhydride
		Chert
Metamorphic	Foliated	Gneiss/Schist
		Schist/Talc
		Phyllite
		Slate
		Mylonite
	Massive	Marble
		Quartzite
		Hornfels
		Serpentine/Soapstone/ Greenstone
Igneous	Phaneritic (Individual crystals distinguishable with unaided eye.)	Granite
		Monzonite/ Quartz Monzonite
		Granodiorite/ Diorite Quartz Diorite
		Gabbro
		Diabase/Perioite
	Aphanitic (Crystals not distinguishable or visible crystals in a fine-grained groundmass.)	Rhyolite
		Latite/Quartz Latite
		Dacite/Andesite
		Basalt
	Glassy	Obsidian/ Pumice/Scoria
Pyroclastic		Agglomerate/ Breccia/Tuff
		Bombs/Blocks/ Cinders/Ash

Relative Hardness

Term	Designation	Approx. Unconfined Compressive Strength	Field Identification
Extremely Soft	R0	< 100 psi	Moldable or friable with finger pressure.
Very Soft	R1	100 - 1,000 psi	Peeled by knife with ease. Crumbles under firm blows with point of a geology pick.
Soft	R2	1,000 - 4,000 psi	Peeled by knife with difficulty. Shallow indentation made by firm blow of geology pick.
Medium Hard	R3	4,000 - 8,000 psi	Scratched by knife with ease. Fractured with a single firm blow of hammer/geology pick.
Hard	R4	8,000 - 16,000 psi	Scratched by knife with difficulty. Several hard hammer blows required to fracture.
Very Hard	R5	> 16,000 psi	Cannot be scratched with knife. Many hard hammer blows required to fracture or chip.

Relative Weathering

Fresh	Crystals are bright; no discoloration in rock fabric
Slightly Weathered	Some discoloration in rock fabric; decomposition extends up to 1 inch
Moderately Weathered	Rock mass is decomposed 50 % or less
Predominately Decomposed	Rock mass is more than 50 % decomposed; can be excavated with pick
Decomposed	Completely decomposed; can be reduced to soil with hand pressure

Structural Descriptions

Spacing (in)	Bedding/Foliation	Joint/Shear/Fracture	Attitude and Angle
< 2	Very Thin	Very Close	Horizontal (0-5°)
2 - 12	Thin	Close	Shallow or Low Angle (5-35°)
12 - 36	Medium	Moderately Close	Moderately Dipping (35-55°)
36 - 120	Thick	Wide	Steep or High Angle (55-85°)
> 120	Very Thick	Very Wide	Vertical (85-90°)

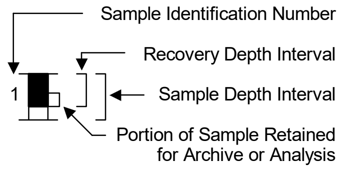
Core Recovery and Rock Quality Designation

$$\text{Core Recovery} = \frac{\textit{length of core recovered}}{\textit{total length of core run}} \times 100$$

$$\text{RQD} = \frac{\textit{total length of all pieces 4 inches or greater}}{\textit{total length of core run}} \times 100$$

Coring and Sampling Key

SAMPLE NUMBER & INTERVAL



SAMPLER TYPE

Code	Description
e	Other - See text if applicable
f	Single Tube Core Barrel
g	Double Tube Core Barrel
4	Other - See text if applicable
5	Air Rotary
6	Wash Rotary
7	Rotosonic

Field and Lab Test Data

Code	Description
W = 10	Moisture Content, %
D = 120	Dry Density, pcf
CS = 1.0	Compressive Strength, tsf
TS = 0.5	Tensile Strength, tsf
GT	Other Geotechnical Testing
CA	Chemical Analysis

Groundwater

∇ ATD	Approximate water elevation at time of drilling (ATD) or on date noted. Groundwater levels can fluctuate due to precipitation, seasonal conditions, and other factors.

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McCall Terminal
Upgrade Project
Portland, Oregon

Rock Classification
System and Key

Figure
A1.2

B-1

LAI Project No: 1374022.010

SAMPLE DATA

SOIL PROFILE

Drilling Method: Mud Rotary

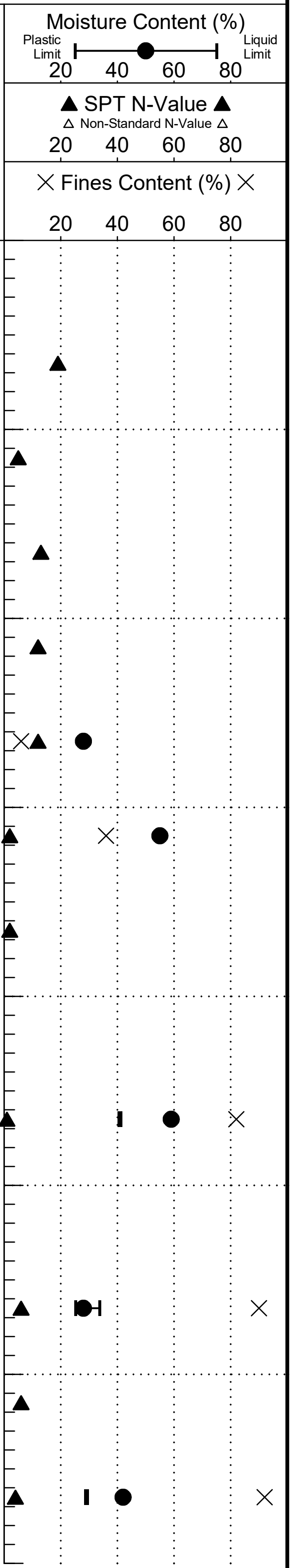
Ground Elevation (ft): Not Measured

Drilled By: Holt

Logged By: BMD Date: 08/11/22

Groundwater: Groundwater Not Determined

Depth (ft)	Sample Number & Interval	Sampler Type	Blows/Foot	Test Data	USCS Symbol	Description
0					SP	Dark gray fine SAND (medium dense, moist) **(FILL)**
	S-1	b2	19			
5	S-2	b2	5			
	S-3	b2	13			
10	S-4	b2	12			
	S-5	b2	12	W = 28 -200 = 6		
15	S-6	b2	2	W = 55 -200 = 36		grades to very loose, silty
	S-7	b2	2		ML	Gray mottled brown sandy SILT (soft, moist) **(ALLUVIUM)** 2-inch fine sandy lense
20	S-8	c3				
	S-9	b2	1	W = 59 GS AL		
25	S-10	c3				
	S-11	b2	6	W = 28 GS AL		grades to medium stiff, with sand
30	S-12	b2	6			increase in mottling and apparant weathering
	S-13	b2	4	W = 42 GS AL		
35						



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

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McCall Terminal
Upgrade Project
Portland, Oregon

Log of Boring B-1

Figure
A2
(1 of 3)

B-1

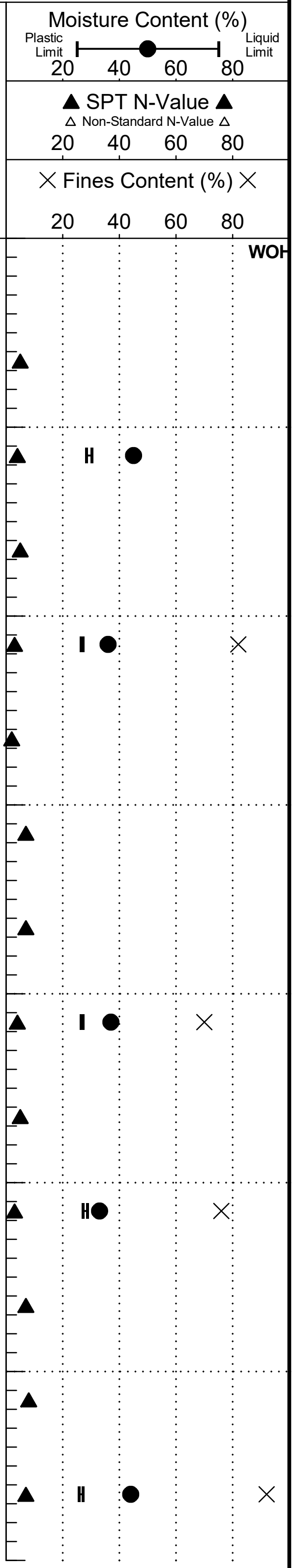
LAI Project No: 1374022.010

SAMPLE DATA

SOIL PROFILE

Drilling Method: Mud Rotary
Ground Elevation (ft): Not Measured
Drilled By: Holt
Logged By: BMD Date: 08/11/22

Depth (ft)	Sample Number & Interval	Sampler Type	Blows/Foot	Test Data	USCS Symbol	Description	Groundwater
35	S-14	b2	WOH		ML	Gray mottled brown sandy SILT (soft, moist) **(ALLUVIUM)**	
	S-15	b2	5				Groundwater Not Determined
40	S-16	b2	4	W = 45 AL		grades to soft brown stratified sandy SILT with fine sand layers (very loose to loose, wet)	
	S-17	b2	5				
45	S-18	b2	3	W = 36 -200 = 82 AL			
	S-19	b2	2				
50	S-20	b2	7			grades to medium stiff	
	S-21	b2	7				
55	S-22	b2	4	W = 37 GS AL			
	S-23	b2	5				
60	S-24	b2	3	W = 33 GS AL			
	S-25	b2	7				
65	S-26	b2	8				
	S-17	b2	7	W = 44 GS AL			
70							



Notes: 1. Stratigraphic contacts are based on field interpretations and are approximate.
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3. Refer to "Soil Classification System and Key" figure for explanation of graphics and symbols.

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McCall Terminal
Upgrade Project
Portland, Oregon

Log of Boring B-1

Figure

A2
(2 of 3)

B-1

LAI Project No: 1374022.010

SAMPLE DATA

SOIL PROFILE

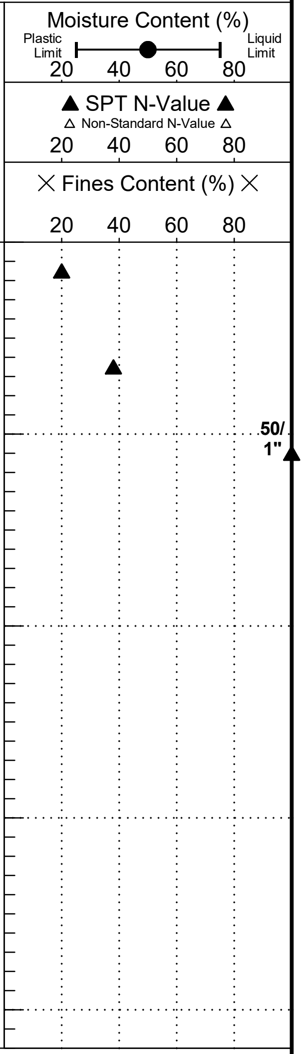
Drilling Method: Mud Rotary

Ground Elevation (ft): Not Measured

Drilled By: Holt

Logged By: BMD Date: 08/11/22

Depth (ft)	Sample Number & Interval	Sampler Type	Blows/Foot	Test Data	Graphic Symbol	USCS Symbol	Description	Groundwater
70	S-18	b2	20			ML	Gray mottled brown sandy SILT (soft, moist) **(ALLUVIUM)** grades to very dense, sharp increase in SPT blows	Groundwater Not Determined
	S-29	b2	38					
75	S-30	b2	50/ 1"				grades to hard	
						BST	Black BASALT; hard, fresh, massively bedded, fine grained with high angle joints and random fractures; CR=100%, RQD=52% (hard, damp) **(COLUMBIA FLOOD BASALT)**	
80							grades to blocky with minimal iron oxide staining along fractures decreases in blocky clast size with slight hydrochloric acid (HCl) reaction along calcarious infilled joints and fractures	
	S-31	f5					grades to massive texture with moderate HCl reaction along calcarious infilled joints 2-inch highly vesicular basalt inclusion	
85							fracture observed with moderate weathering and iron oxide staining	
						BST	Black BASALT; hard, fresh, massively bedded, vesicular; CR=100%, RQD=69% (hard, damp) grades to moderately vesicular with fine to medium sized vugs	
90							grades to highly vesicular with intermediate iron oxide staining and sulfuric mineralization grades to highly vesicular to scoriaceous, moderate to high iron oxide staining	
95								
100								
105								



Boring Completed 08/11/22
Total Depth of Boring = 91.0 ft.

Notes:
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3. Refer to "Soil Classification System and Key" figure for explanation of graphics and symbols.

McCall Terminal
Upgrade Project
Portland, Oregon

Log of Boring B-1

B-2

LAI Project No: 1374022.010

Drilling Method: Mud Rotary

Ground Elevation (ft): Not Measured

Drilled By: Holt

Logged By: BMD Date: 08/10/22

SAMPLE DATA

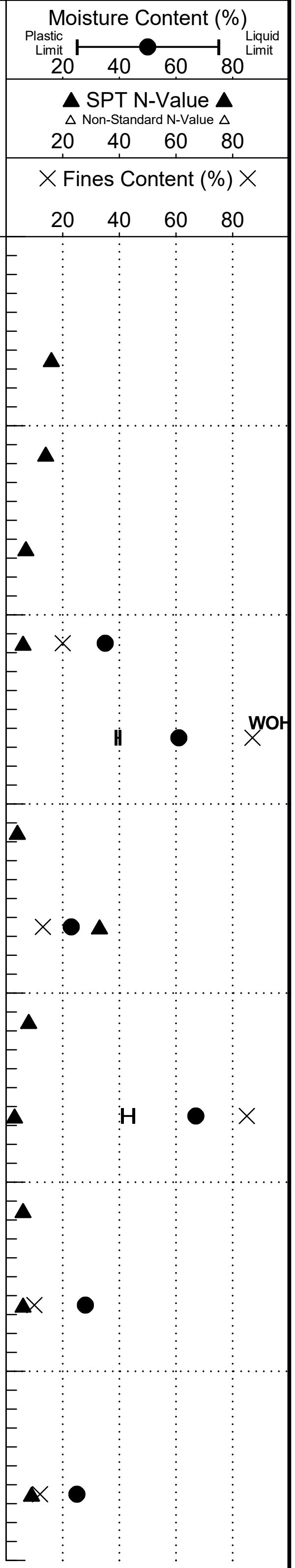
Sample Number & Interval	Sampler Type	Blows/Foot	Test Data
S-1	b2	16	
S-2	b2	14	
S-3	b2	7	
S-4	b2	6	W = 35 -200 = 20
S-5	b2	WOH	W = 61 GS AL
S-6	b2	4	
S-7	b2	33	W = 23 -200 = 13
S-8	b2	8	
S-9	b2	3	W = 67 GS AL
S-10	b2	6	
S-11	b2	6	W = 28 -200 = 10
S-12	c3		
S-13	b2	9	W = 25 -200 = 12

SOIL PROFILE

USCS Symbol	
SP	Brown fine SAND with silt (medium dense, moist) **(FILL)** grades to wet
ML	Gray SILT with sand (soft, moist to wet) **(ALLUVIUM)**
SM/ML	Brown very silty fine SAND with frequent 1 to 4-inch lenses of fine sandy SILT and scattered organic fibers (medium dense, moist to wet) lense of fine sandy SILT with trace organic fibers
ML	grades to fine sandy SILT difficult to progress shelby tube, likely sand contact

Groundwater: Groundwater not determined

Depth (ft): 0, 5, 10, 15, 20, 25, 30, 35



Notes: 1. Stratigraphic contacts are based on field interpretations and are approximate.
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McCall Terminal
Upgrade Project
Portland, Oregon

Log of Boring B-2

Figure
A3
(1 of 3)

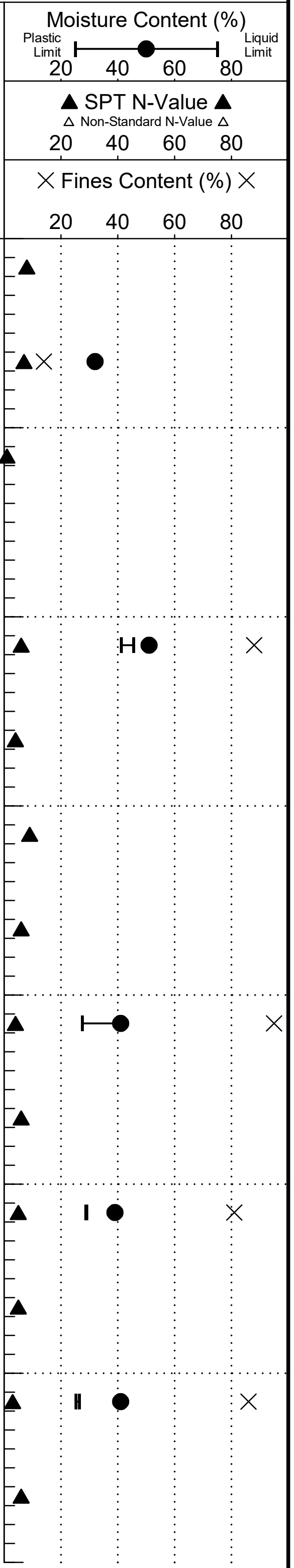
B-2

LAI Project No: 1374022.010

SAMPLE DATA / SOIL PROFILE

Drilling Method: Mud Rotary
Ground Elevation (ft): Not Measured
Drilled By: Holt
Logged By: BMD Date: 08/10/22

Depth (ft)	Sample Number & Interval	Sampler Type	Blows/Foot	Test Data	USCS Symbol	Description	Groundwater
35	S-14	b2	8		ML	grades to fine sandy SILT grades to fine silty SAND	Groundwater not determined
	S-15	b2	7	W = 32 GS			
40	S-16	b2	1			grades to SILT with fine sands (soft, moist)	
	S-17	c3					
45	S-18	b2	6	W = 51 -200 = 88 AL			
	S-19	b2	4				
50	S-20	b2	9				
	S-21	b2	6				
55	S-22	b2	4	W = 41 GS AL			
	S-23	b2	6			Brown/gray SILT (medium stiff, moist) increase in drill chatter towards bottom of sample	
60	S-24	b2	5	W = 39 GS AL			
	S-25	b2	5				
65	S-26	b2	3	W = 41 GS AL			
	S-27	b2	6				
70							



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

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McCall Terminal
Upgrade Project
Portland, Oregon

Log of Boring B-2

Figure
A3
(2 of 3)

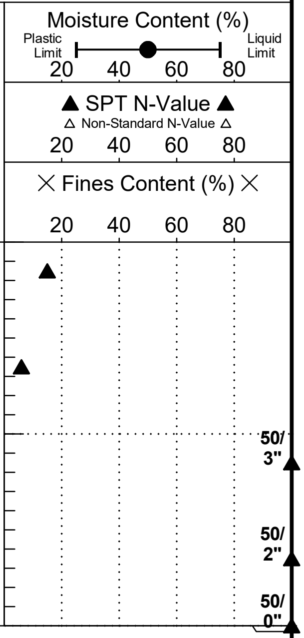
B-2

LAI Project No: 1374022.010

Drilling Method: Mud Rotary
Ground Elevation (ft): Not Measured
Drilled By: Holt
Logged By: BMD Date: 08/10/22

Depth (ft)	Sample Number & Interval	Sampler Type	Blows/Foot	Test Data	USCS Symbol	SOIL PROFILE	Groundwater
70	S-28	b2	15		ML	grades to fine sandy SILT	Groundwater not determined
	S-29	b2	6				
75	S-30	b2	50/ 3"				
					BST	Black BASALT; hard, fresh, fine grained and vesicular stuck in shoe of sampler. (hard, damp) **(COLUMBIA FLOOD BASALT)**	
	S-31	b2	50/ 2"				
80	S-2	b2	50/ 0"				

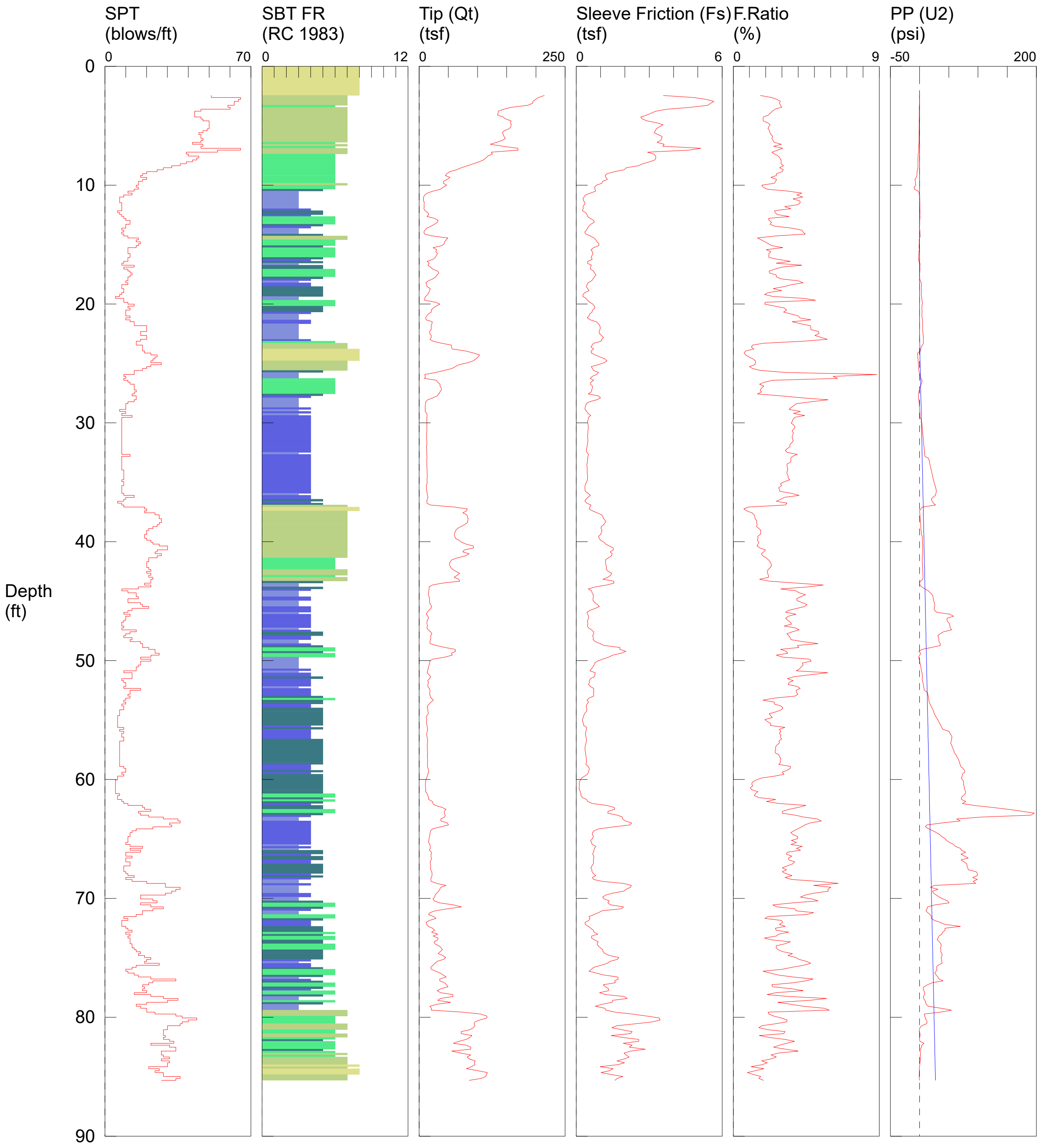
Boring Completed 08/10/22
Total Depth of Boring = 80.0 ft.



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

Landau / CPT-1 / 5480 NW Front Ave Portland

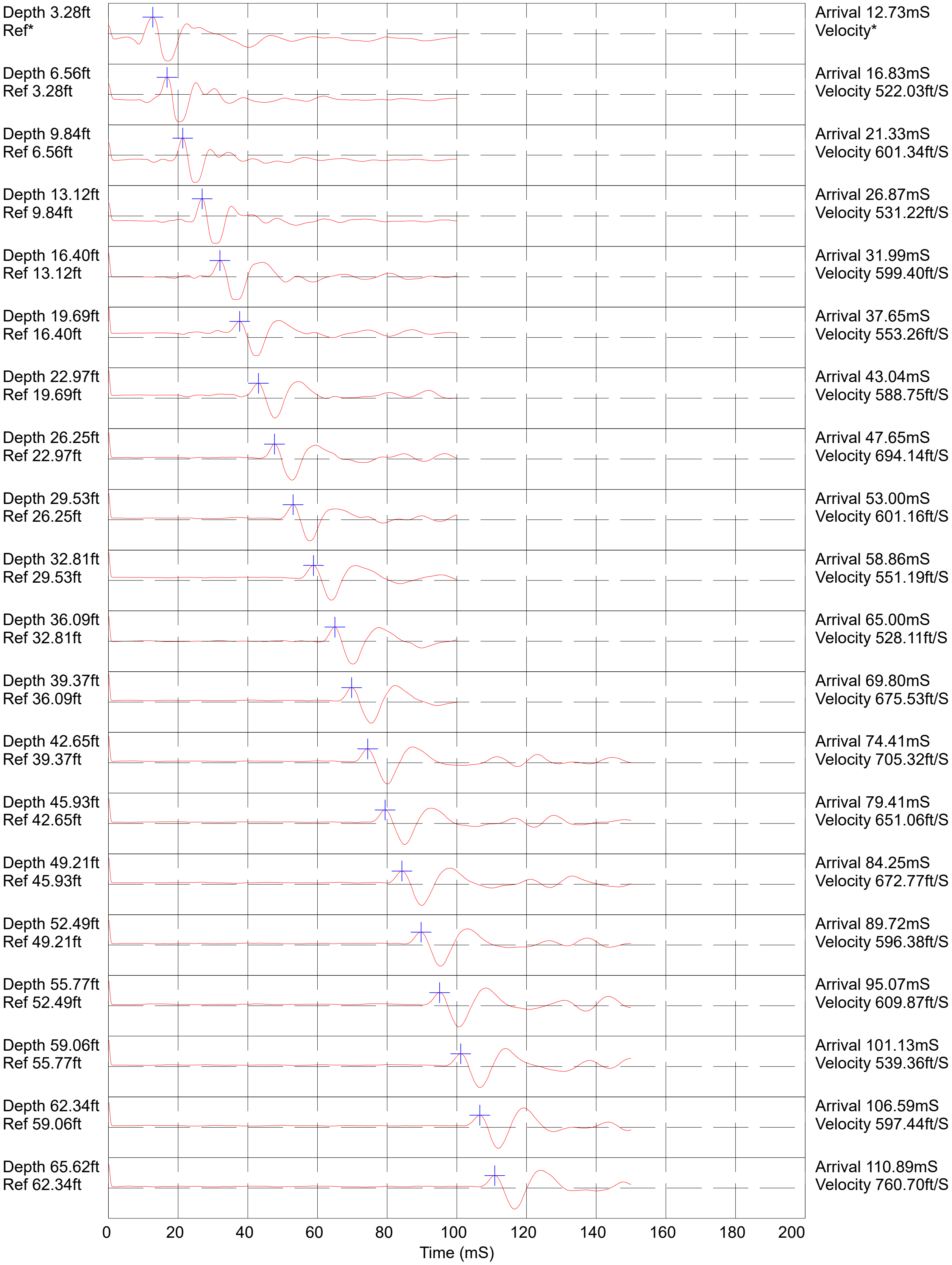
OPERATOR: OGE BAK
CONE ID: DDG1532
HOLE NUMBER: CPT-1
TEST DATE: 8/10/2022 1:53:16 PM
TOTAL DEPTH: 85.302 ft



- 🟥 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 (*)

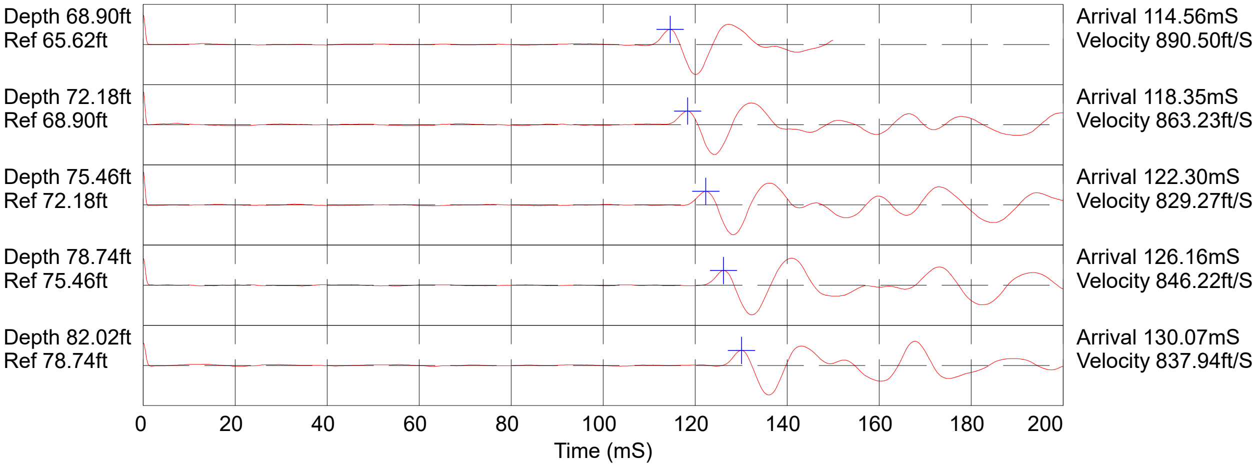
*SBT/SPT CORRELATION: UBC-1983

COMMENT: Landau / CPT-4 / 5480 NW Front Ave Portland



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

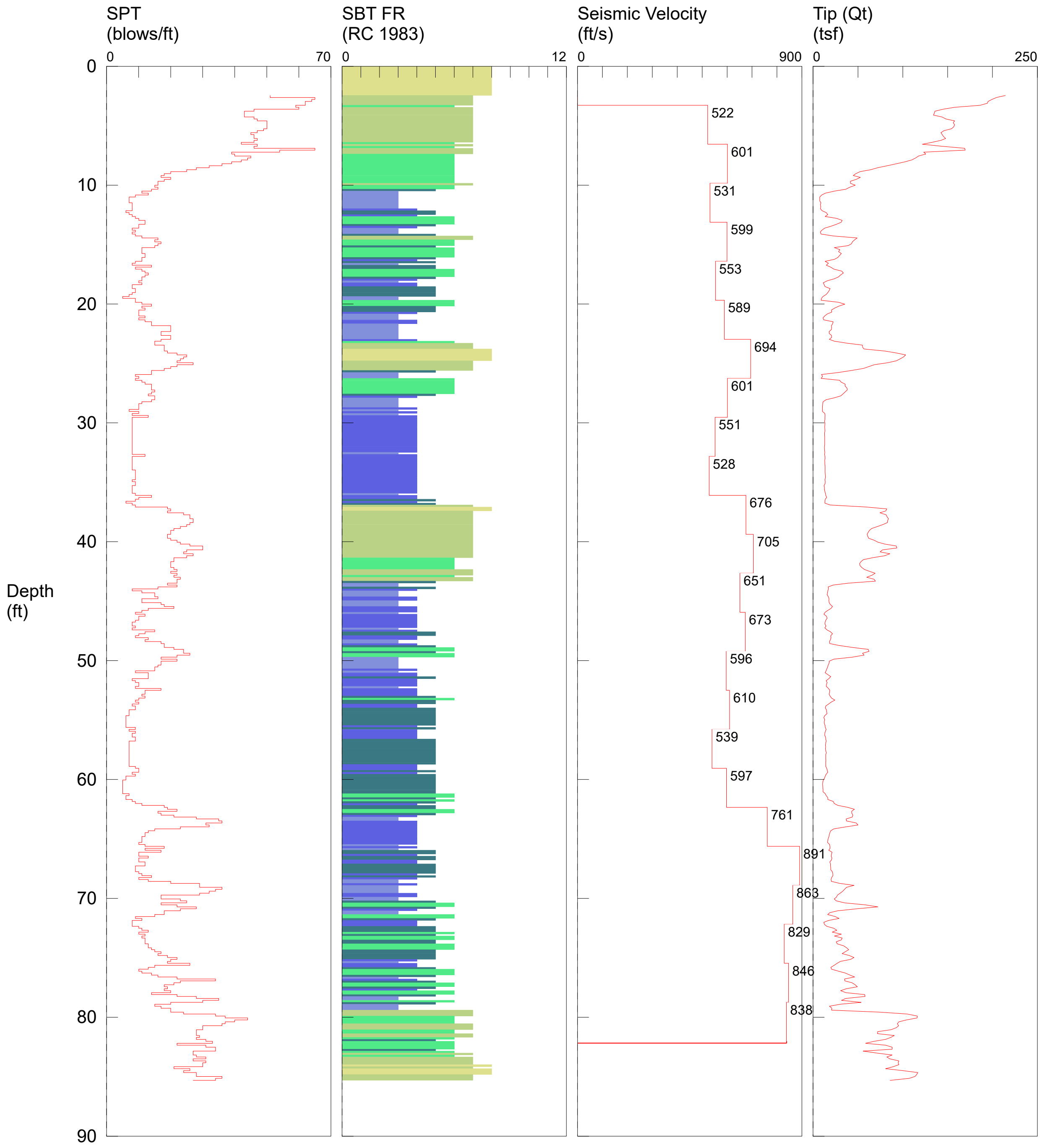
COMMENT: Landau / CPT-4 / 5480 NW Front Ave Portland



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

Landau / CPT-1 / 5480 NW Front Ave Portland

OPERATOR: OGE BAK
CONE ID: DDG1532
HOLE NUMBER: CPT-1
TEST DATE: 8/10/2022 1:53:16 PM
TOTAL DEPTH: 85.302 ft

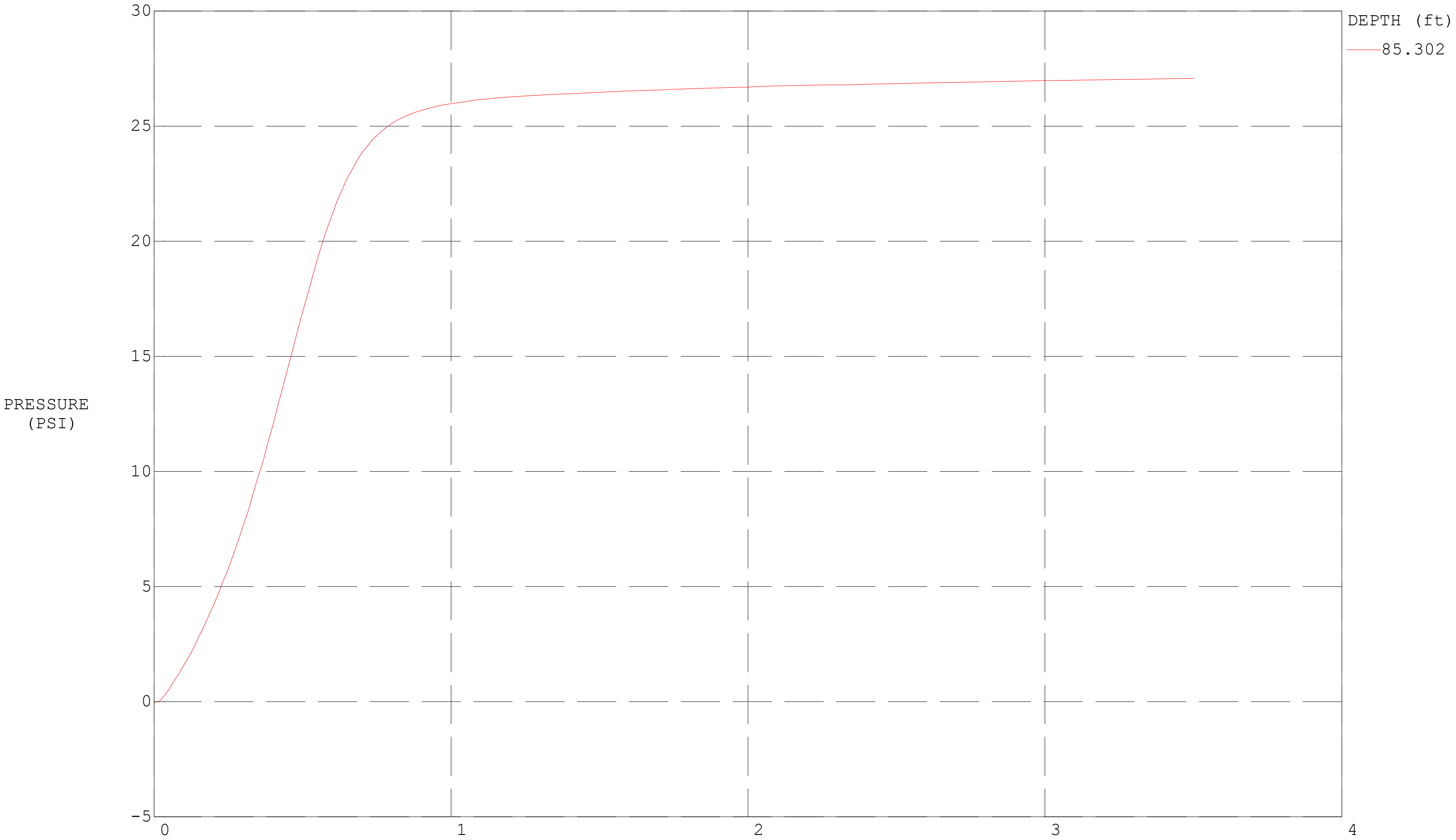


🟥 1 sensitive fine grained
🟥 2 organic material
🟦 3 clay
🟦 4 silty clay to clay
🟩 5 clayey silt to silty clay
🟩 6 sandy silt to clayey silt
🟩 7 silty sand to sandy silt
🟨 8 sand to silty sand
🟫 9 sand
🟧 10 gravelly sand to sand
⬜ 11 very stiff fine grained (*)
⬛ 12 sand to clayey sand (*)

*SBT/SPT CORRELATION: UBC-1983

COMMENT: Landau / CPT-1 / 5480 NW Front Ave Portland

TEST DATE: 8/10/2022 1:53:16 PM

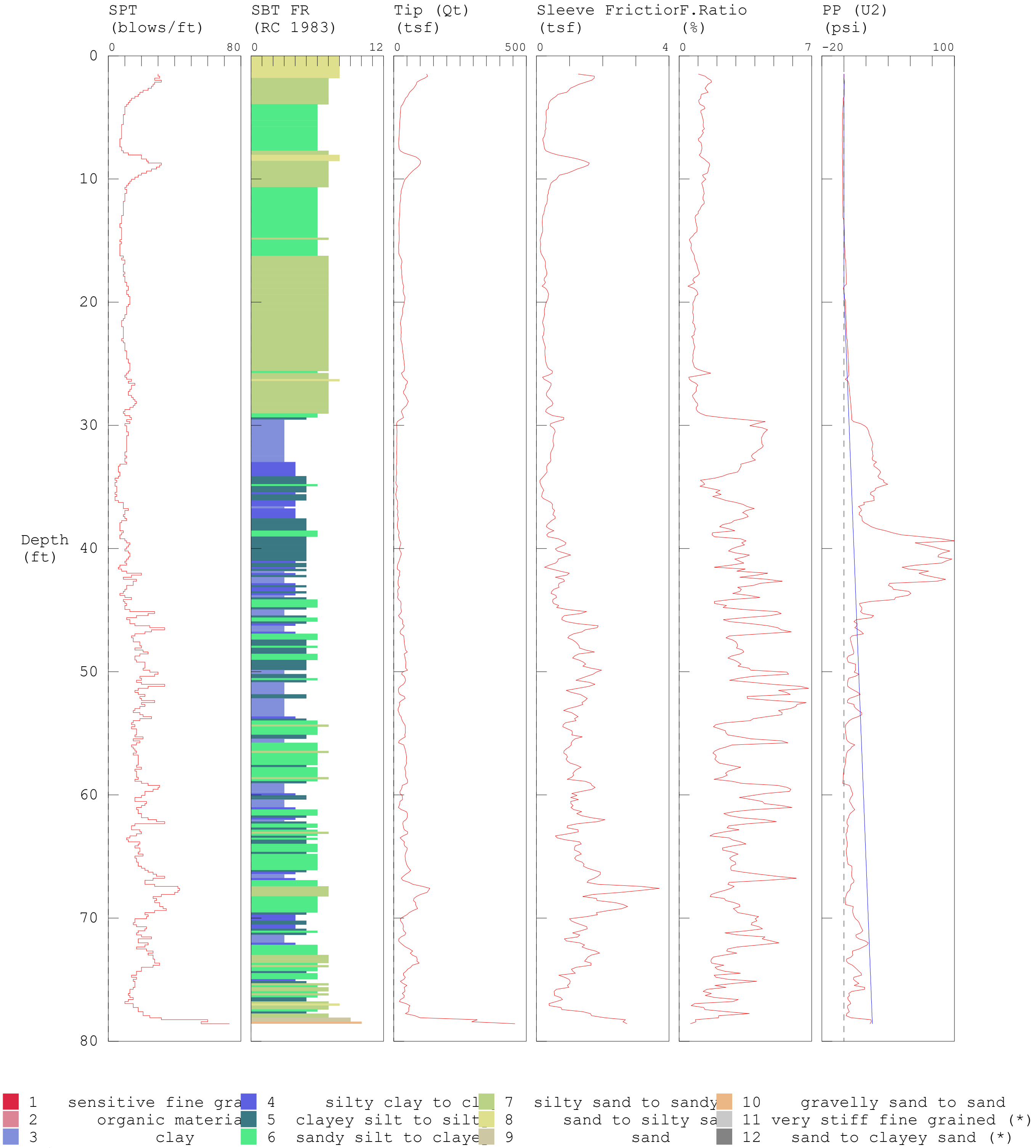


MAXIMUM PRESSURE = 27.072 (PSI)

HYDROSTATIC PRESSURE = 27.101 (PSI), WATER TABLE: 22.77 ft

Landau / CPT-2 / 5480 NW Front Ave Portland

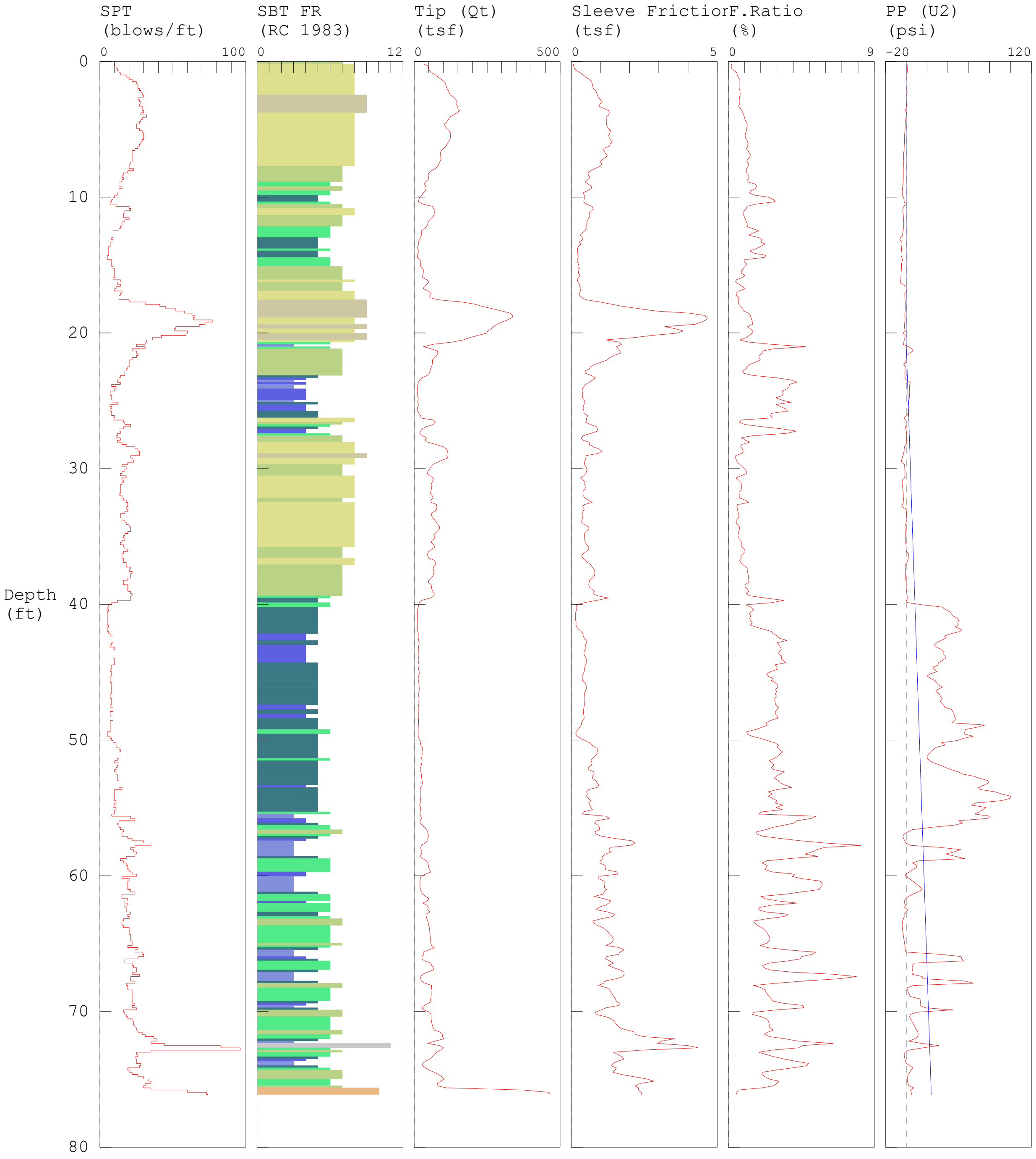
OPERATOR: OGE BAK
CONE ID: DDG1532
HOLE NUMBER: CPT-2
TEST DATE: 8/11/2022 11:28:20 AM
TOTAL DEPTH: 78.576 ft



*SBT/SPT CORRELATION: UBC-1983

Landau / CPT-3 / 5480 NW Front Ave Portland

OPERATOR: OGE BAK
CONE ID: DDG1532
HOLE NUMBER: CPT-3
TEST DATE: 8/11/2022 8:47:20 AM
TOTAL DEPTH: 76.115 ft

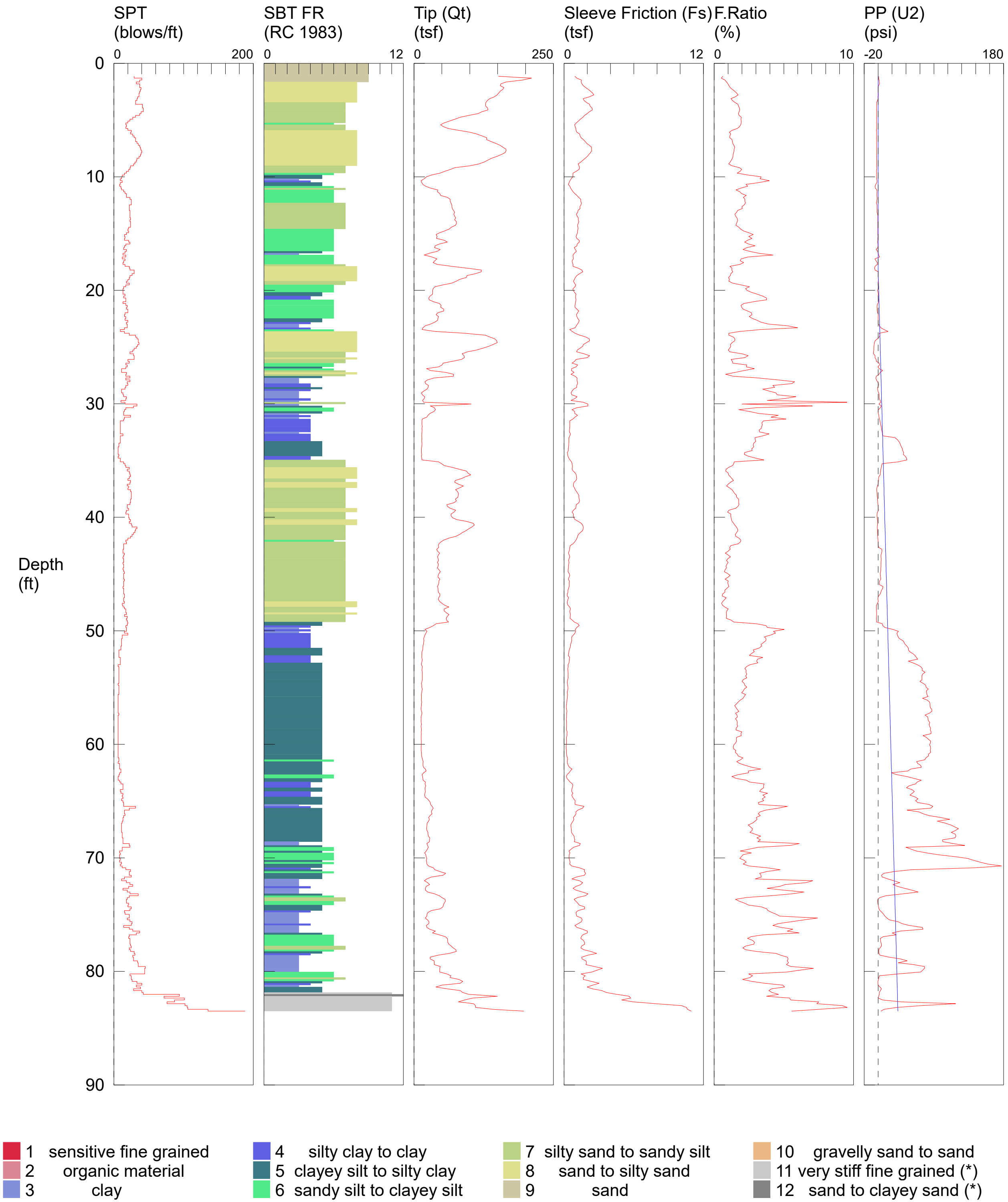


🟥 1 sensitive fine gra
🟥 2 organic materia
🟦 3 clay
🟦 4 silty clay to cl
🟩 5 clayey silt to silt
🟩 6 sandy silt to claye
🟩 7 silty sand to sandy
🟨 8 sand to silty sa
🟫 9 sand
🟧 10 gravelly sand to sand
⬜ 11 very stiff fine grained (*)
⬛ 12 sand to clayey sand (*)

*SBT/SPT CORRELATION: UBC-1983

Landau / CPT-4 / 5480 NW Front Ave Portland

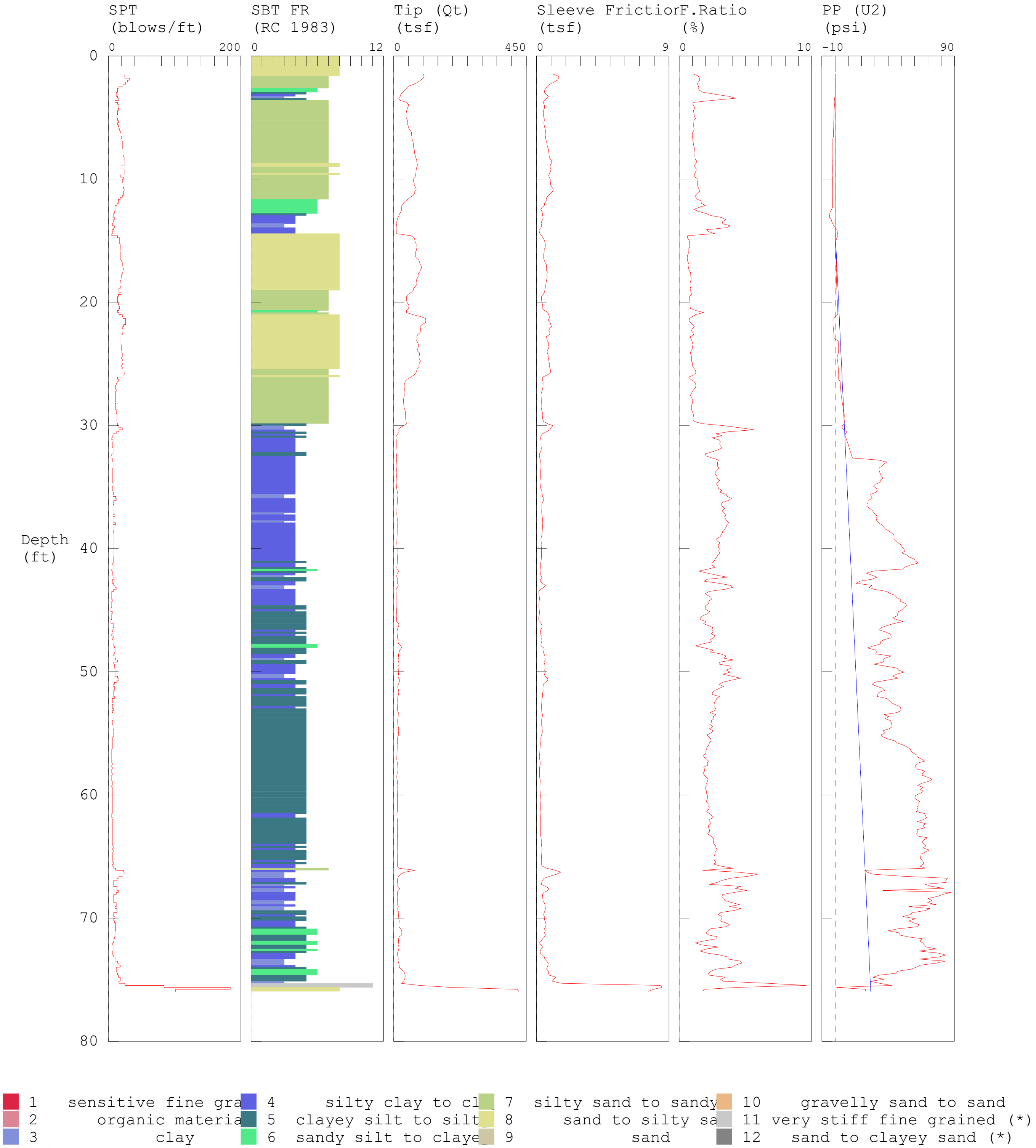
OPERATOR: OGE BAK
CONE ID: DDG1532
HOLE NUMBER: CPT-4
TEST DATE: 8/10/2022 9:00:39 AM
TOTAL DEPTH: 83.497 ft



*SBT/SPT CORRELATION: UBC-1983

Landau / CPT-5 / 5480 NW Front Ave Portland

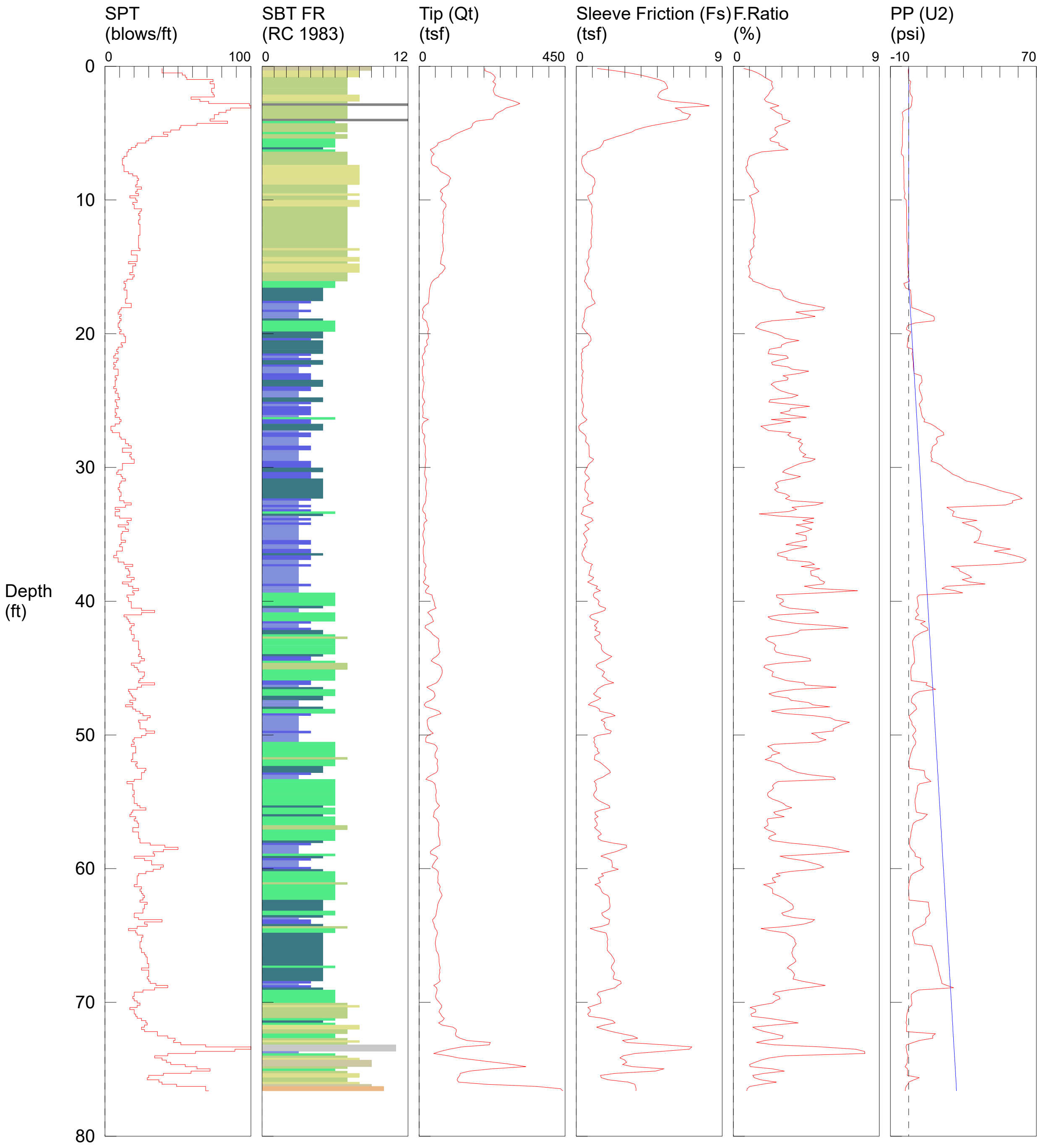
OPERATOR: OGE BAK
CONE ID: DDG1532
HOLE NUMBER: CPT-5
TEST DATE: 8/11/2022 10:10:41 AM
TOTAL DEPTH: 75.951 ft



*SBT/SPT CORRELATION: UBC-1983

Landau / CPT-6 / 5480 NW Front Ave Portland

OPERATOR: OGE BAK
CONE ID: DDG1532
HOLE NUMBER: CPT-6
TEST DATE: 8/10/2022 11:42:05 AM
TOTAL DEPTH: 76.608 ft



🟥 1 sensitive fine grained
🟥 2 organic material
🟦 3 clay

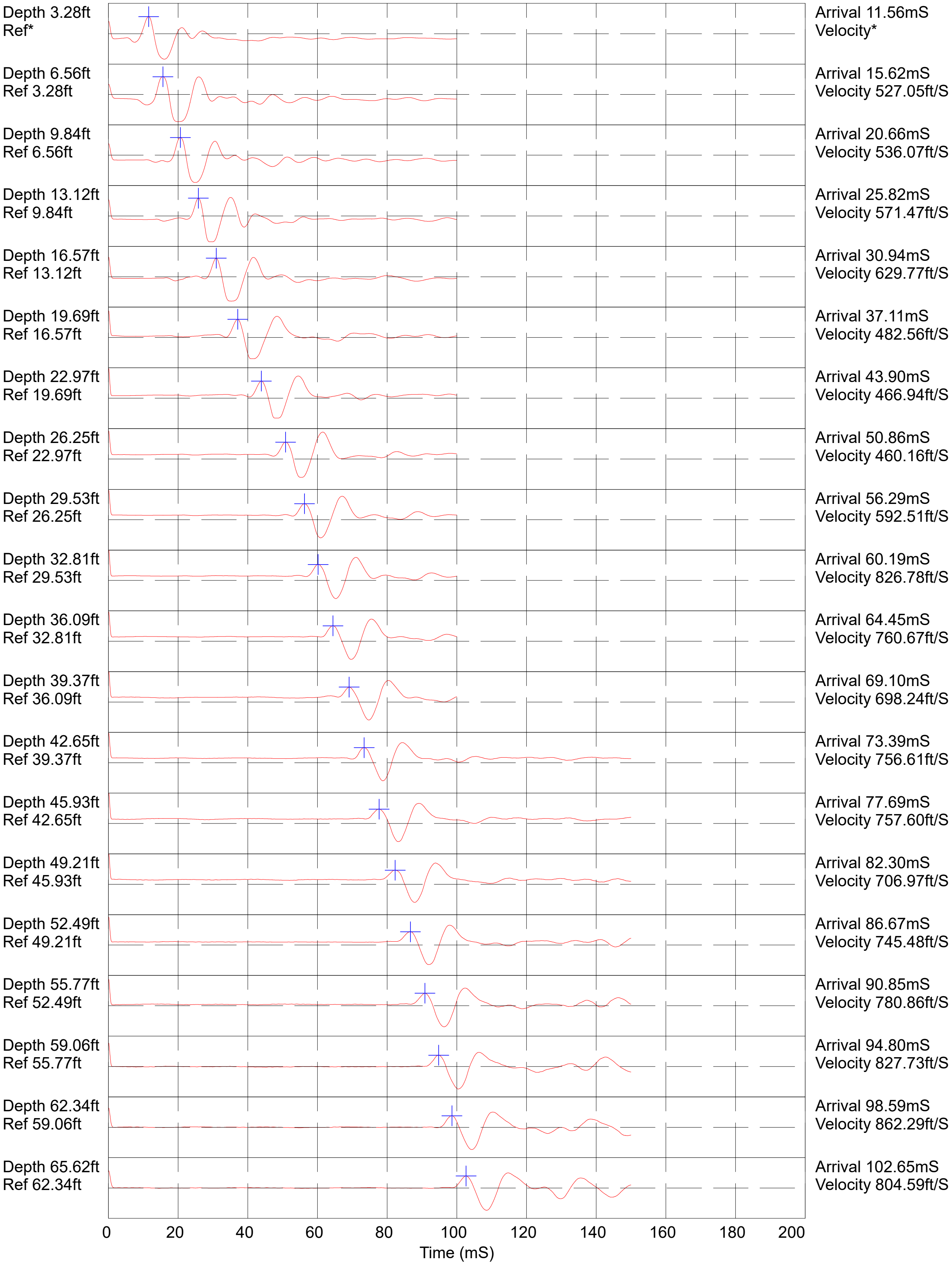
🟦 4 silty clay to clay
🟩 5 clayey silt to silty clay
🟩 6 sandy silt to clayey silt

🟩 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 (*)

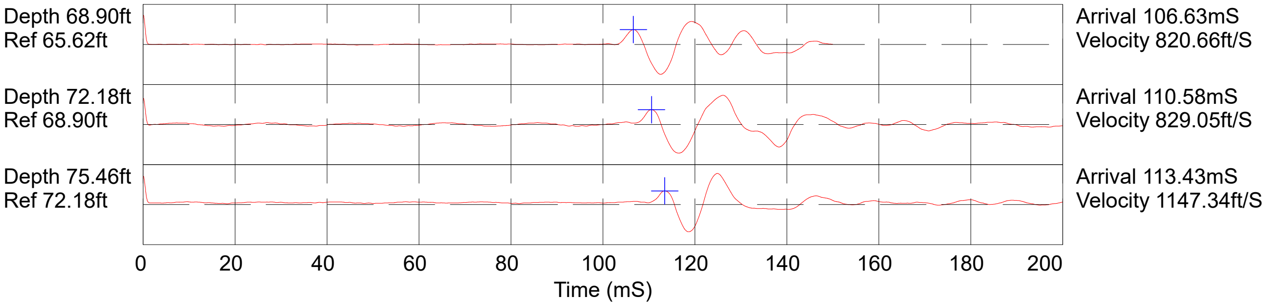
*SBT/SPT CORRELATION: UBC-1983

COMMENT: Landau / CPT-6 / 5480 NW Front Ave Portland



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

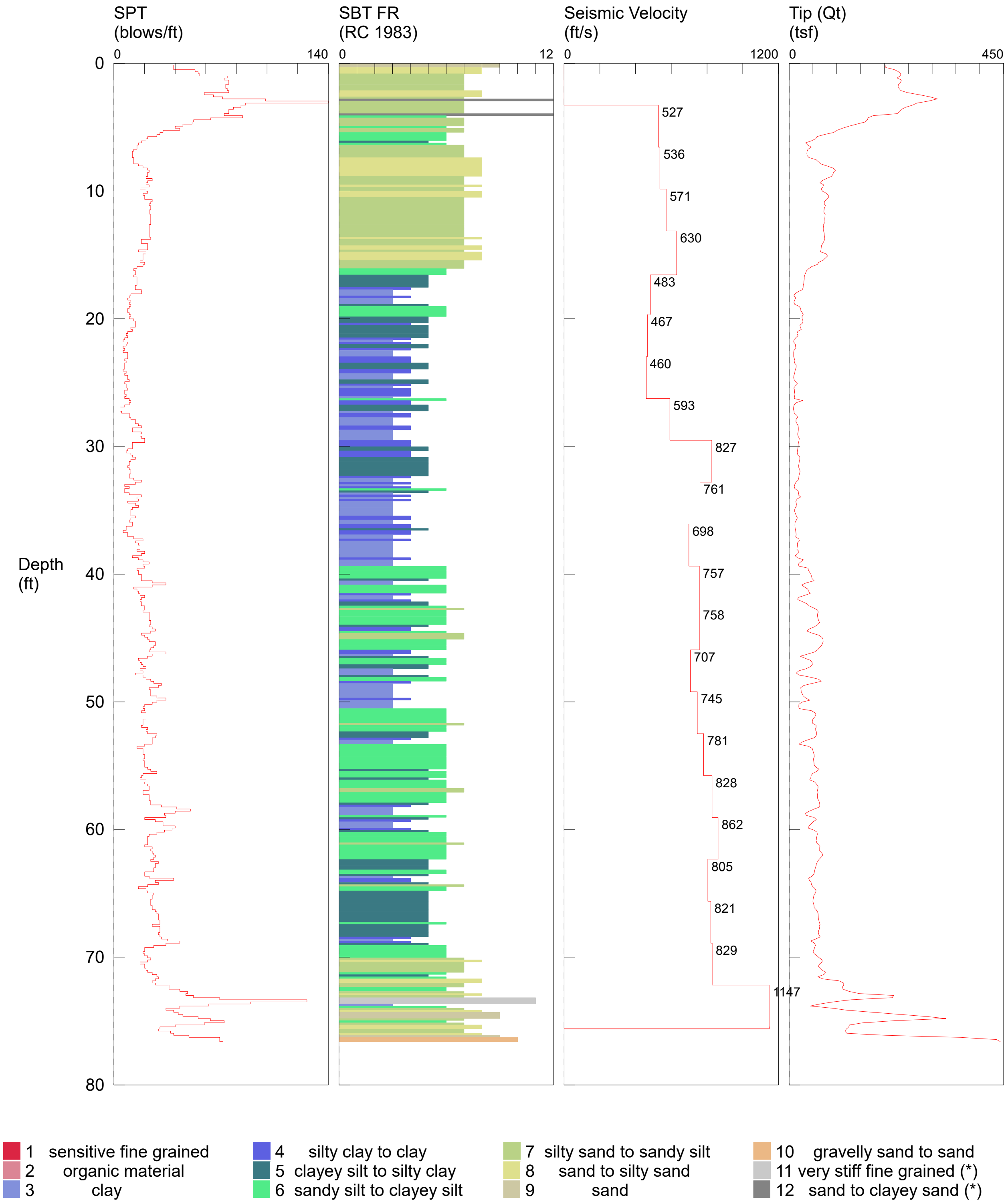
COMMENT: Landau / CPT-6 / 5480 NW Front Ave Portland



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

Landau / CPT-6 / 5480 NW Front Ave Portland

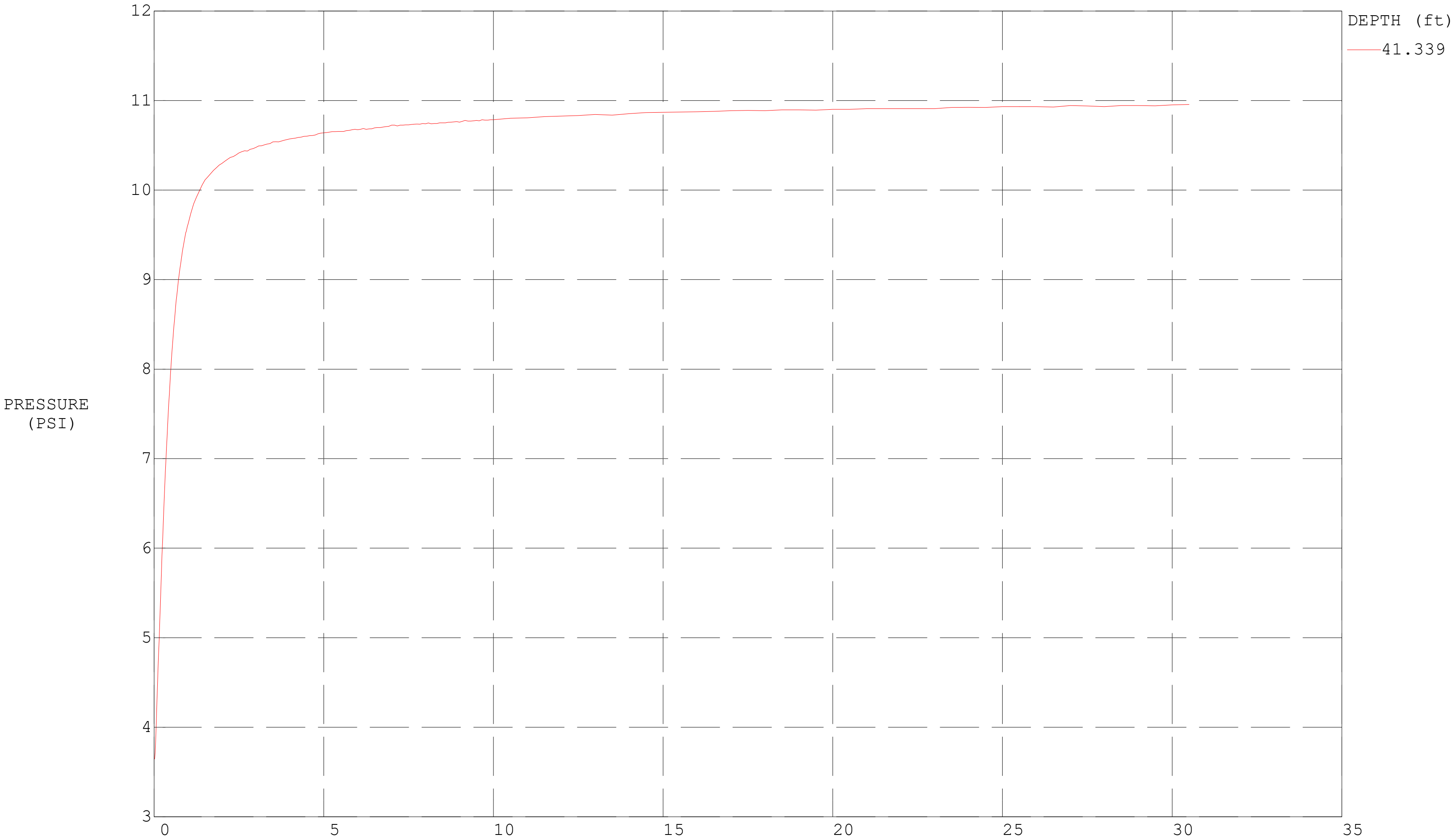
OPERATOR: OGE BAK
CONE ID: DDG1532
HOLE NUMBER: CPT-6
TEST DATE: 8/10/2022 11:42:05 AM
TOTAL DEPTH: 76.608 ft



*SBT/SPT CORRELATION: UBC-1983

COMMENT: Landau / CPT-6 / 5480 NW Front Ave Portland

TEST DATE: 8/10/2022 11:42:05 AM



MAXIMUM PRESSURE = 10.957 (PSI) TIME: (MINUTES)

HYDROSTATIC PRESSURE = 10.968 (PSI), WATER TABLE: 16.03 ft

APPENDIX B

Laboratory Soil Testing

APPENDIX B
LABORATORY SOIL TESTING

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

Natural Moisture Content

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

Grain Size Analysis

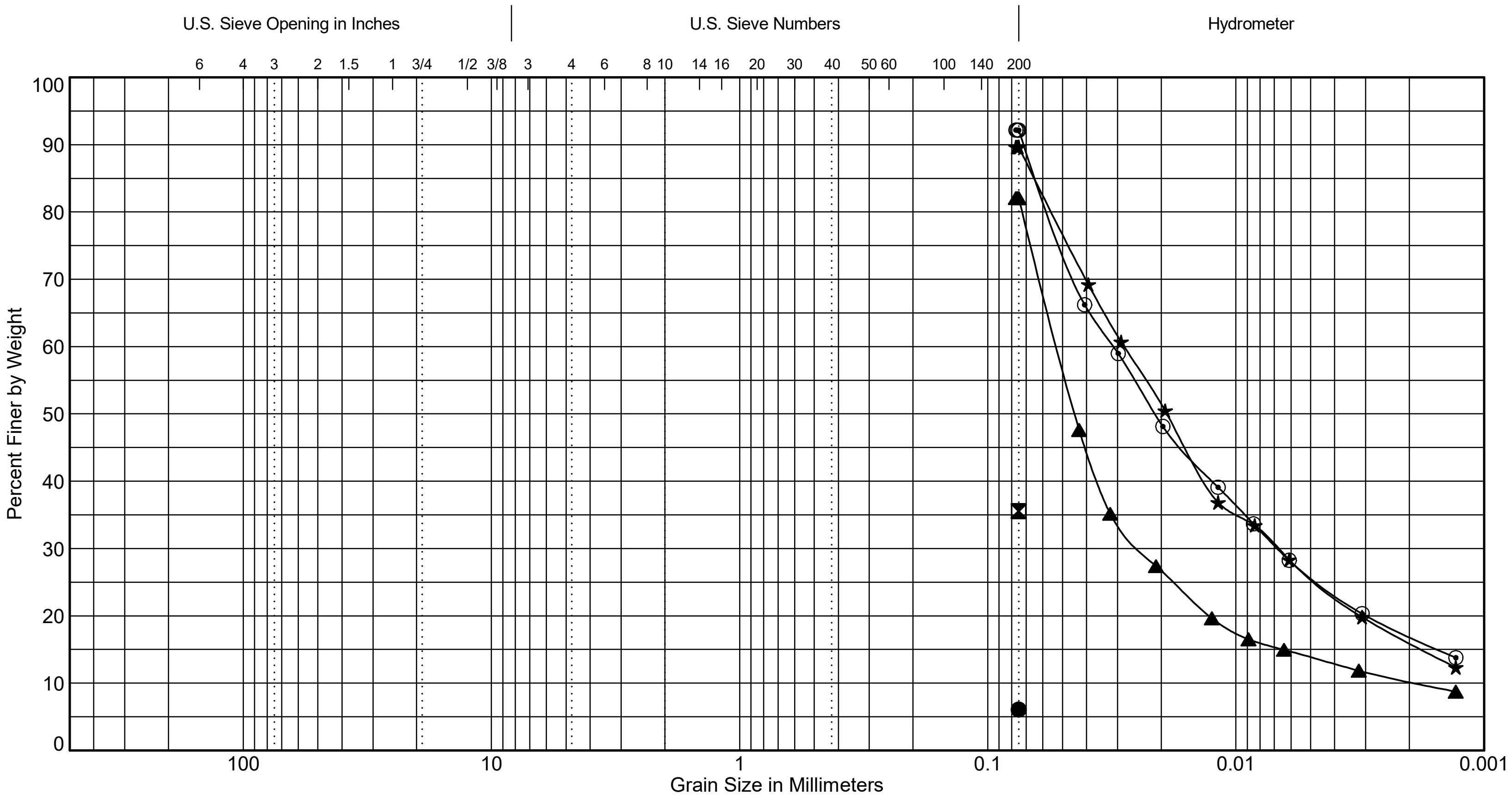
To provide an indication of the grain size distribution of site soils, grain size analyses were performed in accordance with ASTM test method D422. Samples selected for grain size analysis are designated with a "GS" in the column labeled "Test Data" on the summary boring logs in Appendix A. The results of the grain size analyses are presented in the form of grain size distribution curves on Figures B1 through B3.

Atterberg Limit Determination

To assess the plasticity of fine-grained site soils, Atterberg limit tests were performed in general accordance with ASTM test method D4318. Samples selected for Atterberg limit tests are designated with an "AL" in the column labeled "Test Data" on the summary boring logs in Appendix A. The results of the Atterberg limit tests are presented in graphical and tabular form on Figures B4 and B5.

Core Recovery and Rock Quality Designation

Fifteen feet (ft) of basalt core was collected for the purpose of determining the rock quality designation (RDQ) as well as the total core recovery (CR) as a standard parameter in general accordance with ASTM method D6032/D6032M-17. RQD and CR values are located on the summary boring logs in Appendix A.



Cobbles	Gravel		Sand			Silt or Clay
	Coarse	Fine	Coarse	Medium	Fine	

Symbol	Exploration Number	Sample Number	Depth (ft)	Natural Moisture (%)	Soil Description	Unified Soil Classification
●	B-1	S-5	12.5	28		
⧗	B-1	S-6	15.0	55		
▲	B-1	S-9	22.5	59	Sandy SILT	ML
★	B-1	S-11	27.5	28	SILT with sand	ML
⊙	B-1	S-13	32.5	42	SILT with sand	ML

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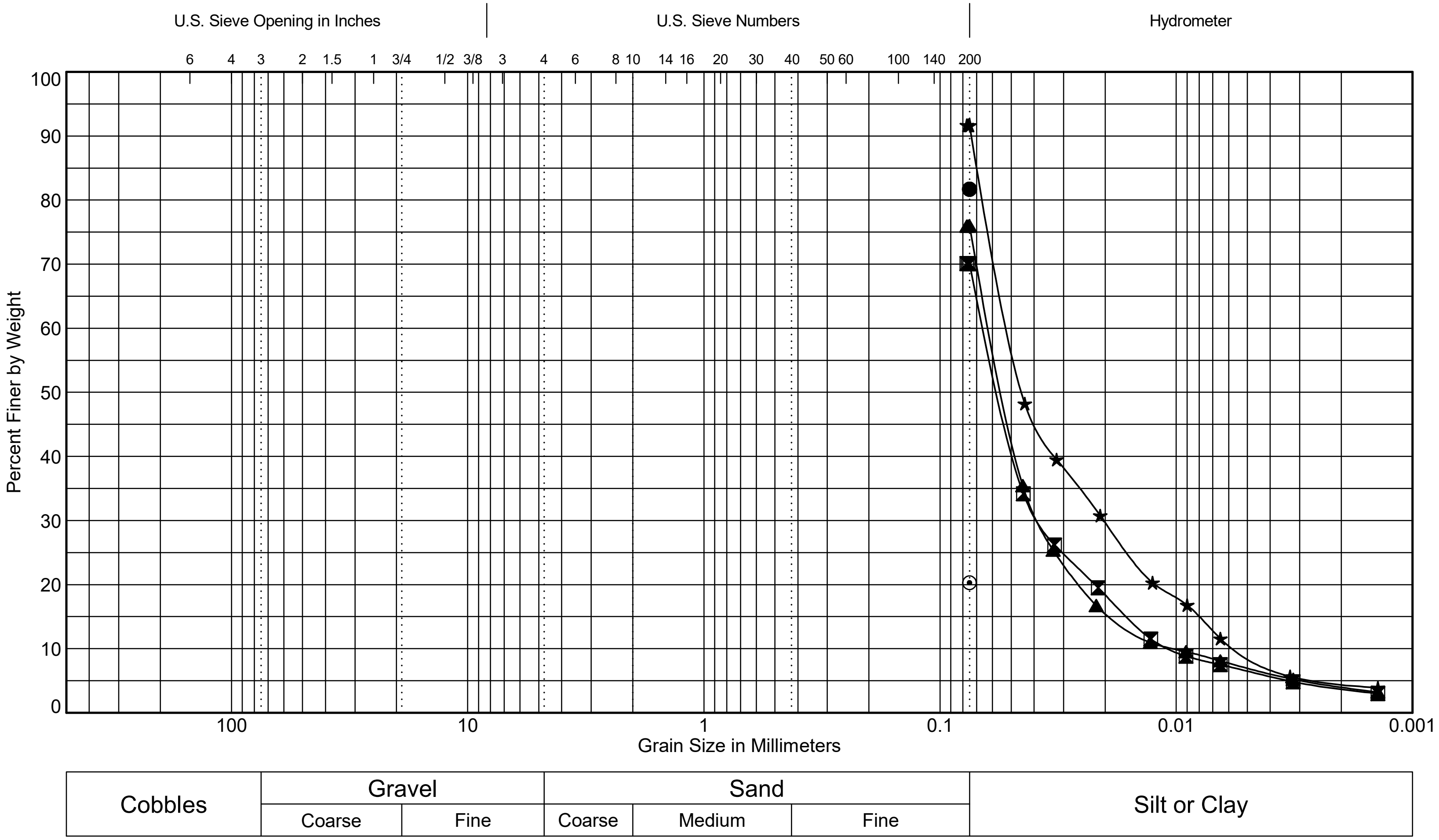
Portland, Oregon

Grain Size Distribution

Figure
B1



Symbol	Exploration Number	Sample Number	Depth (ft)	Natural Moisture (%)	Soil Description	Unified Soil Classification
●	B-1	S-18	45.0	36	Sandy SILT	ML
⧗	B-1	S-22	55.0	37	Sandy SILT	ML
▲	B-1	S-24	60.0	33	Sandy SILT	ML
★	B-1	S-27	67.5	44	SILT with sand	ML
⊙	B-2	S-4	10.0	35		

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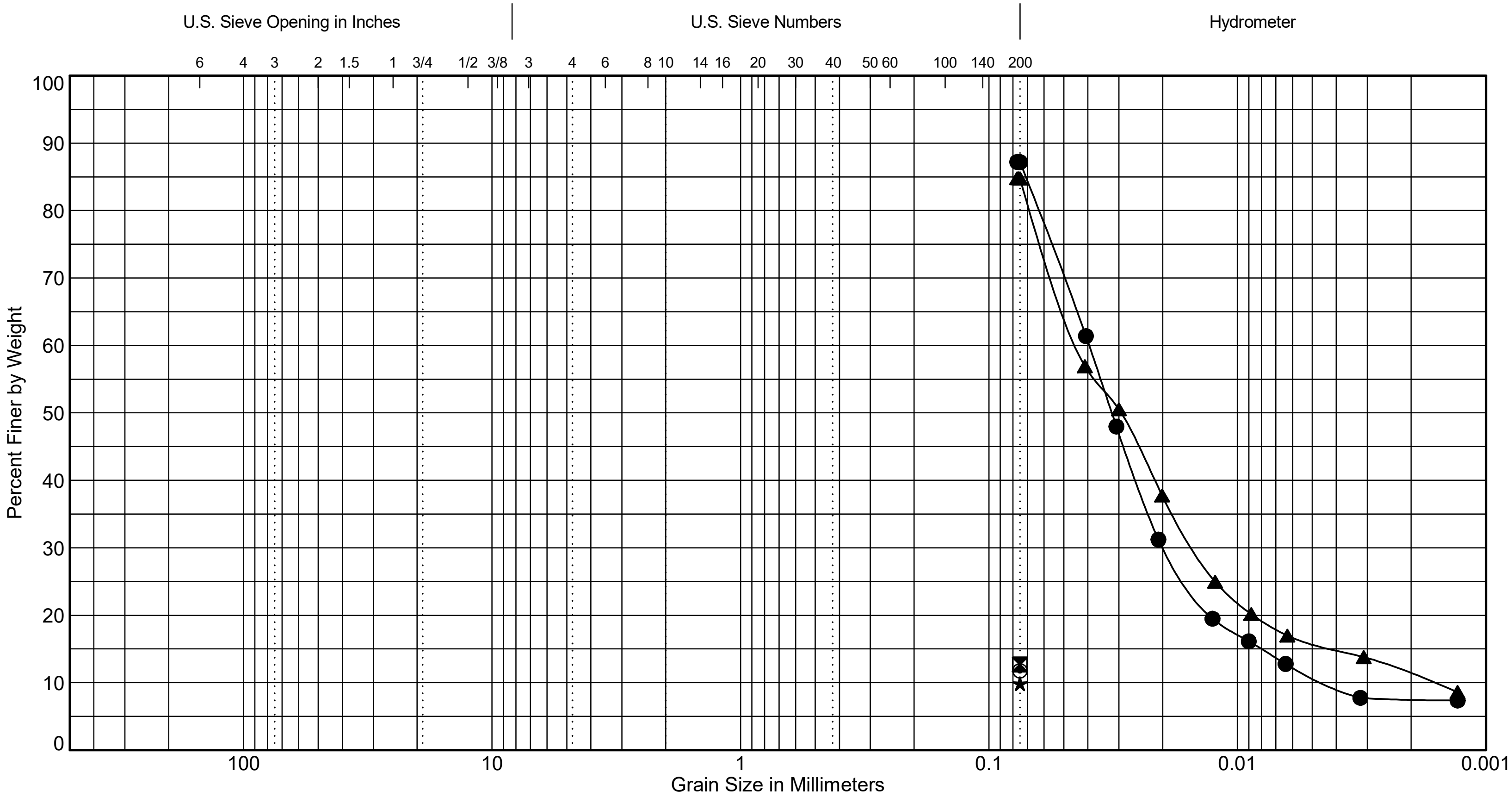
Portland, Oregon

Grain Size Distribution

Figure
B2



<table>
<tr><td rowspan="2">Cobbles</td><td colspan="2">Gravel</td><td colspan="3">Sand</td><td rowspan="2">Silt or Clay</td></tr>
<tr><td>Coarse</td><td>Fine</td><td>Coarse</td><td>Medium</td><td>Fine</td></tr>
</table>

Symbol	Exploration Number	Sample Number	Depth (ft)	Natural Moisture (%)	Soil Description	Unified Soil Classification
●	B-2	S-5	12.5	61	SILT with sand	ML
⧗	B-2	S-7	17.5	23		
▲	B-2	S-9	22.5	67	Sandy SILT	ML
★	B-2	S-11	27.5	28		
⦿	B-2	S-13	32.5	25		

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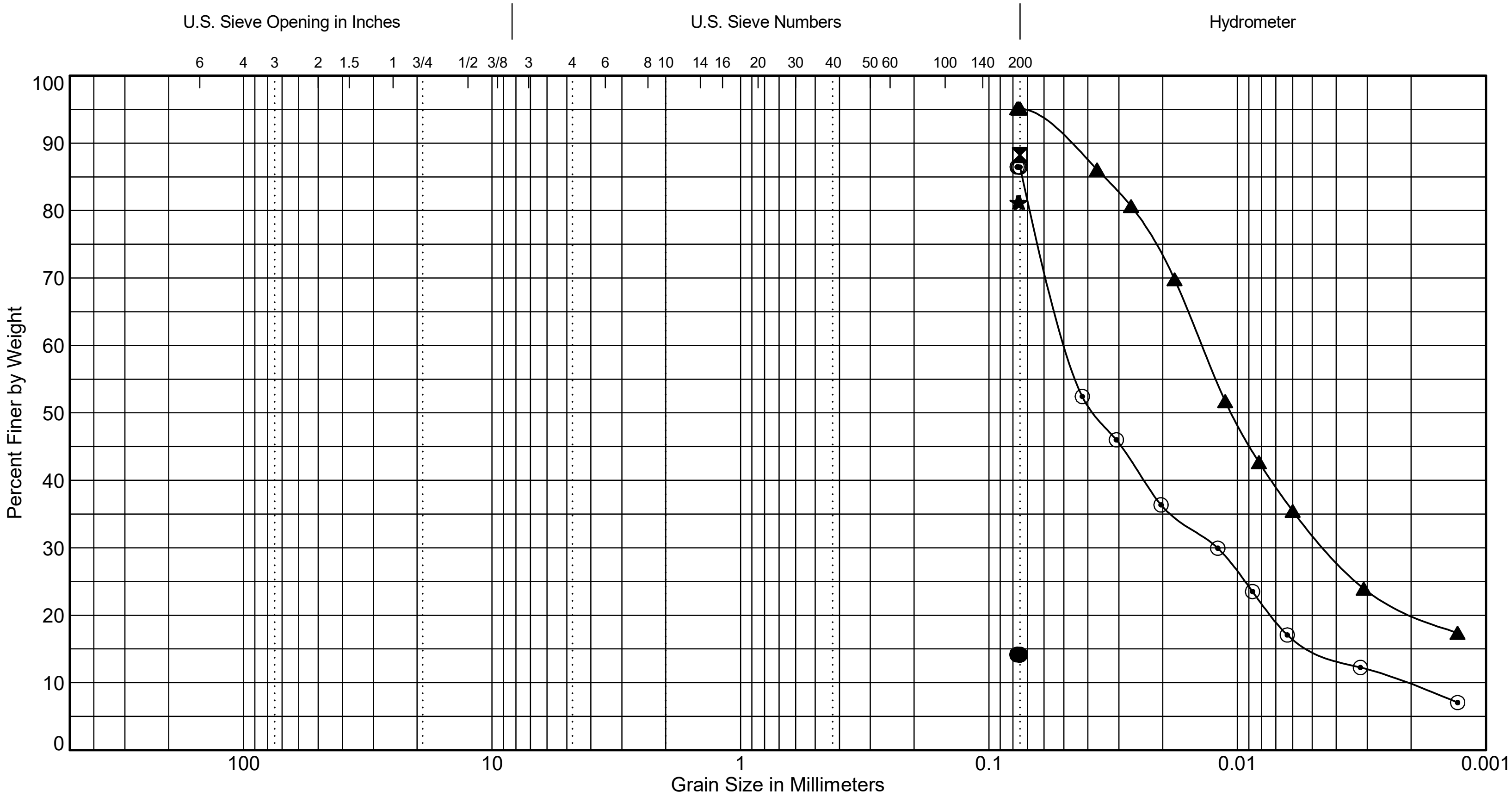
Upgrade Project
Portland, Oregon

Grain Size Distribution

Figure
B3



Cobbles	Gravel		Sand			Silt or Clay
	Coarse	Fine	Coarse	Medium	Fine	

Symbol	Exploration Number	Sample Number	Depth (ft)	Natural Moisture (%)	Soil Description	Unified Soil Classification
●	B-2	S-15	37.5	32		
⧗	B-2	S-18	45.0	51	SILT with sand	ML
▲	B-2	S-22	55.0	41	SILT	ML
★	B-2	S-24	60.0	39	Sandy SILT	ML
⊙	B-2	S-26	65.0	41	SILT with sand	ML

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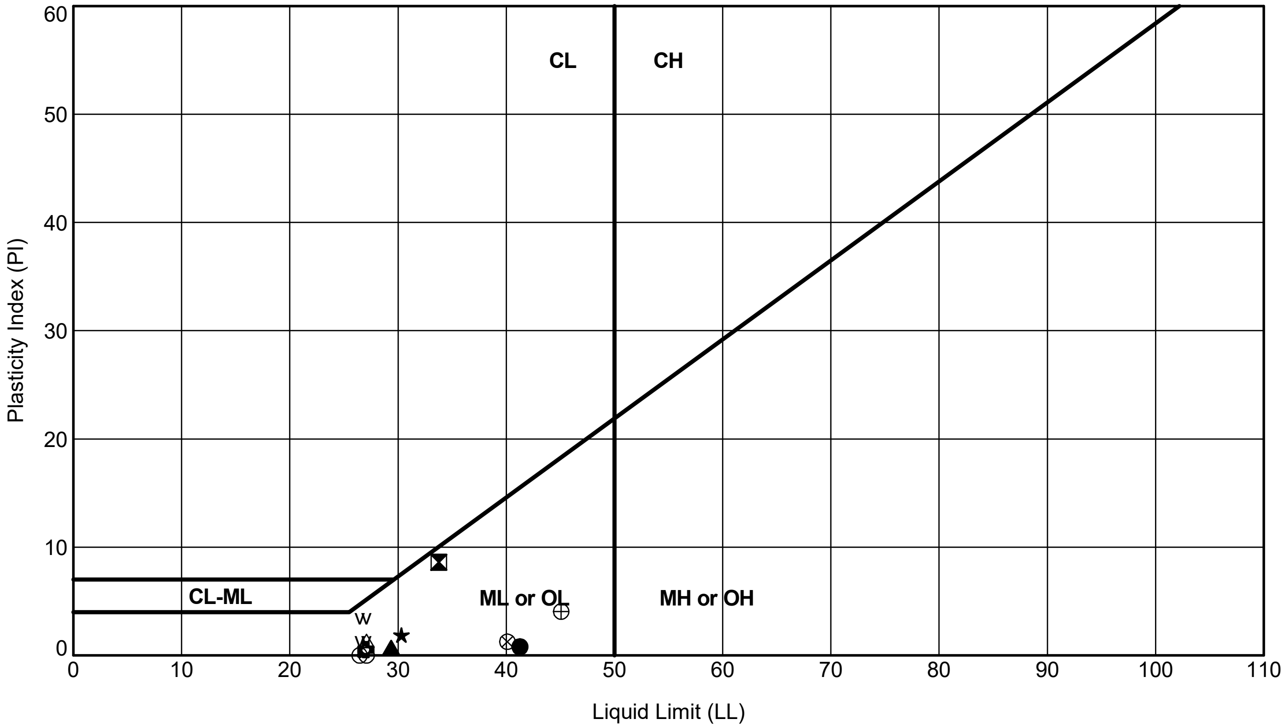
Portland, Oregon

Grain Size Distribution

Figure
B4



ATTERBERG LIMIT TEST RESULTS

Symbol	Exploration Number	Sample Number	Depth (ft)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Natural Moisture (%)	Soil Description	Unified Soil Classification
●	B-1	S-9	22.5	41	40	1	59	Sandy SILT	ML
⧗	B-1	S-11	27.5	34	25	9	28	SILT with sand	ML
▲	B-1	S-13	32.5	29	29	NP	42	SILT with sand	ML
★	B-1	S-16	40.0	30	28	2	45	SILT with sand	ML
⊙	B-1	S-18	45.0	26	27	NP	36	Sandy SILT	ML
✿	B-1	S-22	55.0	27	26	1	37	Sandy SILT	ML
○	B-1	S-24	60.0	27	29	NP	33	Sandy SILT	ML
△	B-1	S-27	67.5	27	26	1	44	SILT with sand	ML
⊗	B-2	S-5	12.5	40	39	1	61	SILT with sand	ML
⊕	B-2	S-9	22.5	45	41	4	67	Sandy SILT	ML

ASTM D 4318 Test Method

1374022.01 1/12/23 Y:\1374\022.010\T\GINT\1374022.010.GPJ ATTERBERG LIMITS FIGURE

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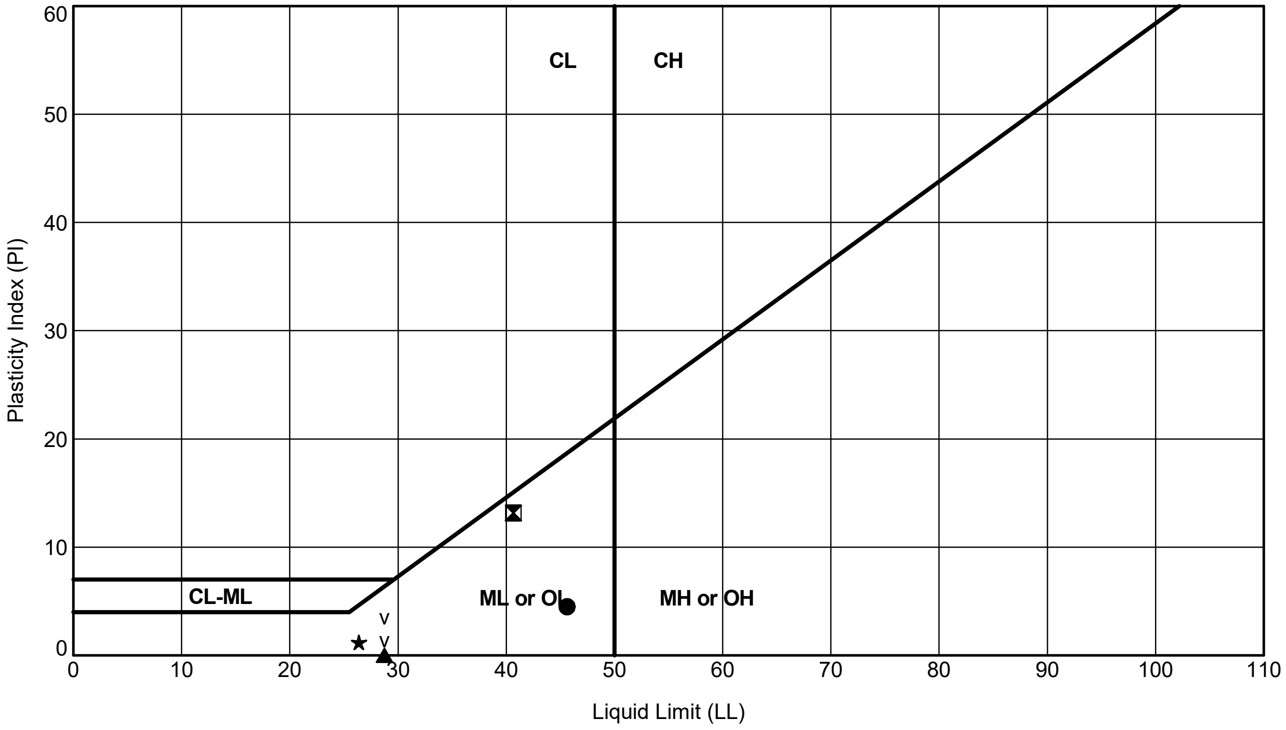
Plasticity Chart

Figure
B5



ATTERBERG LIMIT TEST RESULTS

Symbol	Exploration Number	Sample Number	Depth (ft)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Natural Moisture (%)	Soil Description	Unified Soil Classification
●	B-2	S-18	45.0	46	41	5	51	SILT with sand	ML
⧗	B-2	S-22	55.0	41	28	13	41	SILT	ML
▲	B-2	S-24	60.0	29	29	NP	39	Sandy SILT	ML
★	B-2	S-26	65.0	26	25	1	41	SILT with sand	ML

ASTM D 4318 Test Method

1374022.01 1/12/23 Y:\1374\022.010\T\GINT\1374022.010.GPJ ATTERBERG LIMITS FIGURE

LANDAU ASSOCIATES

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Portland, Oregon

Plasticity Chart

Figure
B6

GENERAL NOTES

SAMPLE IDENTIFICATION

The Unified Soil Classification System (USCS), AASHTO 1988 and ASTM designations D2487 and D-2488 are used to identify the encountered materials unless otherwise noted. Coarse-grained soils are defined as having more than 50% of their dry weight retained on a #200 sieve (0.075mm); they are described as: boulders, cobbles, gravel or sand. Fine-grained soils have less than 50% of their dry weight retained on a #200 sieve; they are defined as silts or clay depending on their Atterberg Limit attributes. Major constituents may be added as modifiers and minor constituents may be added according to the relative proportions based on grain size.

DRILLING AND SAMPLING SYMBOLS

SFA:	Solid Flight Auger - typically 4" diameter flights, except where noted.
HSA:	Hollow Stem Auger - typically 3¼" or 4¼ I.D. openings, except where noted.
M.R.:	Mud Rotary - Uses a rotary head with Bentonite or Polymer Slurry
R.C.:	Diamond Bit Core Sampler
H.A.:	Hand Auger
P.A.:	Power Auger - Handheld motorized auger

☒	SS:	Split-Spoon - 1 3/8" I.D., 2" O.D., except where noted.
■	ST:	Shelby Tube - 3" O.D., except where noted.
▯	RC:	Rock Core
⬇	TC:	Texas Cone
✋	BS:	Bulk Sample
▨	PM:	Pressuremeter
	CPT-U:	Cone Penetrometer Testing with Pore-Pressure Readings

SOIL PROPERTY SYMBOLS

N:	Standard "N" penetration: Blows per foot of a 140 pound hammer falling 30 inches on a 2-inch O.D. Split-Spoon.
N_{60}:	A "N" penetration value corrected to an equivalent 60% hammer energy transfer efficiency (ETR)
Q_u:	Unconfined compressive strength, TSF
Q_p:	Pocket penetrometer value, unconfined compressive strength, TSF
w%:	Moisture/water content, %
LL:	Liquid Limit, %
PL:	Plastic Limit, %
PI:	Plasticity Index = (LL-PL),%
DD:	Dry unit weight, pcf
▼,▽,▼	Apparent groundwater level at time noted

RELATIVE DENSITY OF COARSE-GRAINED SOILS

Relative Density	N - Blows/foot
Very Loose	0 - 4
Loose	4 - 10
Medium Dense	10 - 30
Dense	30 - 50
Very Dense	50 - 80
Extremely Dense	80+

ANGULARITY OF COARSE-GRAINED PARTICLES

Description	Criteria
Angular:	Particles have sharp edges and relatively plane sides with unpolished surfaces
Subangular:	Particles are similar to angular description, but have rounded edges
Subrounded:	Particles have nearly plane sides, but have well-rounded corners and edges
Rounded:	Particles have smoothly curved sides and no edges

GRAIN-SIZE TERMINOLOGY

Component	Size Range
Boulders:	Over 300 mm (>12 in.)
Cobbles:	75 mm to 300 mm (3 in. to 12 in.)
Coarse-Grained Gravel:	19 mm to 75 mm (¾ in. to 3 in.)
Fine-Grained Gravel:	4.75 mm to 19 mm (No.4 to ¾ in.)
Coarse-Grained Sand:	2 mm to 4.75 mm (No.10 to No.4)
Medium-Grained Sand:	0.42 mm to 2 mm (No.40 to No.10)
Fine-Grained Sand:	0.075 mm to 0.42 mm (No. 200 to No.40)
Silt:	0.005 mm to 0.075 mm
Clay:	<0.005 mm

PARTICLE SHAPE

Description	Criteria
Flat:	Particles with width/thickness ratio > 3
Elongated:	Particles with length/width ratio > 3
Flat & Elongated:	Particles meet criteria for both flat and elongated

RELATIVE PROPORTIONS OF FINES

Descriptive Term	% Dry Weight
Trace:	< 5%
With:	5% to 12%
Modifier:	>12%

GENERAL NOTES

(Continued)

CONSISTENCY OF FINE-GRAINED SOILS

Q_U - TSF	N - Blows/foot	Consistency
0 - 0.25	0 - 2	Very Soft
0.25 - 0.50	2 - 4	Soft
0.50 - 1.00	4 - 8	Firm (Medium Stiff)
1.00 - 2.00	8 - 15	Stiff
2.00 - 4.00	15 - 30	Very Stiff
4.00 - 8.00	30 - 50	Hard
8.00+	50+	Very Hard

MOISTURE CONDITION DESCRIPTION

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

RELATIVE PROPORTIONS OF SAND AND GRAVEL

Descriptive Term	% Dry Weight
Trace:	< 15%
With:	15% to 30%
Modifier:	>30%

STRUCTURE DESCRIPTION

Description	Criteria
Stratified:	Alternating layers of varying material or color with layers at least ¼-inch (6 mm) thick
Laminated:	Alternating layers of varying material or color with layers less than ¼-inch (6 mm) thick
Fissured:	Breaks along definite planes of fracture with little resistance to fracturing
Slickensided:	Fracture planes appear polished or glossy, sometimes striated

Description	Criteria
Blocky:	Cohesive soil that can be broken down into small angular lumps which resist further breakdown
Lensed:	Inclusion of small pockets of different soils
Layer:	Inclusion greater than 3 inches thick (75 mm)
Seam:	Inclusion 1/8-inch to 3 inches (3 to 75 mm) thick extending through the sample
Parting:	Inclusion less than 1/8-inch (3 mm) thick

SCALE OF RELATIVE ROCK HARDNESS

Q_U - TSF	Consistency
2.5 - 10	Extremely Soft
10 - 50	Very Soft
50 - 250	Soft
250 - 525	Medium Hard
525 - 1,050	Moderately Hard
1,050 - 2,600	Hard
>2,600	Very Hard

ROCK VOIDS

Voids	Void Diameter
Pit	<6 mm (<0.25 in)
Vug	6 mm to 50 mm (0.25 in to 2 in)
Cavity	50 mm to 600 mm (2 in to 24 in)
Cave	>600 mm (>24 in)

ROCK BEDDING THICKNESSES

Description	Criteria
Very Thick Bedded	Greater than 3-foot (>1.0 m)
Thick Bedded	1-foot to 3-foot (0.3 m to 1.0 m)
Medium Bedded	4-inch to 1-foot (0.1 m to 0.3 m)
Thin Bedded	1¼-inch to 4-inch (30 mm to 100 mm)
Very Thin Bedded	½-inch to 1¼-inch (10 mm to 30 mm)
Thickly Laminated	1/8-inch to ½-inch (3 mm to 10 mm)
Thinly Laminated	1/8-inch or less "paper thin" (<3 mm)

GRAIN-SIZED TERMINOLOGY

(Typically Sedimentary Rock)

Component	Size Range
Very Coarse Grained	>4.76 mm
Coarse Grained	2.0 mm - 4.76 mm
Medium Grained	0.42 mm - 2.0 mm
Fine Grained	0.075 mm - 0.42 mm
Very Fine Grained	<0.075 mm

ROCK QUALITY DESCRIPTION

Rock Mass Description	RQD Value
Excellent	90 -100
Good	75 - 90
Fair	50 - 75
Poor	25 -50
Very Poor	Less than 25

DEGREE OF WEATHERING

Slightly Weathered:	Rock generally fresh, joints stained and discoloration extends into rock up to 25 mm (1 in), open joints may contain clay, core rings under hammer impact.
Weathered:	Rock mass is decomposed 50% or less, significant portions of the rock show discoloration and weathering effects, cores cannot be broken by hand or scraped by knife.
Highly Weathered:	Rock mass is more than 50% decomposed, complete discoloration of rock fabric, core may be extremely broken and gives clunk sound when struck by hammer, may be shaved with a knife.

SOIL CLASSIFICATION CHART

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

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

BORING GP1

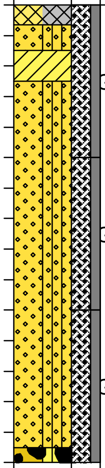
DATE STARTED: 10/24/19
DATE COMPLETED: 10/24/19
COMPLETION DEPTH 15.0 ft
BENCHMARK: N/A
ELEVATION: 36 ft
LATITUDE: 45.562891°
LONGITUDE: -122.735369°
STATION: N/A **OFFSET:** N/A
REMARKS:

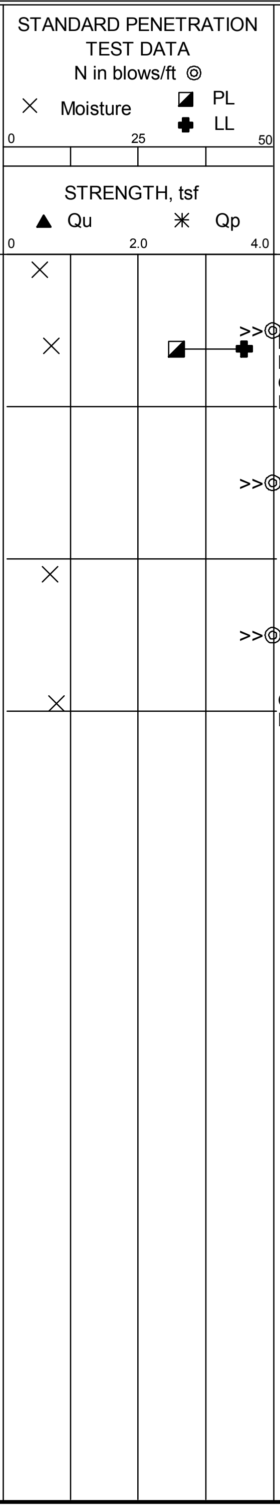
DRILL COMPANY: Oregon Geotechnical Explorations
DRILLER: Terry **LOGGED BY:** HR
DRILL RIG: Track Mounted Geoprobe
DRILLING METHOD: Geoprobe
SAMPLING METHOD: Continuous Core
HAMMER TYPE:
EFFICIENCY N/A%
REVIEWED BY: SB

Water		
∇	While Drilling	7 feet
▼	Upon Completion	N/A
∇	Delay	N/A

BORING LOCATION:

Elevation (feet)	Depth, (feet)	Sample Type	Sample No.	Recovery (inches)	MATERIAL DESCRIPTION	USCS Classification	Moisture, %	Additional Remarks
	0				**GRAVEL FILL with SAND** Brown, moist		7	
35					**SAND with SILT** Black, moist	SW-SM		
					LEAN CLAY Dark gray, moist	CL		
			GEO1	60	**SAND with SILT** Brown, moist to wet at 7 feet bgs		9	LL = 45 PL = 32 Gradation: Fines = 8.7%
30	5							
			GEO2	60		SP		
25	10						9	
			GEO3	60				
	15				**GRAVEL with SILT and SAND** Black, wet	GW-GM	10	Gradation: Fines = 8.6%
					Geoprobe terminated at 15 feet due to no recovery			





intertek psi

Professional Service Industries, Inc.
6032 N. Cutter Circle, Suite 480
Portland, OR 97219
Telephone: (503) 289-1778

PROJECT NO.: 07041274
PROJECT: McCall Oil and Chemical Corporation
LOCATION: 5480 NW Front Avenue
Portland, Oregon

The stratification lines represent approximate boundaries. The transition may be gradual.

DATE STARTED: 10/24/19
DATE COMPLETED: 10/24/19
COMPLETION DEPTH 35.0 ft
BENCHMARK: N/A
ELEVATION: 36 ft
LATITUDE: 45.562877°
LONGITUDE: -122.73538°
STATION: N/A **OFFSET:** N/A
REMARKS:

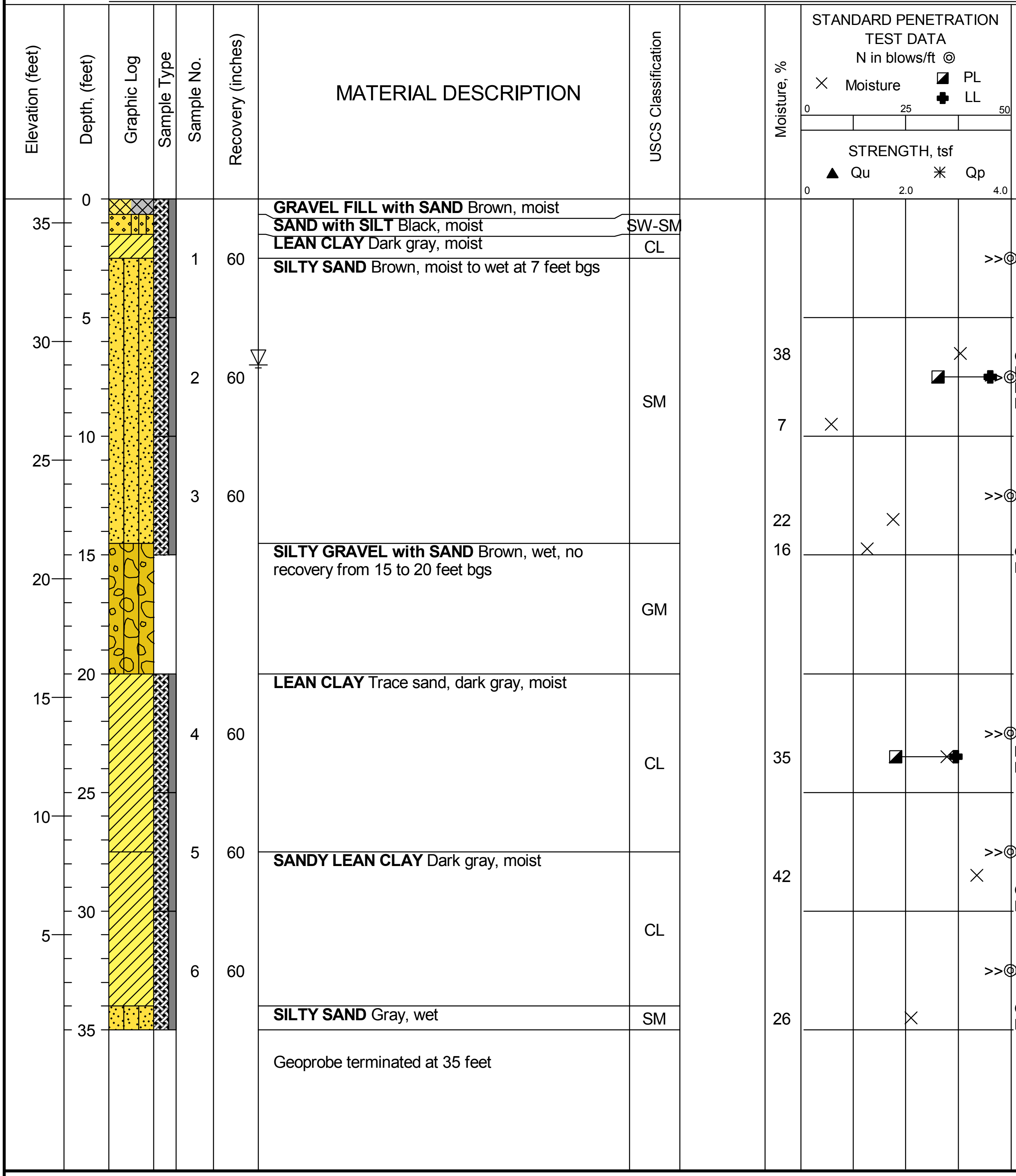
DRILL COMPANY: Oregon Geotechnical Explorations
DRILLER: Terry **LOGGED BY:** HR
DRILL RIG: Track Mounted Geoprobe
DRILLING METHOD: Geoprobe
SAMPLING METHOD: Continuous Core
HAMMER TYPE:
EFFICIENCY N/A%
REVIEWED BY: SB

BORING GP2

Water
▽ While Drilling 7 feet
▼ Upon Completion N/A
▽ Delay N/A

BORING LOCATION:

Sample No.	Recovery (inches)	MATERIAL DESCRIPTION	USCS Classification	Moisture, %	STANDARD PENETRATION TEST DATA N in blows/ft ◎	Additional Remarks
		GRAVEL FILL with SAND Brown, moist				
		SAND with SILT Black, moist	SW-SM			
		LEAN CLAY Dark gray, moist	CL			
1	60	**SILTY SAND** Brown, moist to wet at 7 feet bgs	SM		>>◎	
2	60			38 7	◎	Gradation: Fines = 47.4% LL = 45 PL = 33
3	60			22 16	>>◎	Gradation: Fines = 14.2%
		SILTY GRAVEL with SAND Brown, wet, no recovery from 15 to 20 feet bgs	GM			
4	60	**LEAN CLAY** Trace sand, dark gray, moist	CL	35	>>◎	LL = 37 PL = 23
5	60	**SANDY LEAN CLAY** Dark gray, moist	CL	42	>>◎	Gradation: Fines = 63%
6	60				>>◎	
		SILTY SAND Gray, wet	SM	26		Gradation: Fines = 12.6%
		Geoprobe terminated at 35 feet				



Professional Service Industries, Inc.
6032 N. Cutter Circle, Suite 480
Portland, OR 97219
Telephone: (503) 289-1778

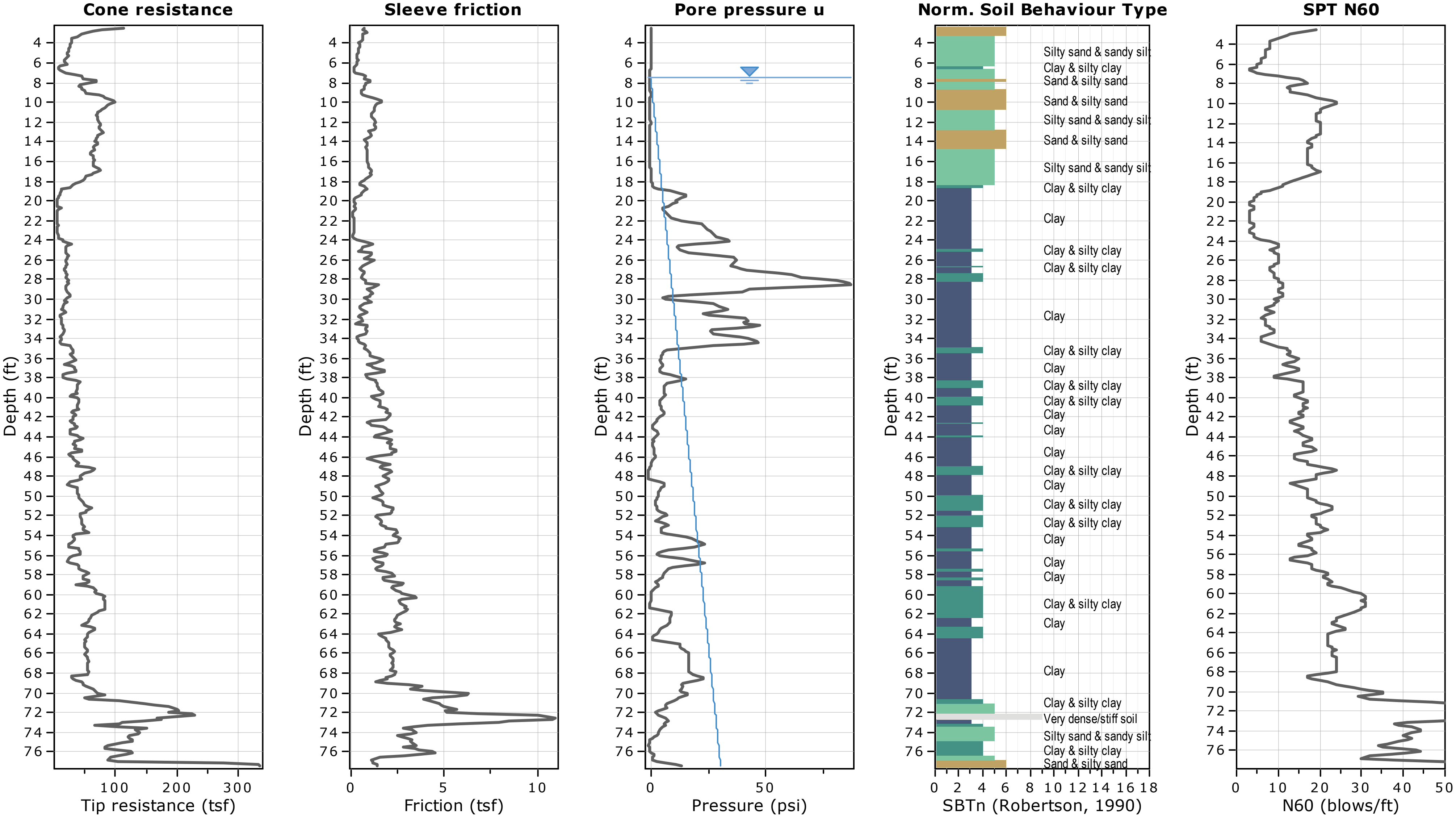
PROJECT NO.: 07041274
PROJECT: McCall Oil and Chemical Corporation
LOCATION: 5480 NW Front Avenue
Portland, Oregon

The stratification lines represent approximate boundaries. The transition may be gradual.

Intertek PSI
6032 North Cutter Circle Suite 480

Project: **McCall Oil and Chemical Corporation PMA Project**
Location: **Front Ave, Portland, OR**

CPT: 19181 CPT-1 Text File
Total depth: 77.43 ft, Date: 11/13/2019
Surface Elevation: 0.00 ft
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator:

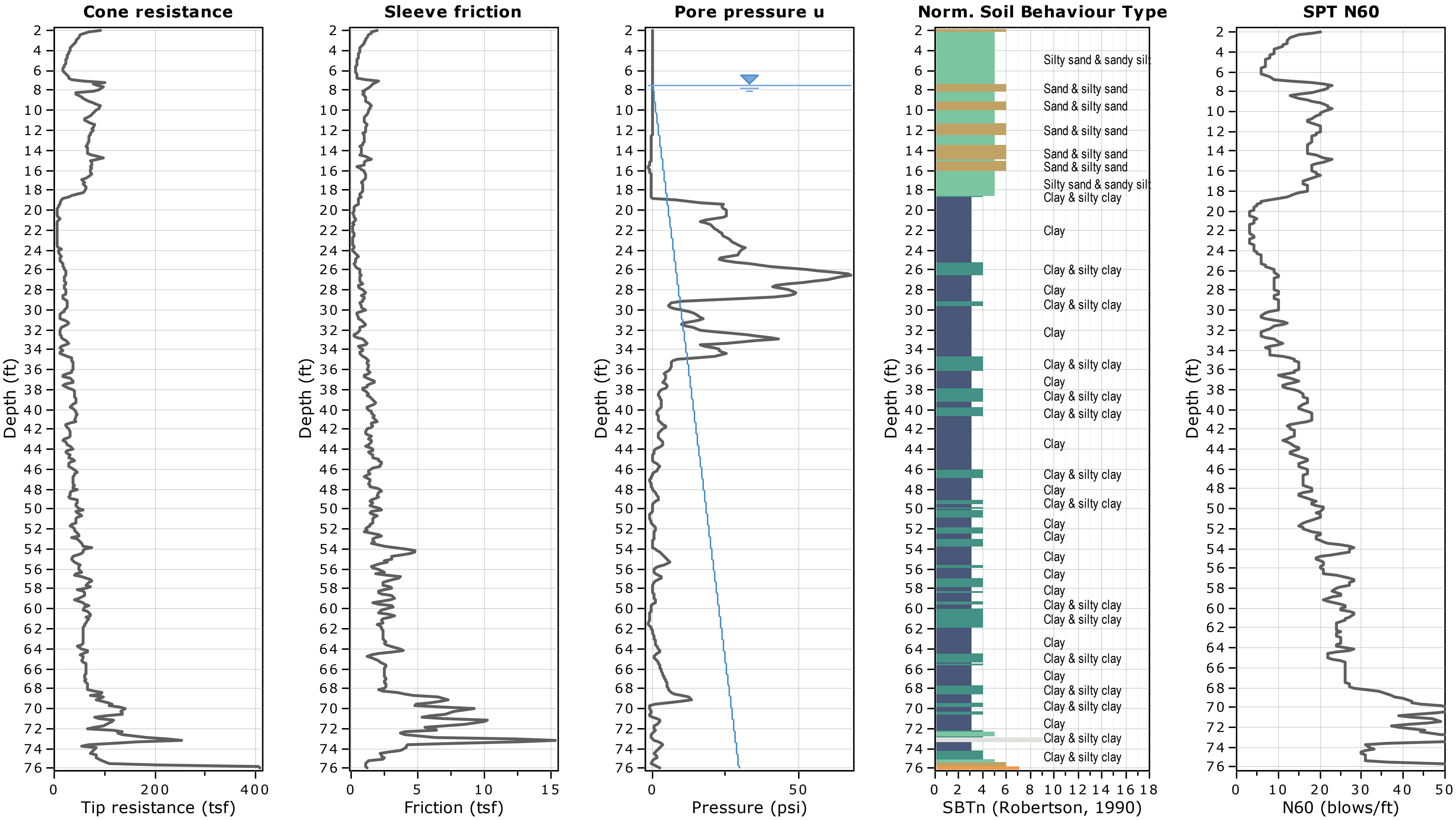


CPeT-IT v.3.0.3.2 - CPTU data presentation & interpretation software - Report created on: 11/13/2019, 2:01:44 PM
Project file: \\portland-fs1\PROJECTS\704 Geotech & Environmental\0704700 to present\07041274 McCall Oil and Chemical PMA Project\CPT Data\CPET-IT Data Mccall oil.cpt

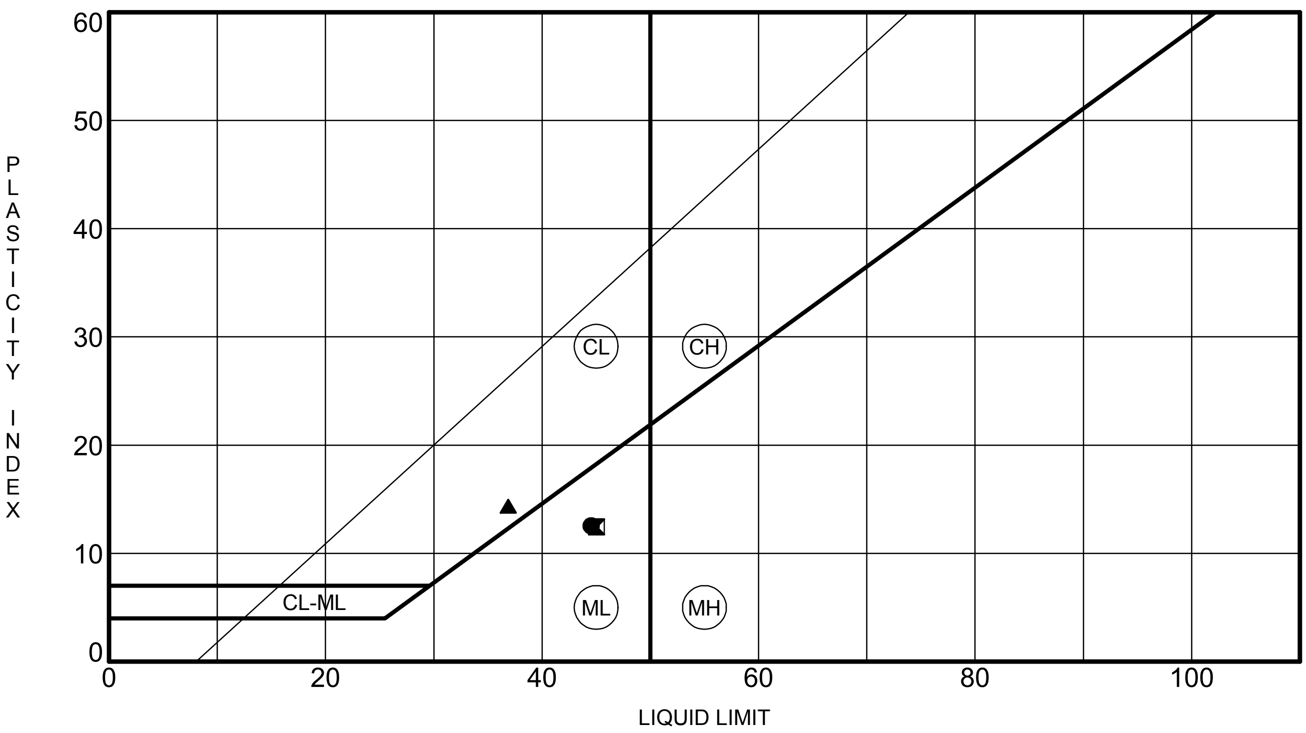
Intertek PSI
6032 North Cutter Circle Suite 480

CPT: 19181 CPT-2 Text File
Total depth: 75.95 ft, Date: 11/13/2019
Surface Elevation: 0.00 ft
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator:

Project: **McCall Oil and Chemical Corporation PMA Project**
Location: **Front Ave, Portland, OR**



APPENDIX B – LABORATORY TEST RESULTS



	Boring	Depth (ft)	LL	PL	PI	Fines	Classification (*Visual)
●	**GP1**	**1.5**	**45**	**32**	**13**		
☒	**GP2**	**7.5**	**45**	**33**	**12**		
▲	**GP2**	**23.5**	**37**	**23**	**14**		

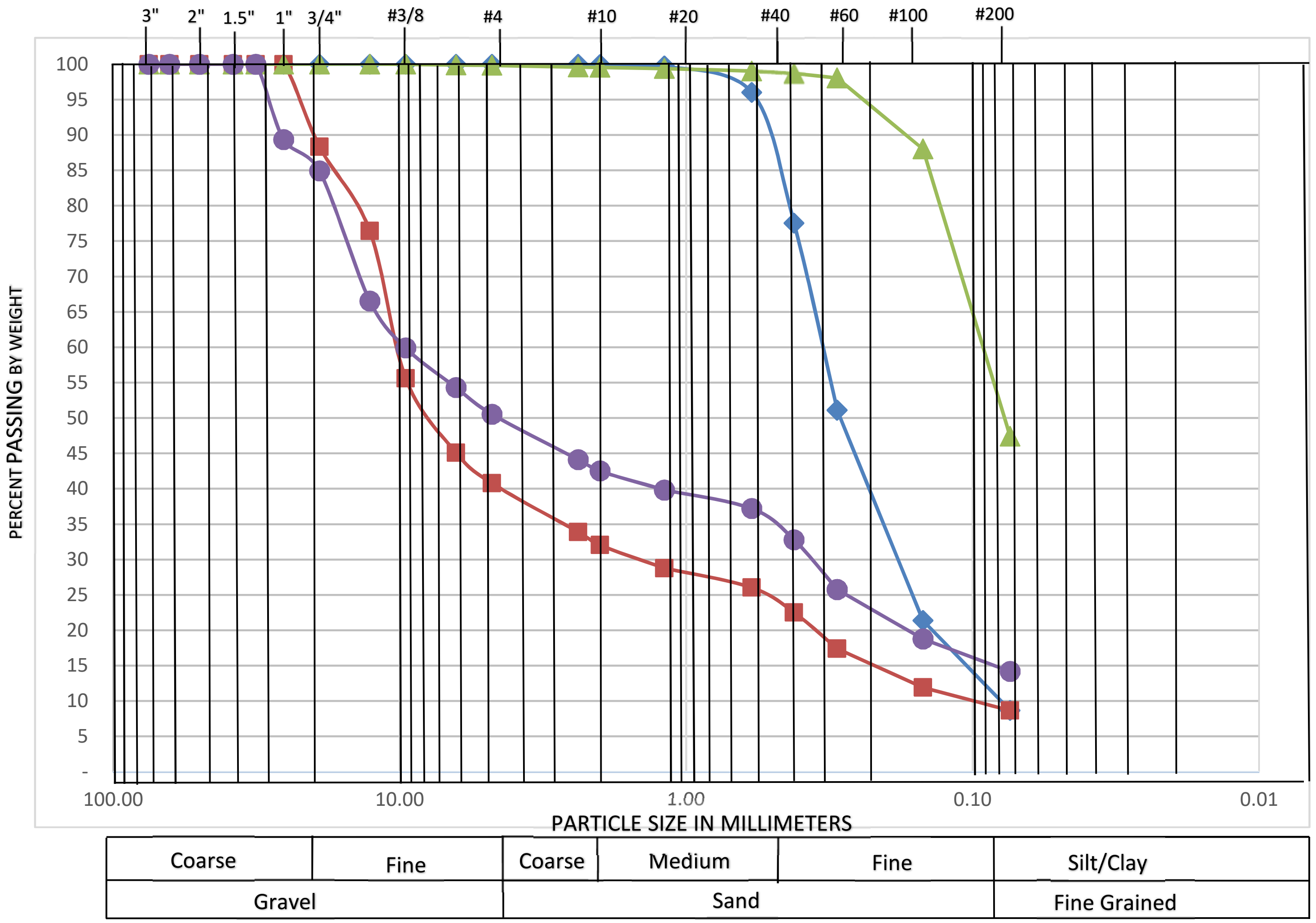
Professional Service Industries, Inc.
6032 N. Cutter Circle, Suite 480
Portland, OR 97219
Telephone: (503) 289-1778
Fax: (503) 289-1918

ATTERBERG LIMIT RESULTS

PSI Job No.: 07041274
Project: McCall Oil and Chemical
Location:

PARTICLE SIZE ANALYSIS - ASTM (D-422)

Project Name McCall Oil and Chemical Corporation
Project Location Portland, OR
Project Number 07041274
Tested By EL
Date of Sampling 10/24/2019
Date of Testing 11/8/2019
Reviewed By SB

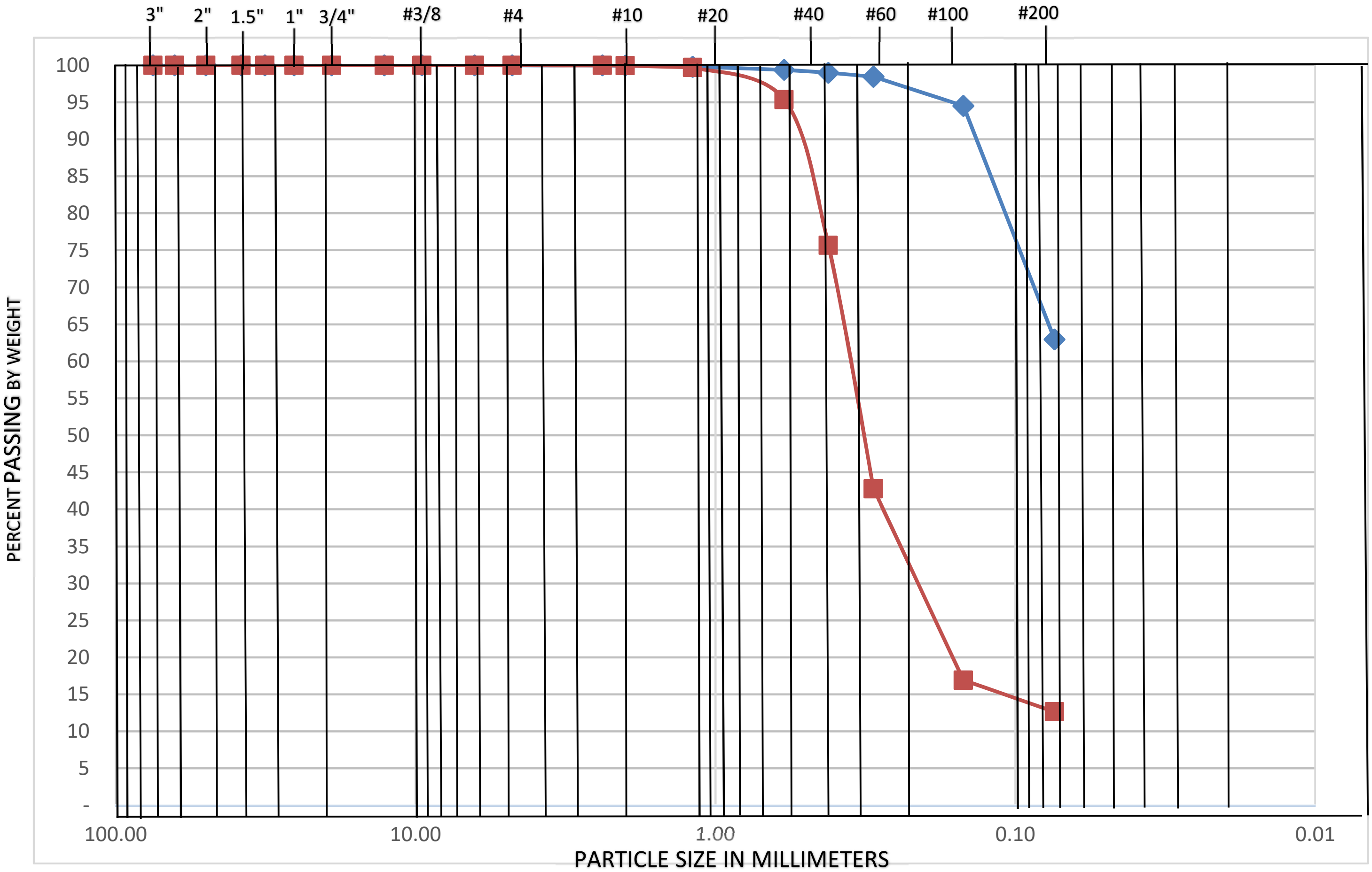


Boring #	Depth	% Gravel	% Sand	% Fines	PL	LL	PI	Moisture (%)
GP1	3		91.3%	8.7%				8.9%
GP1	14.75	59.2%	32.1%	8.6%				9.9%
GP2	6.5	0.2%	52.4%	47.4%				38.1%
GP2	14.75	49.5%	36.3%	14.2%				15.8%

Boring #	Depth	USCS Symbol	USCS Name	Plot Lines
GP1	3	SW-SM	Well Graded SAND with Silt	◆
GP1	14.75	GW-GM	Well Graded GRAVEL with Silt and Sand	■
GP2	6.5	SM	Silty SAND	▲
GP2	14.75	GM	Silty GRAVEL with Sand	●

PARTICLE SIZE ANALYSIS - ASTM (D-422)

Project Name McCall Oil and Chemical Corporation
Project Location Portland, OR
Project Number 07041274
Tested By EL
Date of Sampling 10/24/2019
Date of Testing 11/8/2019
Reviewed By SB



Coarse	Fine	Coarse	Medium	Fine	Silt/Clay
Gravel		Sand			Fine Grained

Boring #	Depth	% Gravel	% Sand	% Fines	PL	LL	PI	Moisture (%)
GP2	29		37.0%	63.0%				42.0%
GP2	34.50		87.4%	12.6%				26.2%

Boring #	Depth	USCS Symbol	USCS Name	Plot Lines
GP2	29	CL	Sandy Lean CLAY	◆
GP2	34.50	SM	Silty SAND	■

Appendix A.2

Historical Geotechnical Reports

PACRIM GEOTECHNICAL INC.

GEOTECHNICAL REPORT
PROPOSED REPLACEMENT OF ASPHALT TANKS
5480 FRONT AVENUE
PORTLAND, OREGON
FOR
MCCALL OIL AND CHEMICAL COMPANY

Project No. 097-001
December 10, 1999

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geotechnical
engineering
and
applied
earth
sciences

PACRIM GEOTECHNICAL INC.

GEOTECHNICAL ENGINEERING AND APPLIED EARTH SCIENCES

10700 Meridian Ave. N., Suite 210 • Seattle, WA 98133 • Phone: (206) 365-8770 • Fax: (206) 365-8405

December 10, 1999
Project Number 097-001

Mr. Steve Vertner
Beacon Engineers Inc.
18940 – NE 150th Street
Woodinville, Washington 98072

Geotechnical Report
Proposed Replacement of Asphalt Tanks
5480 Front Ave.
Portland, Oregon

Dear Steve:

In accordance with your request, PacRim Geotechnical Inc. performed a geotechnical engineering study for the proposed replacement of asphalt storage tanks TK 1010, TK 1011, TK 1012, and TK 1013 at the McCall Oil & Chemical Company Facility in Portland, Oregon. The results of our study are presented in the accompanying geotechnical report.

We appreciate the opportunity to provide geotechnical services on this project. Should you have any questions or comments, or if we may be of further service, please call.

Sincerely,

PACRIM GEOTECHNICAL INC.

Harbans L. Chabra, P.E.
Principal

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GEOTECHNICAL REPORT
PROPOSED REPLACEMENT OF ASPHALT TANKS
5480 FRONT AVENUE
PORTLAND, OREGON

1.0 INRODUCTION

1.1 GENERAL

PacRim Geotechnical Inc. (PacRim) completed a geotechnical engineering study related to the proposed asphalt storage tank replacements at the McCall Oil & Chemical Company facility in Portland, Oregon. The approximate location of the project site and general site layout are shown on the Vicinity Map (Figure 1) and the Site and Exploration Map (Figure 2). The purpose of the geotechnical study was to explore and evaluate the surface and subsurface conditions at the site, and based on the conditions observed, provide recommendations pertaining to geotechnical aspects of the project.

1.2 PROJECT DESCRIPTION

The project involves replacing Tank Numbers TK 1010, TK 1011, TK 1012, and TK 1013. The tanks are located at the southwest corner of the property within a group of about 8 storage tanks, varying in diameter from 10 to 42.5 feet, Figure 2. All of the tanks are supported on grade without foundation elements. Immediately to the north of the tank area is the Great Western chemical tank farm. A 10-foot high by 1 to 1.5 feet thick concrete wall separates the two properties. Tanks TK 1012 and TK 1013 are adjacent to the concrete wall and are located about 7 feet away from the chemical storage tanks on the Great Western property.

The four tanks that will be replaced are used to store asphalt. The existing tanks are 20 feet in diameter, 18 feet high, with an approximate storage capacity of 1000 barrels. We understand that the tanks will be replaced with tanks of the same diameter, but 40 feet in height, which will double the storage capacity. The tanks will continue to be used to store asphalt.

1.3 AUTHORIZATION AND SCOPE OF WORK

A proposed scope of work and cost estimate for this geotechnical investigation was submitted by PacRim to Beacon Engineers on November 12, 1999. Mr. Steve Vertner with Beacon Engineers subsequently authorized the proposed scope of work. Our scope of work included conducting a geotechnical exploration program including drilling two borings, performing laboratory testing and engineering analyses to develop geotechnical design recommendations for the proposed improvements, and preparing this report.

2.0 FIELD EXPLORATIONS

On November 30, 1999, PacRim personnel conducted subsurface explorations at the site. The planned depths of the soil borings were increased from 40 to 51 feet, due to the greater than expected thickness of loose sand. The explorations performed consisted of drilling and sampling two borings (B-1-99 and B-2-99) to a depth of 51.5 feet below the existing ground surface. Approximate locations of the explorations are shown on the Site and Exploration Map, Figure 2.

The explorations were conducted under the direction of a PacRim geotechnical engineer, who also logged and obtained soil samples at selected intervals in each of the borings. Appendix A contains summary logs of the explorations and describes the field exploration methodology in greater detail.

3.0 SITE CONDITIONS

3.1 SURFACE CONDITIONS

The project area is relatively flat and located approximately 800 feet from the south shore of the Willamette River, as shown on the Vicinity Map, Figure 1. The area surrounding the site is also relatively flat with a slight slope towards the river. Front Ave. bounds the McCall Oil property to the southwest and the Willamette River borders the property to the northeast. To the northwest is the Great Western chemical production facility, and to the southeast is a tube fabrication plant. The project area is located in an area that was formed by reclaiming flood lands.

3.2 GENERAL GEOLOGIC CONDITIONS

Geologic information for the site was obtained from mapping by M.H. Beeson and others (1991), and I.P. Madin (1990). The area is underlain by artificial fill overlying naturally-occurring alluvial sediments. The artificial fill is of variable concentration including sand, silt and clay fills with subordinate amounts of gravel, debris, and local concentrations of sawdust. The most common material used for fill is dredged river sand (Madin, 1990). The mapped fill areas are shown on the geologic maps where sufficient fill has been placed to eliminate lakes, sloughs, or gullies present during the 1898 survey for the earliest topographic map of Portland. Fill depths of 5 to 15 feet are common along the Willamette River flood plain.

Underlying the fill are alluvial sediments that were deposited in the channels and on flood plains. The alluvial sediments are up to 135 feet thick and consist of river and stream deposits of silt, sand, and organic rich clay with subordinate gravel (Beeson, M.H., 1991).

3.2.1 Siesmicity:

General information, regarding site seismicity and earthquake hazards, was obtained from the Earthquake Hazard Maps of the Portland Quadrangle, by Mabey and others (1993). Hazards identified on the maps were determined for two plausible earthquakes that the Portland area is susceptible to: a large subduction zone event and a shallow crustal earthquake. The large subduction zone event used for identifying hazards is assumed to be 100 km away with Mw=8.5 and peak ground acceleration of a=0.23g. The shallow-focus crustal earthquake is 10 km away with Mw=6.5 and a=0.31g.

The maps show the project area as being underlain by 10 to 30 feet of liquifiable sediments. The estimates of lateral ground displacements are 3 to 4 feet during the design large subduction zone event and 1.5 to 2.5 feet for the design shallow crustal earthquake. The ground motion amplification factor is shown as 1 to 1.4 for the project area.

3.3 SUBSURFACE CONDITIONS

3.3.1 Soil

During our exploration program, artificial fill was encountered from the ground surface to depths of 18 feet in Boring B-1-99 and 22 feet in B-2-99. Alluvium was encountered beneath the fill in both borings. The borings were both terminated in Alluvium. Below are descriptions of the soil deposits encountered in our explorations in the order by which they were encountered in the borings, with the youngest unit described first.

- Artificial Fill – The artificial fill encountered in the borings consists primarily of brown fine sand with trace of silt. The consistency of the sand varies from loose to medium dense.

- Alluvium

    - Sand – The alluvial sand encountered beneath the artificial fill is composed of gray fine sand with scattered bits of wood, and occasional seams of silt. The consistency of the sand ranges from loose to medium dense.

    - Organic Silt – In both borings a 5 to 8 feet thick layer of brown organic silt was encountered. The silt is medium stiff in consistency and contains abundant organic debris. The silt displays low strength and moderate compressibility.

    - Interbedded Silty Clay, Silt and Silty Sand – Below the organic silt, interbedded deposits of dark gray to greenish gray silt, silty clay, and silty sand were encountered. The finer grained layers were typically medium stiff to stiff in consistency, and the granular silty sand strata were medium dense. The borings were terminated within this unit.

3.3.2 Groundwater

Groundwater was encountered at a depth of about 14 feet in both borings. Groundwater elevations were measured following drilling and after the bore holes were left open for about 1 hour. It should be noted that observations of groundwater levels during or immediately after drilling can be misleading. The groundwater conditions reported on the boring logs are for the specific date and locations indicated, and therefore may not necessarily be indicative of other times and/or locations. Furthermore, we anticipate that groundwater conditions will vary depending on the season, local subsurface conditions, and particularly the water level in the Willamette River.

4.0 LABORATORY TESTS

Laboratory tests were conducted on selected soil samples to characterize certain physical properties of the on-site soils. Laboratory testing included determination of moisture content, grain size distribution, fines content, Atterberg Limits, and a consolidation test. Testing of soil samples was conducted in general accordance with appropriate American Society for Testing and Materials (ASTM) standards, as discussed in Appendix B. Moisture content test results are displayed on the appropriate summary boring logs in Appendix A. Grain size distribution, fines content, Atterberg Limits, and consolidation test results are presented in Appendix B.

5.0 CONCLUSIONS AND RECOMMENDATIONS

Based on the results of our subsurface investigation, laboratory testing, and engineering analyses, it is our opinion that the new storage tanks can be supported on either pile foundations or on two large mat foundations. The loose nature of the artificial fill and alluvial sand deposits underlying the site are highly susceptible to liquefaction during the design 500 year seismic event. The proximity of the tank locations to the river along with the possibility of liquefaction results in the potential for lateral spreading. Typical pile foundations, capable of supporting the estimated static loads, will be susceptible to down drag and significant lateral forces as a result of soil liquefaction that is predicted for the design 500 year earthquake. Based on the seismic hazards and likelihood of pile failure, we believe that two large mat foundations supporting the four tanks is the preferred type of foundation support. The following report sections present recommendations for structural foundations, seismic considerations, site earthwork, and excavation.

5.1 SEISMIC CONSIDERATIONS

5.1.1 Site Seismicity

The project site lies within Seismic Zone 3, as defined in Chapter 16, Division IV of the 1997 Uniform Building Code (UBC). Seismic Zone 3 includes the western portion of Oregon, and represents an area of relatively high seismic risk. For comparison, much of California and southern Alaska are defined as Seismic Zone 4, which is an area of highest seismic risk. Consequently, moderate levels of earthquake shaking should be anticipated during the design life of the proposed improvements, and the structures should be designed to resist earthquake loading in accordance with the methodology described in the 1997 UBC.

Based on the subsurface conditions we observed during our exploration program, UBC Soil Type S_E may be assumed for the site. The corresponding normalized response spectra for the assumed UBC Soil Type is considered appropriate for the site. Based on the 1996 USGS *National Seismic Hazards Mapping Project*, the peak ground acceleration indicated for the site is 0.19g. The referenced USGS seismic design map is based on an earthquake hazard consistent with a "500-year" event, or a seismic event that has a 10 percent probability of being exceeded in a 50 year period.

5.1.2 Soil Liquefaction and Lateral Spreading

Liquefaction occurs when loose, saturated and relatively cohesionless soil deposits temporarily lose strength as a result of earthquake shaking. Potential effects of soil liquefaction include temporary loss of bearing capacity and lateral soil resistance, liquefaction-induced settlement, and lateral spreading, any of which could result in significant structural damage. Primary factors controlling the development of liquefaction include intensity and duration of strong ground motion, characteristics of subsurface soil, in-situ stress conditions and the depth to groundwater.

We performed soil liquefaction analyses, using subsurface data from the current study, the methodology of Seed et al. (1985), and assuming the following seismicity parameters:

Table 1. Seismic Design Parameters

	Magnitude (M)	Max. Horizontal Acceleration (g)
500 year seismic event (USGS 1996)	7.5	0.19
Maximum Plausible Shallow Crustal Earthquake	6½	0.31
Maximum Plausible Subduction Zone Earthquake	8½	0.23

Based on the field and laboratory data, and engineering analyses, it is estimated that large areas of the site would experience liquefaction under the 500-year seismic event and the two maximum plausible events. Thickness of the liquefiable soil zone is about 20 to 22 feet. We estimate liquefaction induced settlement for the 500 year seismic event may be in the range of about 5 inches. The proximity of the tank locations to the Willamette River, along with the potential for widespread liquefaction, results in the possibility of lateral spread during a seismic event. Lateral spread displacement was estimated using the empirical analysis method of Bartlett and Youd 1992. We estimate displacement due to lateral spread to be about 3 feet.

5.2 FOUNDATION SUPPORT

Pile foundations are typically used to mitigate effects of liquefaction induced settlement. However, significantly deep piles and/or extensive ground improvement would be required to withstand the downdrag and lateral forces due to liquefaction. Shorter more economical piles could be used for static loading conditions, however, they will likely fail during the 500 year seismic event. A large rigid mat foundation, supporting two tanks, will also be susceptible to the liquefaction induced settlement and lateral spread, but is expected to provide a stable platform that will settle or tilt uniformly during a seismic event, and still maintain tank integrity.

We recommend using two large mat foundations to support the four tanks. Tanks TK 1010 and TK 1011 can be supported on one mat and TK 1012 and TK 1013 on the other. We understand the mat foundation will be rectangular, approximately 26 feet by 55 feet in plan, with the base of the slab approximately 4 feet below grade. The new footings and mat foundation should be designed using an allowable bearing pressure of 1,500 pounds per square foot (psf) to limit differential settlement. The mat foundation excavation subgrade should be prepared as recommended in Section 5.3.1; structural fill should be placed as recommended in Section 5.3.2.

Assuming construction is accomplished as recommended herein, and for the foundation loads anticipated, we estimate total settlement of the mat foundations of less than about 1 to 1½ inches and differential settlement across the mat of less than about 1/2 inch. We anticipate that the majority of the estimated settlement will occur during construction as the loads are applied, and as the tanks are hydrotested. We recommend using flexible connectors where piping is attached to the tanks, or to machinery supported on the mat foundation.

For design of rigid mat foundations we recommend using a modulus of subgrade reaction of 250 pounds per cubic inch for the native sand at the base of the foundation excavation.

The value provided above assumes that the subgrade will be prepared in accordance with the recommendations given below in section 5.3.

Wind, earthquakes, and unbalanced earth loads will subject the proposed structure to lateral forces. Lateral forces on a structure will be resisted by a combination of sliding resistance of its base or foundation on the underlying soil and passive earth pressure against the buried portions of the structure. For use in design, a coefficient of friction of 0.4 may be assumed along the interface between the base of the footing and the layer of structural fill placed on the native soils. Passive earth pressure of 250 pcf may be assumed for soils adjacent to footings or other below-grade elements. The upper 1 foot of passive resistance should be neglected in design, unless the footing is protected by a floor slab or pavement. These values assume a safety factor of approximately 1.5.

5.3 SITE EARTHWORK

5.3.1 Subgrade Preparation

Subgrade preparation beneath the mat foundations begins with excavation to the required depth. Excavation should extend 12-inches below the base of the foundation. The exposed subgrade should be compacted to a dense unyielding condition. We anticipate that this can be done with a vibratory plate compactor or static rolling with a small smooth drum roller. Vibratory compaction efforts should be halted and reevaluated if surrounding structures are adversely affected. After excavation and recompaction the geotechnical engineer should evaluate the subgrade. Soft or saturated soils that can not be recompacted should be overexcavated and replaced with approved structural fill material.

We do not expect ground water to be encountered during excavation. Ground water elevations measured during drilling were 14 feet below the existing ground surface.

After excavation and compaction of the subgrade, a working surface consisting of 12 inches of compacted 1 ¼ -inch minus crushed rock should then be placed over the subgrade. The layer should be placed in two lifts and compacted to a dense unyielding condition.

5.3.2 Structural Fill Materials and Compaction

Based on the results of our field exploration and laboratory testing program, we anticipate that the excavation spoils will consist of fine to medium sand. Provided the spoils do not contain deleterious material such as wood, garbage, or other undesirables, and the moisture content is within 2 percent of the optimum moisture content, the material can be reused as structural fill around the mat foundation.

Imported fill materials should meet the requirements for Gravel Borrow, as described in Section 9-03.14 of the 1998 WSDOT *Standard Specifications for Road, Bridge, and Municipal Construction* (WSDOT *Standard Specifications*).

Structural fill soils should be moisture conditioned to within about 2 percent of optimum moisture content, placed in loose, horizontal lifts less than 8 inches in thickness, and compacted to at least 95 percent of the MDD, as determined using test method ASTM D 1557 (Modified Proctor).

Given the access restrictions around the tank storage area, it may be more economical to use flowable controlled density fill (CDF) to backfill around the mat foundation. The flowable fill can be pumped to the locations where it is needed.

The procedure to achieve the specified minimum relative compaction depends on the size and type of compacting equipment, the number of passes, thickness of the layer being compacted, and certain soil properties. When the size of the excavation restricts the use of heavy equipment, smaller equipment can be used, but the soil must be placed in thin enough lifts to achieve the required compaction. A sufficient number of in-place density tests and/or hand probing should be performed as the fill is placed to verify the required relative compaction is being achieved.

5.4 Excavation

Excavations will be required to construct the mat foundations. Based on the soil conditions observed in our explorations, we anticipate that the on-site soils can be excavated with conventional excavating equipment; however, care must be taken during construction to maintain stability of open excavations. Maintenance of safe working conditions, including temporary excavation stability, is the responsibility of the contractor.

6.0 UNCERTAINTY AND LIMITATIONS

We have prepared this report for Beacon Engineers Inc. and McCall Oil, for use in design of this project. This report should be provided in its entirety to prospective contractors for bidding and estimating purposes; however, the conclusions and interpretations presented in this report should not be construed as a warranty of the subsurface conditions. Experience has shown that soil and groundwater conditions can vary significantly over small distances. Inconsistent conditions can occur between explorations that may not be detected by a geotechnical study. If, during future site operations, subsurface conditions are encountered which vary appreciably from those described herein, PacRim should be notified for review of the recommendations of this report, and revision of such if necessary.

We recommend that PacRim be retained to review the plans and specifications and verify that our recommendations have been interpreted and implemented as intended. Sufficient geotechnical monitoring, testing and consultation should be provided during construction to confirm that the conditions encountered are consistent with those indicated by explorations.

Within the limitations of scope, schedule and budget, PacRim executed these services in accordance with generally accepted professional principles and practices in the fields of geotechnical engineering and engineering geology at the time the report was prepared. No warranty, express or implied, is made. The scope of our work did not include environmental assessments or evaluations regarding the presence or absence of wetlands or hazardous or toxic substances in the soil, surface water, or groundwater at this site.

◆

We appreciate the opportunity to be of service.

Sincerely,

PACRIM GEOTECHNICAL INC.

12,550
JULY 13, 1984
HARBANS LALL CHABRA

12/10/99

Kevin J Lamb

Kevin J. Lamb, P.E.
Project Engineer

Harbans L. Chabra, P.E.
Principal

REFERENCES

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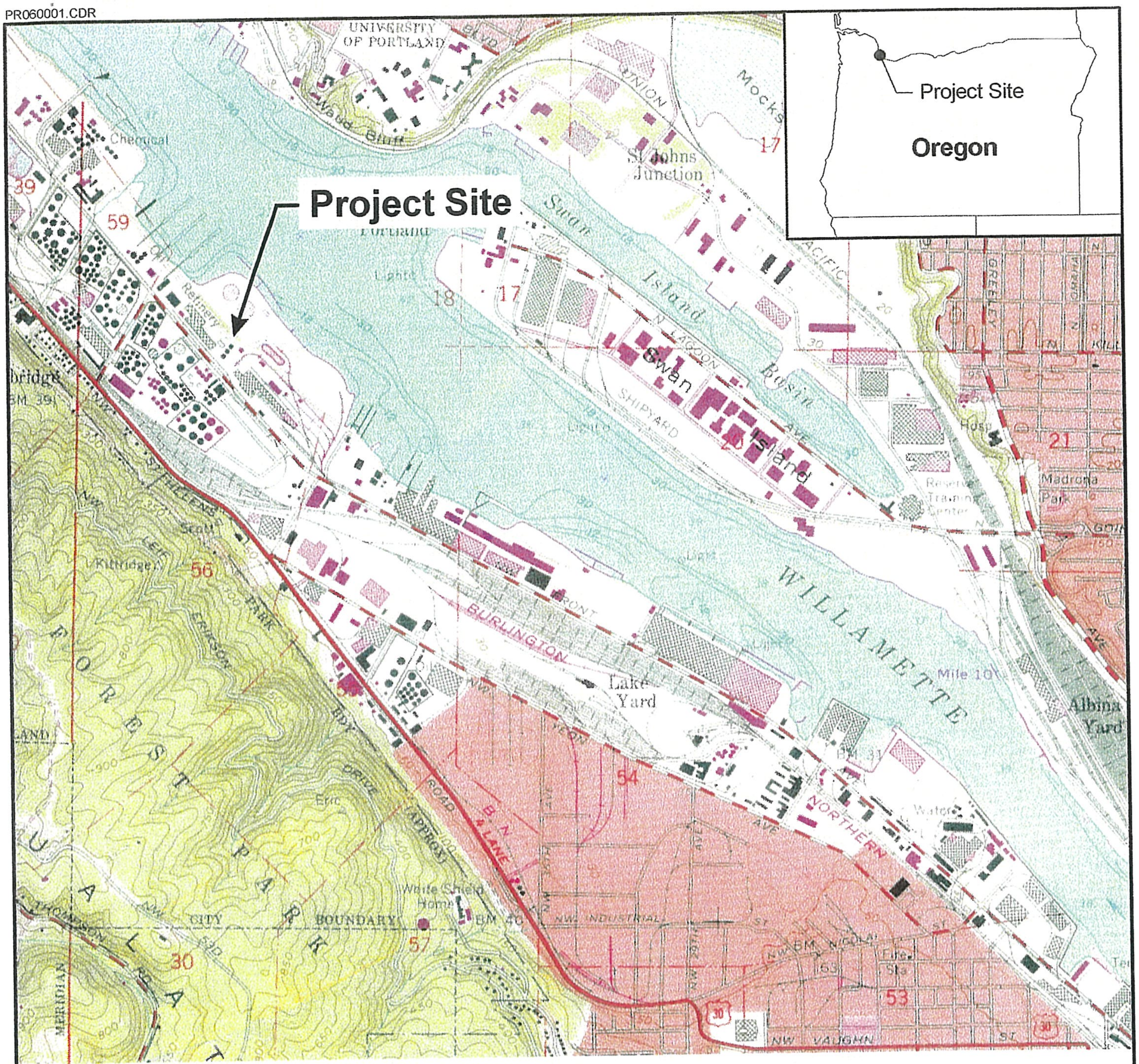
Beeson, M.H., T. L. Tolan, 1991, Geologic Map of the Portland Quadrangle, Multnomah and Washington Counties, Oregon, and Clark County, Washington, State of Oregon DOGAMI, Geologic Map Series, GMS 75.

Mabley, M.A., I.P. Madin, T.L. Youd, C.F. Jones, 1993, Earthquake Hazard Maps of the Portland Quadrangle, Multnomah and Washington Counties, Oregon, and Clark County, Washington, State of Oregon DOGAMI, Geologic Map Series GMS 79.

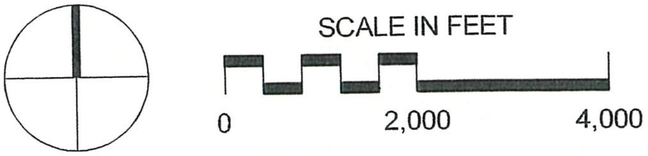
Madin, I.P., 1990, Earthquake-Hazard Geology Maps of the Portland Metropolitan Area, Oregon: Text and Map Explanation, State of Oregon, DOGAMI, Open File Report 0-90-2.

Seed, H.B., K. Tokimatsu, L.F. Harder, R.M. Chung, (1985), Influence of SPT Procedures in Soil Liquefaction Resistance Evaluations, Journal of the Geotechnical Engineering Division, American Society of Civil Engineers, Vol. 111, No. GT12, pp. 1425-1445.

PR060001.CDR



SOURCE: USGS topographic map, Portland, Washington, 1977



McCall Oil
Portland, Oregon

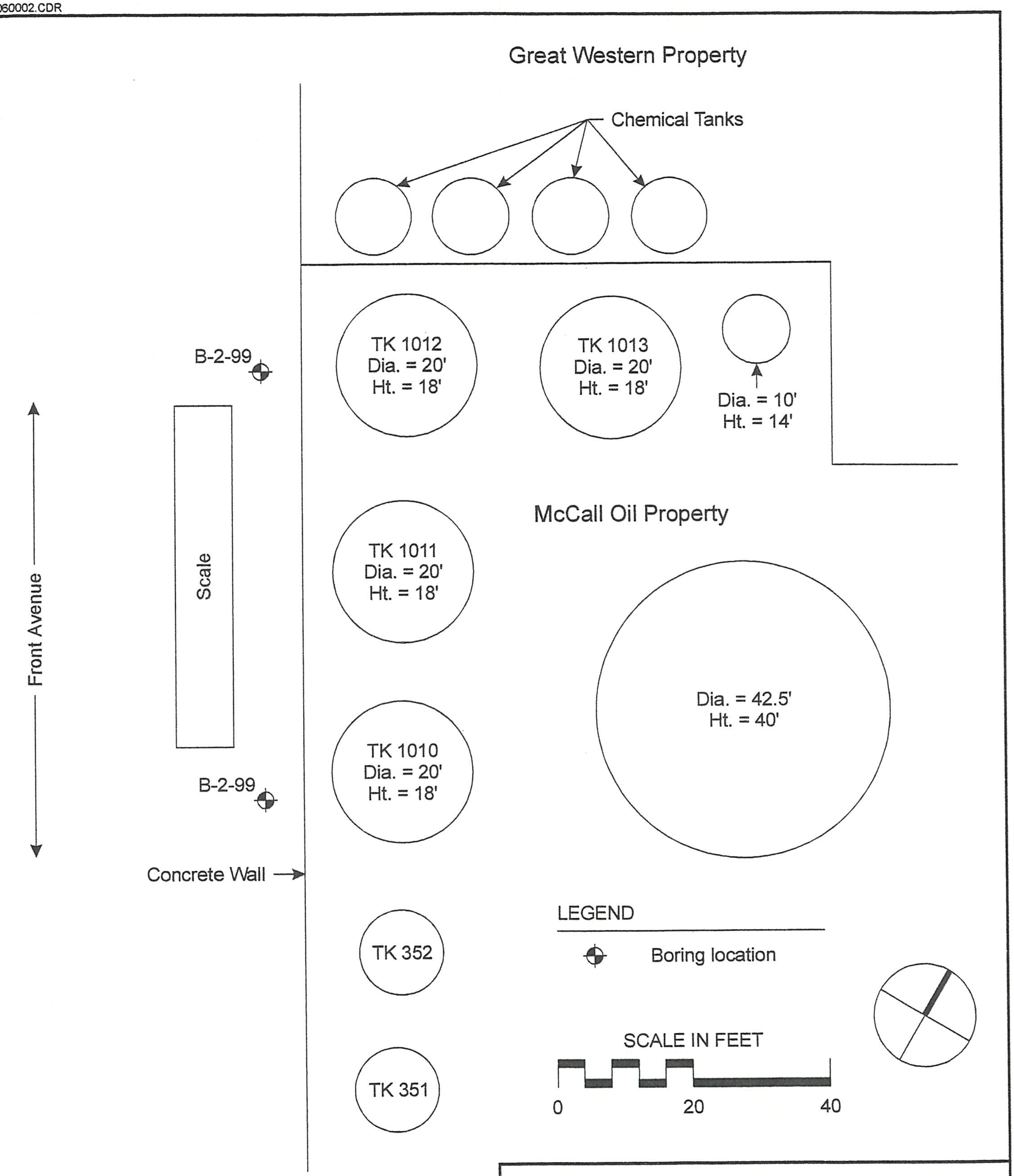
VICINITY MAP

Project No.: 097-001

FIGURE 1

PACRIM GEOTECHNICAL INC.
GEOTECHNICAL ENGINEERING AND APPLIED EARTH SCIENCES

PR060002.CDR



McCall Oil
Portland, Oregon

PACRIM GEOTECHNICAL INC.
GEOTECHNICAL ENGINEERING AND APPLIED EARTH SCIENCES

SITE AND EXPLORATION PLAN

Project No.: 097-001

FIGURE 2

APPENDIX A

FIELD EXPLORATION PROGRAM

Subsurface conditions for the project area were explored by drilling two borings using mud rotary drilling techniques. The two borings (B-1-99 and B-2-99) were drilled at selected locations on November 30, 1999, to a depth of 51.5 feet below ground surface. The borings were completed using a truck-mounted, modified Mobile B-61 drill rig owned and operated by Geo-Tech Explorations of Tualatin, Oregon under subcontract to PacRim Geotechnical Inc. The drill rig was equipped with a 2 7/8 inch outside diameter (O.D.) tricone drill bit with 2¾-inch O.D. drill rods.

The approximate locations of the explorations are shown on Figure 2 in the main body of the report. Subsurface information from the borings is shown on the summary boring logs included in this appendix as Figures A-1 and A-2. Figure A-3 is a key to symbols and descriptions used on the boring logs.

Soil samples were obtained from all of the borings at 2.5 feet intervals to a depth of 10 feet and 5-foot intervals thereafter. Samples were obtained using a 2-inch outside diameter Standard Penetration Test (SPT) sampler. A relatively undisturbed sample of the organic silt, encountered in boring B-2-99 was obtained at a depth of 40 to 42 feet with a Shelby tube. The sampler was driven into the soil a distance of 18 inches using a 140-pound hammer freely falling a distance of 30 inches. The hammer was operated using a rope and cathead system. Recorded blows for each 6 inches of sampler penetration are shown on the boring logs. The blow counts provide a qualitative measure of the relative density of cohesionless soil, or the relative consistency of fine-grained soils. Representative portions of the samples were placed in moisture tight plastic jars and transported to our laboratory for further observation and testing.

The explorations were located in the field by measurement from existing features shown on site plans. The approximate ground surface elevations at the exploration locations were also determined based on the available topographic maps. The locations and elevations of the explorations should be considered accurate only to the degree implied by the methods used.

PacRim personnel were present throughout the field exploration program to observe the borings, assist in sampling, and to prepare descriptive logs of the explorations. Soils were classified in general accordance with ASTM D-2488, *Standard Practice for Description and Identification of Soils (Visual-Manual Procedure)*. The exploration logs in this appendix represent our interpretation of subsurface conditions observed in the field, and the results of laboratory testing.

Other Tests	Sample No.	Dry Density (pcf)	Moisture Content (%)	Blows/6 inches	Sample	Depth (feet)	USCS	Graphic Symbol	DESCRIPTION	Elevation (feet)	Groundwater	Well Detail
									Loose, brown, fine SAND; dry. (ARTIFICIAL FILL)			
	1		19	4 2 3								
	2			5 4 6		5						
	3		16	5 6 5					Becomes medium dense.			
							SP					
	4			6 6 6		10						
											ATD ▼	
	5		37	5 4 4		15			Becomes loose and wet, trace silt.			
									Medium dense, gray, fine SAND; wet. (ALLUVIUM)			
	6		31	6 6 8		20						
	7			4 4 3		25	SP		Becomes loose.			
FC	8		33	4 4 6		30			Scattered seams of gray organic silt with organic debris.			
									Medium stiff, dark brown, organic SILT, trace clay; moist.			
	9		100	2 3 5		35	OL					
									Interbedded, loose to medium stiff, dark gray, clayey SILT and fine sandy SILT to silty fine SAND; moist to wet; scattered organic debris and fine sand			

Surface Elevation:
Date Completed: 11/30/99
Logged By: KJL
Equipment: Mobile B-59
Drilling Method: Mud Rotary
Hammer System: 140# Rope and Cathead

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Portland, Oregon

Beacon Engineering

LOG OF BORING B-1

SHEET 1 OF 2

Project No. 097-001

FIGURE A-1

Other Tests	Sample No.	Dry Density (pcf)	Moisture Content (%)	Blows/6 inches	Sample	Depth (feet)	USCS	Graphic Symbol	DESCRIPTION	Elevation (feet)	Groundwater	Well Detail
	10		87	2 2 3	◧				layers.			
	11			3 3 5	◧	45			Mottled dark brown and green-gray, grading geen gray at 46 feet.			
	12			5 5 10	◧	50						
									Borehole backfilled with soil cuttings and bentonite chips.			
						55						
						60						
						65						
						70						
						75						

Date Completed: 11/30/99
Logged By: KJL
Equipment: Mobile B-59
Drilling Method: Mud Rotary
Hammer System: 140# Rope and Cathead

PACRIM GEOTECHNICAL INC.
GEOTECHNICAL ENGINEERING AND APPLIED EARTH SCIENCES

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Beacon Engineering

LOG OF BORING B-1

SHEET 2 OF 2

Project No. 097-001

FIGURE A-1

Other Tests	Sample No.	Dry Density (pcf)	Moisture Content (%)	Blows/6 inches	Sample	Depth (feet)	USCS	Graphic Symbol	DESCRIPTION	Elevation (feet)	Groundwater	Well Detail
							GP		Loose, brown to black, sandy GRAVEL; moist. (ARTIFICIAL FILL)			
									Medium dense, brown, fine SAND; dry to moist.			
	1		27	5 4 7	◨							
						5						
	2			4 4 6	◨							
									Becomes loose.			
	3		28	4 4 4	◨							
						10						
	4			4 4 4	◨		SP					
											ATD ▼	
						15						
MA	5		38	5 3 5	◨				Becomes wet, trace silt, scattered dark gray silt partings.			
						20						
	6			7 6 5	◨							
									Loose, gray, fine SAND; wet; scattered organic debris and dark gray silt seams with organics. (ALLUVIUM)			
						25						
MA	7		34	4 3 5	◨							
							SP		Becomes medium dense.			
						30						
	8		30	8 8 10	◨							
									Medium stiff, brown, organic SILT with fine sand; moist to wet; abundant organic debris.			
						35						
FC, Cons	9		103	2 2 3	◨							
							OL					

Surface Elevation:
Date Completed: 11/30/99
Logged By: KJL
Equipment: Mobile B-59
Drilling Method: Mud Rotary
Hammer System: 140# Rope and Cathead

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LOG OF BORING B-2

SHEET 1 OF 2

Project No. 097-001

FIGURE A-2

RELATIVE DENSITY OR CONSISTENCY VERSUS SPT N-VALUE

COHESIONLESS SOILS			COHESIVE SOILS		
Density	N (blows/ft)	Approximate Relative Density (%)	Consistency	N (blows/ft)	Approximate Undrained Shear Strength (psf)
Very Loose	0 to 4	0 - 15	Very Soft	0 to 2	<250
Loose	4 to 10	15 - 35	Soft	2 to 4	250 - 500
Medium Dense	10 to 30	35 - 65	Medium Stiff	4 to 8	500 - 1000
Dense	30 to 50	65 - 85	Stiff	8 to 15	1000 - 2000
Very Dense	over 50	85 - 100	Very Stiff	15 to 30	2000 - 4000
			Hard	over 30	>4000

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			GROUP DESCRIPTIONS	
Coarse Grained Soils More than 50% Retained on No. 200 Sieve Size	**Gravel and Gravelly Soils** More than 50% of Coarse Fraction Retained on No. 4 Sieve	Clean Gravel (little or no fines)	GW	Well-graded GRAVEL
			GP	Poorly-graded GRAVEL
		Gravel with Fines (appreciable amount of fines)	GM	Silty GRAVEL
			GC	Clayey GRAVEL
	Sand and Sandy Soils 50% or More of Coarse Fraction Passing No. 4 Sieve	Clean Sand (little or no fines)	SW	Well-graded SAND
			SP	Poorly-graded SAND
		Sand with Fines (appreciable amount of fines)	SM	Silty SAND
			SC	Clayey SAND
Fine Grained Soils 50% or More Passing No. 200 Sieve Size	**Silt and Clay**	Liquid Limit Less than 50%	ML	SILT
			CL	Lean CLAY
			OL	Organic SILTor CLAY
	Silt and Clay	Liquid Limit 50% or More	MH	Elastic SILT
			CH	Fat CLAY
			OH	Organic SILTor CLAY
Highly Organic Soils			PT	PEAT

DESCRIPTORS FOR SOIL STRATA AND STRUCTURE

General Thickness or Spacing		Structure		General Attitude	
Parting:	less than 1/16 in.	Pocket:	Erratic, discontinuous deposit of limited extent	Near horizontal:	0 to 10 deg.
Seam:	1/16 to 1/2 in.			Low angle:	10 to 45 deg.
Layer:	1/2 to 12 in.	Lens:	Lenticular deposit	High angle:	45 to 80 deg.
Stratum:	greater than 12 in.	Varved:	Alternating seams of silt and clay	Near vertical:	80 to 90 deg.
Scattered:	less than 1 per ft.	Laminated:	Alternating seams		
Numerous:	more than 1 per ft.	Interbedded:	Alternating layers		

Notes:

1. Sample descriptions in this report are based on visual field and laboratory observations, which include density/consistency, moisture condition, grain size, and plasticity estimates, and should not be construed to imply field nor laboratory testing unless presented herein. Visual-manual classification methods of ASTM D 2488 were used as an identification guide. Where laboratory data are available, soil classifications are in general accordance with ASTM D 2487.

2. Solid lines between soil unit descriptions indicate change in interpreted geologic unit. Dashed lines indicate stratigraphic change within the unit.

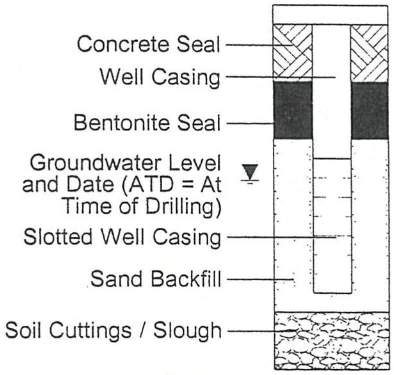
LABORATORY TEST SYMBOLS

AL	Atterberg Limits
FC	Fines Content
GSD	Grain Size Distribution
MC	Moisture Content
MD	Moisture Content/Dry Density
Comp	Compaction Test (Proctor)
SG	Specific Gravity
CBR	California Bearing Ratio
RM	Resilient Modulus
Perm	Permeability
TXP	Triaxial Permeability
Cons	Consolidation
VS	Vane Shear
DS	Direct Shear
UC	Unconfined Compression
TXS	Triaxial Compression
UU	Unconsolidated, Undrained
CU	Consolidated, Undrained
CD	Consolidated, Drained

SAMPLE TYPE SYMBOLS

▯	Std. Penetration Test (2.0" OD)
☒	Ring Sampler (3.25" OD)
⧗	California Sampler (3.0" OD)
■	Undisturbed Tube Sample
G	Grab Sample
^	Core Run
▯	Non-standard Penetration Test (with split spoon sampler)

GROUNDWATER WELL COMPLETIONS



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KEY TO EXPLORATION LOGS

Project No. 097-001 **FIGURE A-3**

APPENDIX B

LABORATORY TESTING

PacRim personnel, and our subcontract Laboratory – Rosa Environmental and Geotechnical Labs, performed laboratory soil tests in general accordance with appropriate ASTM test methods. We tested selected soil samples to determine moisture content, grain size distribution, fines content, Atterberg Limits, and consolidation characteristics. The test procedures and test results are discussed below.

MOISTURE CONTENT

Laboratory tests were conducted to determine the moisture content of selected soil samples, in general accordance with ASTM D-2216. Test results are indicated at the sampled intervals on the appropriate boring logs in Appendix A.

GRAIN SIZE DISTRIBUTION

Grain size distribution was determined for selected samples in general accordance with ASTM D-422. Results of grain size analyses are plotted on Figure B-1.

FINES CONTENT

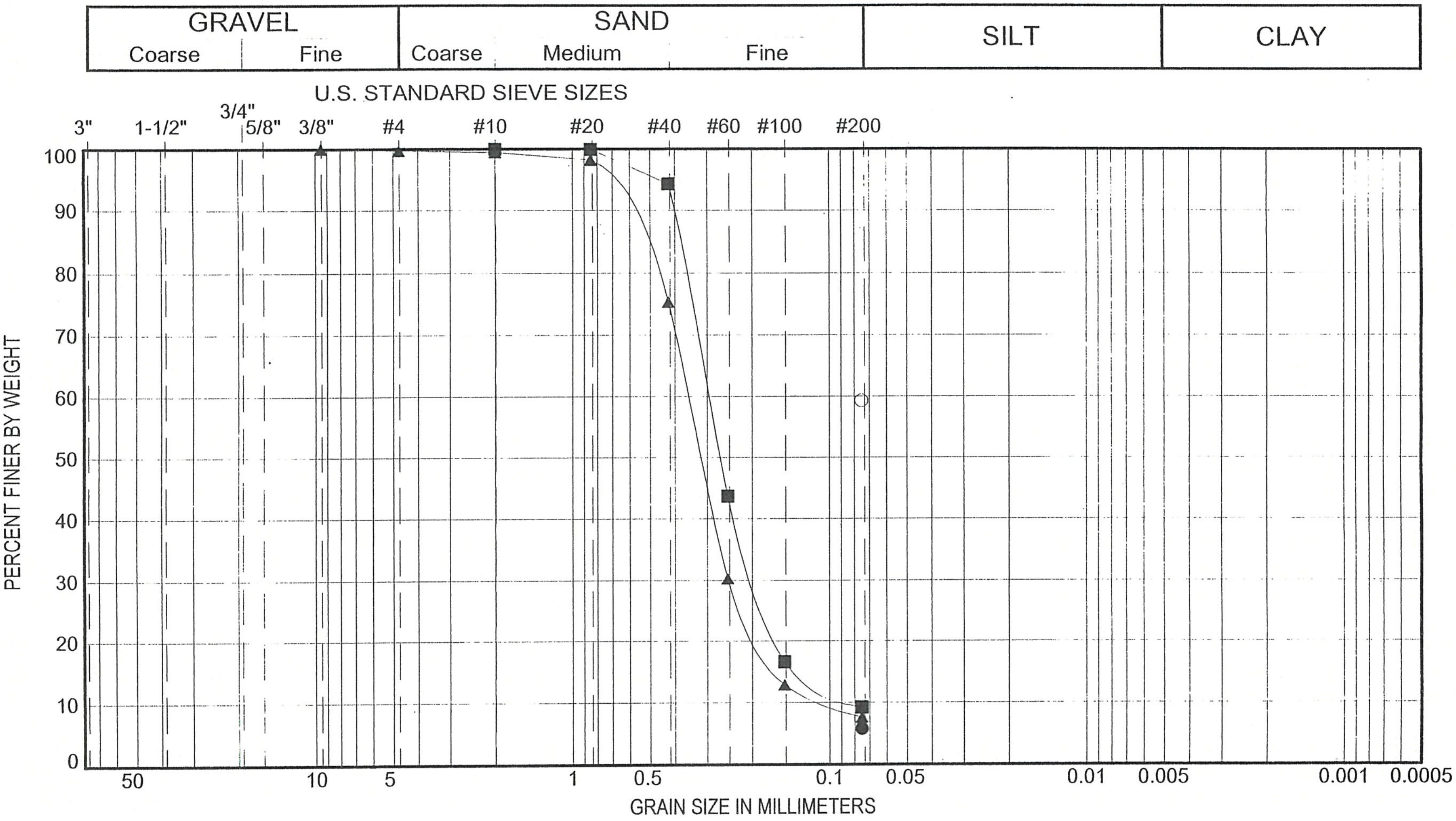
Selected soil samples were washed through the U.S. Standard No. 200 sieve in general accordance with ASTM D 1140 to determine the percentage of fines. The test results are plotted on the Grain Size Distribution plot Figure B-1.

ATTERBERG LIMITS

Atterberg Limits indices were determined for sample S-11in Boring B-2-99. The test was performed in general accordance with ASTM D-4318. The results are shown on Figure B-2.

CONSOLIDATION TEST

Consolidation characteristics were determined for sample S-10 in Boring B-2-99. The test was performed in general accordance with ASTM D-2435. The test results are shown on Figure B-3.



SYMBOL	SAMPLE	DEPTH (ft)	CLASSIFICATION	% MC	LL	PL	PI	% Gravel	% Sand	% Fines
●	B-1 8	30.0 - 31.5	Gray, fine SAND with silt (SP)	33						6
■	B-2 5	15.0 - 16.5	Brown, fine SAND with silt (SP-SM)	38				0	91	9
▲	B-2 7	25.0 - 26.5	Black, fine to medium SAND with organics and silt (SP-SM)	34				0	92	8
○	B-2 9	35.0 - 36.5	Brown, orgainc SILT with fine sand (OL)	103						59

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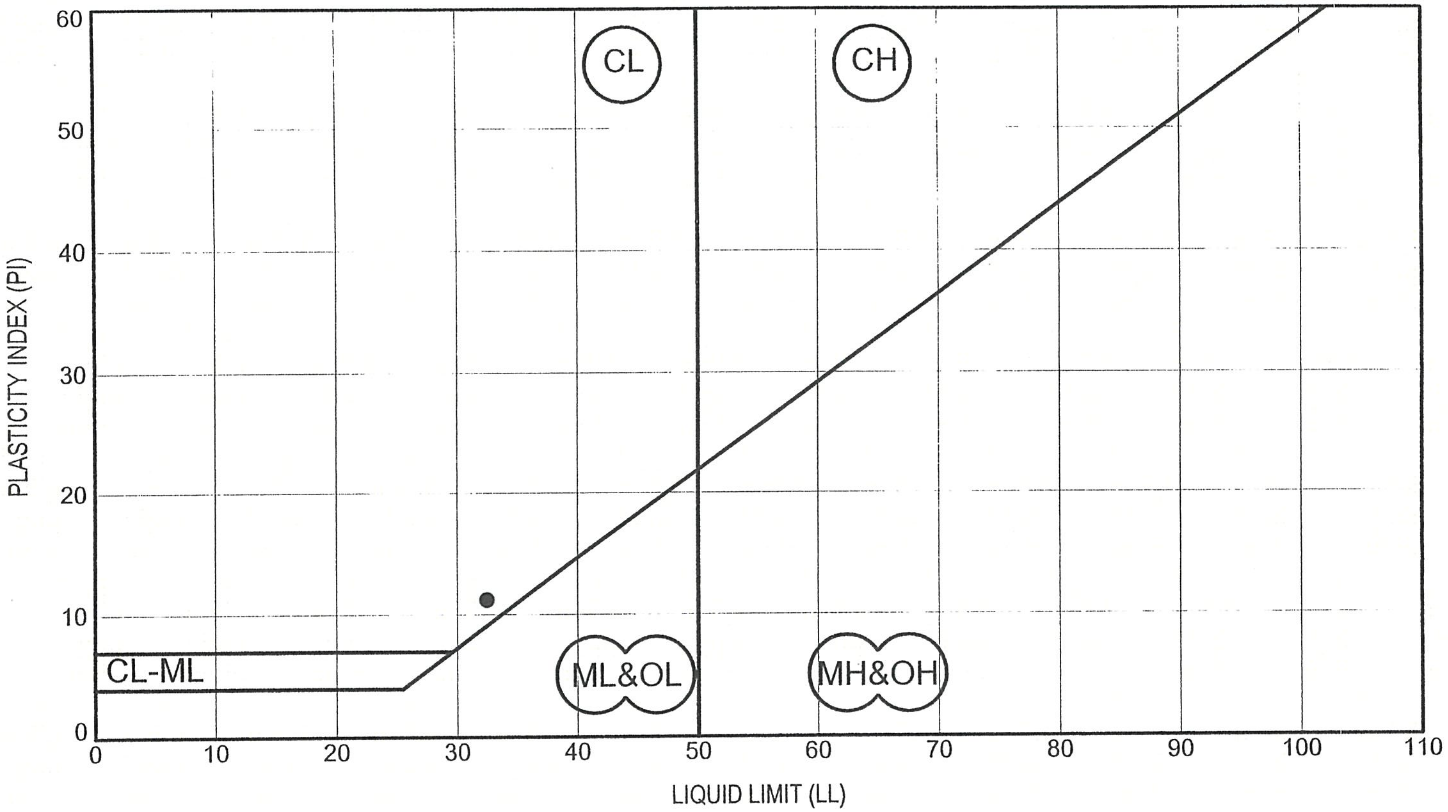
GRAIN SIZE ANALYSIS TEST RESULTS

Project No. 097-001

FIGURE 1



SYMBOL	SAMPLE		DEPTH (ft)	CLASSIFICATION	% MC	LL	PL	PI	% Fines
●	B-2	11	42.0 - 43.5	Gray, silty CLAY (CL)		32	21	11	

PACRIM GEOTECHNICAL INC.

GEOTECHNICAL ENGINEERING AND APPLIED EARTH SCIENCES

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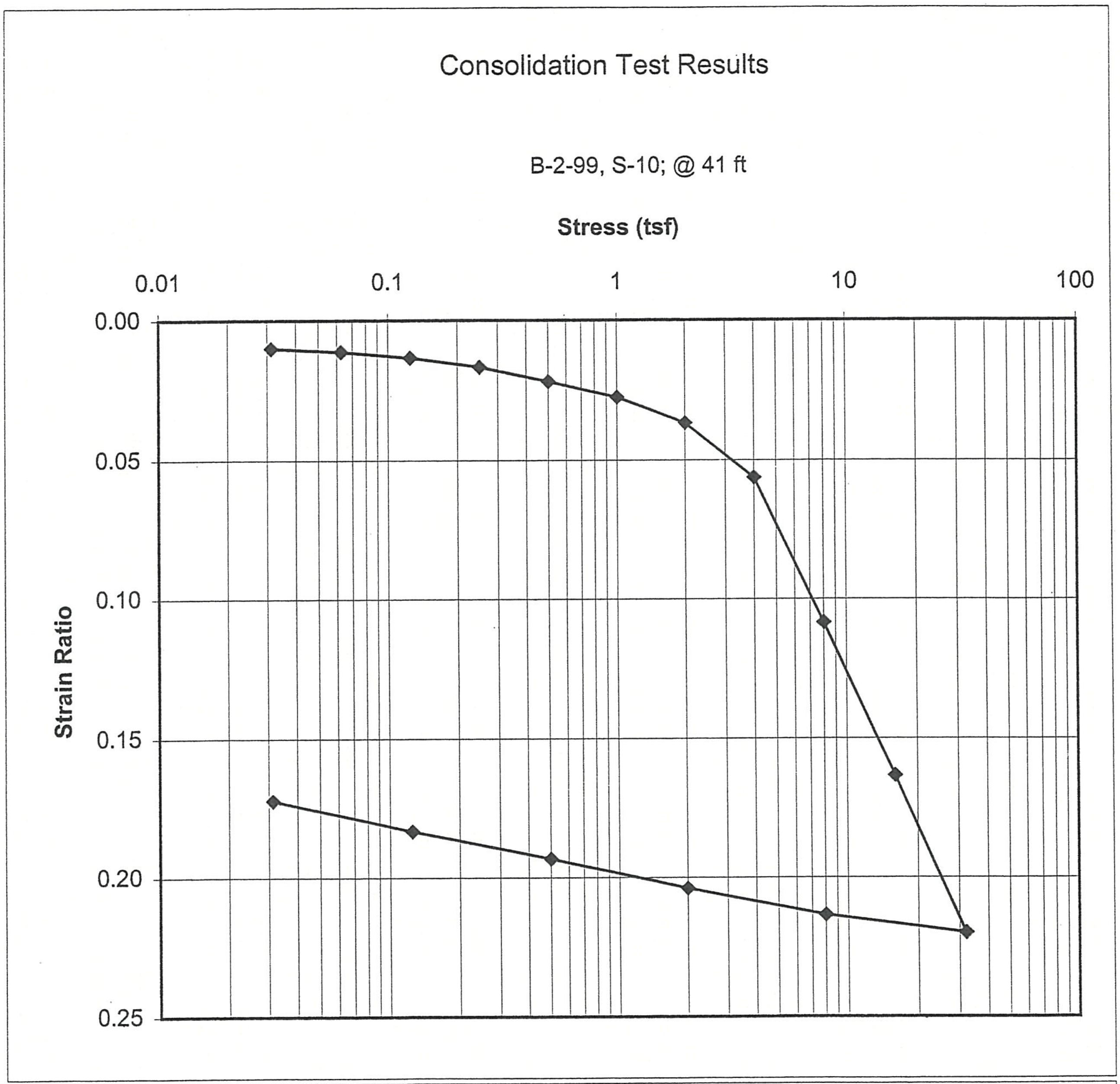
ATTERBERG LIMITS TEST RESULTS

Project No. 097-001

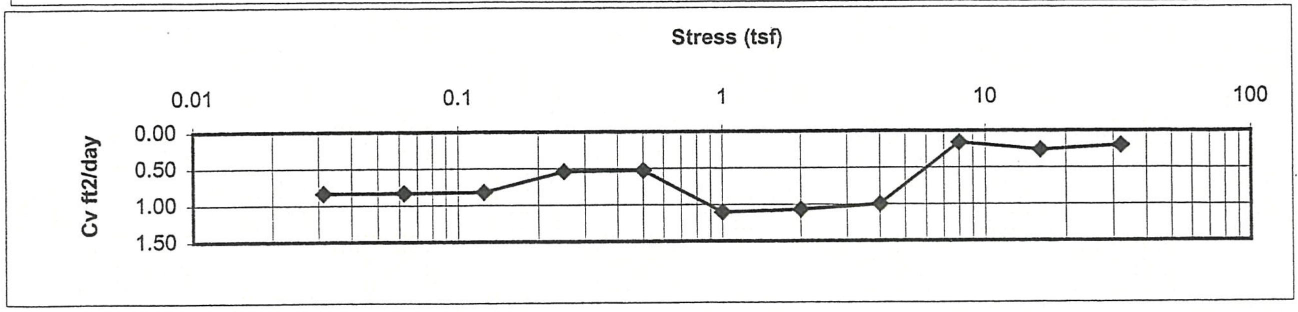
FIGURE 1

Rosa Environmental and Geotechnical Laboratory, LLC

PacRim Geotechnical, Inc.
097-001


Consolidation Test Results

B-2-99, S-10; @ 41 ft



Geotechnical Engineering Report
McCall Companies Terminal Upgrades
5480 Northwest Front Avenue
Portland, Oregon

January 12, 2023

Prepared for

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Geotechnical Engineering Report
McCall Companies Terminal Upgrades
5480 Northwest Front Avenue
Portland, Oregon

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Date: January 13, 2023
Project No.: 1374.022.010
File Path: o:\1374\022.010\r\final geotechnical engineering report
Project Coordinator: Katie Endter

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01/12/2023

LANDAU
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FIGURES

APPENDICES

LIST OF ABBREVIATIONS AND ACRONYMS

AREMA	American Railway Engineering and Maintenance-of-Way Association
ASCE	American Society of Civil Engineers
ASTM	ASTM International
bgs	below ground surface
CIDH	cast-in-drilled-hole
CPT	cone penetrometer test
Ecology	Washington State Department of Ecology
FS	factor of safety
ft	foot/feet
ft^2	square foot/feet
g	force of gravity
IBC	International Building Code
ksf	kips per square foot
Landau	Landau Associates, Inc.
MDD	maximum dry density
Norwest	Norwest Engineering
PGA	peak ground acceleration
psf	pounds per square foot
Qaf	artificial fill
Qal	quaternary alluvium
QC	quality control
SBTn	normalized soil behavior type
Tgsb	Sentinel Member flood basalts
WAC	Washington Administrative Code

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1.0 INTRODUCTION

This report summarizes the results of geotechnical engineering services provided by Landau Associates, Inc. (Landau) in support of the McCall Terminal Upgrades project located at 5480 Northwest Front Avenue in Portland, Oregon (site; Figure 1).

This report has been prepared with information provided by Norwest Engineering (Norwest; prime engineer) and data collected during Landau's geotechnical field investigation and laboratory testing programs.

1.1 Project Understanding

McCall Companies (McCall, project owner) proposes to upgrade its Portland-based terminal. The project primarily consists of a new alignment (see attached figures) of pipes generally running from the dock to the southwest side of the property near Northwest Front Avenue where a new rail spur and pipe racks are proposed. Mechanical equipment will be installed at grade at various points along the alignment. The piping is near grade within the tank farm area but will either become elevated overhead or buried below grade as it continues south towards the proposed rail spur. We understand the product to be renewable diesel fuel. The scope of work outlined herein includes geotechnical exploration and engineering in support of upgrades.

1.2 Scope of Services

Norwest retained Landau's services to support design of the McCall Terminal Upgrades project. Services were provided in accordance with the scope outlined in Landau's June 16, 2022 proposal.

2.0 GEOLOGIC AND SUBSURFACE CONDITIONS

The site is located within the McCall Portland terminal along the western banks of the Willamette River. The site is used to process and transport vegetable oil feedstock and renewable diesel along with offering vessel services and marine fueling.

The following sections describe the geologic setting of the site and the surrounding area, as well as the surface and subsurface conditions observed during Landau's field investigation. Interpretations of site conditions are based on a review of available geologic and geotechnical information and on the results of Landau's site reconnaissance, subsurface explorations, and laboratory testing.

2.1 Geologic Setting

Geologic information for the site was obtained from the *Preliminary Geologic Map of the Linnton 7.5' Quadrangle, Multnomah and Washington Counties, Oregon* (Madin et al. 2008). Near surface soil at the site is mapped as both artificial fill (Qaf) and quaternary alluvium (Qal) underlain by the Grande Basalt Sentinel Bluffs member of the Columbia River Basalt (Tgsb; Madin, I.P., 2004 and 2008). Field investigation results were found to be consistent with the local geologic mapping in the area, and which are described below:

- **[Qaf] artificial fill (Anthropocene)** — man-made deposits of mixed clay, silt, sand, gravel, debris, and rubble.
- **[Qal] quaternary alluvium (Holocene)** — gravel, sand, silt, and clay deposited in active channels and on floodplains of rivers and streams. The age of the alluvium in most streams is Holocene, as most of the streams at lower elevations were affected by the latest Pleistocene Missoula floods, and any alluvial deposits must postdate the floods.
- **[Tgsb] Grande Ronde Basalt Sentinel Bluffs Member, Columbia River Flood Basalt Group (middle Miocene**) — black basalt flows with sparse plagioclase phenocrysts. The lava is typically dark gray or black where fresh, weathering to grayish brown. Sparse plagioclase phenocrysts up to ½-inch occur. The flows typically are blocky to platy jointed and are typically highly vesicular near the flow tops with horizontal bands of flattened vesicles and vugs. The lava weathers to form rounded core stones up to 1.5 ft in diameter. The typical thickness of the unit in the map area is approximately 130 ft.

2.2 Surface Conditions

The site is relatively flat; secondary containment berms surround above ground storage tanks on the northern side of the property. The southern portion of the site is developed with single- and multi-story office buildings; equipment pads; and aboveground product storage, processing, and transport facilities.

2.3 Subsurface Explorations

On August 10 and 11, 2022, Landau's drilling subcontractor advanced two geotechnical borings (B-1 and B-2) approximately 80 to 91 feet (ft) below ground surface (bgs). Six cone penetration tests (CPTs) soundings were advanced 76 to 85 ft bgs (CPT-1 through CPT-6). Of the CPT soundings, seismic wave velocity measurements were collected in CPT-1 and CPT-6. Summary boring logs and CPT results are provided in Appendix A.

Landau personnel monitored the explorations, collected representative soil and core samples, and maintained detailed logs of the subsurface soil and groundwater conditions observed. A single core sample was collected in B-1 from 76 to 91 ft bgs, which correlated with the vesicular upper contact of the Sentinel Member flood basalts (Tgsb). Subsurface conditions were described using the soil classification system provided in Appendix A, and in general accordance with ASTM International (ASTM) standard D2488, *Standard Practice for Description and Identification of Soils (Visual-Manual Procedures)*.

Laboratory test results are provided in Appendix B. The laboratory testing program was performed in accordance with the ASTM standard test procedures also noted in Appendix B. Field log descriptions were checked against the samples and updated, where appropriate, in accordance with ASTM standard D2487, *Standard Classification of Soils for Engineering Purposes (Unified Soil Classification System)*.

2.3.1 Soil Conditions

The following soil classifications are based on an empirical correlation of the CPT data to normalized soil behavior type (SBTn, Robertson 1990) and on data collected from standard penetration test samples and laboratory test results.

Soils observed underlying existing surface conditions (i.e., asphalt and/or gravel surfacing) can be categorized into three general units:

- **Fill:** Fill was observed beneath existing surface conditions in all the explorations. The fill typically consisted of sand with variable amounts of silt. Based on the conditions observed in Landau's explorations, the fill is estimated to extend to approximately 13 ft bgs.
- **Alluvium**: Alluvium was observed beneath the fill in all the explorations to depths of 76 and 84 ft bgs. The alluvium typically consisted of very soft to medium-stiff silt or very loose to loose sand with variable amounts of silt. The lower limit of the alluvium was observed as a sharp contact with the underlying basalt deposits. All CPT soundings terminated in this unit.
- **Basalt**: Basalt was observed in both B-1 and B-2 during drilling. A 15-ft run of HQ core was collected on B-1 from 76 to 91 ft bgs. The basalt consisted of fresh, fine-grained, vesicular basalt with slight secondary mineralization within the vesicles. B-2 was terminated at 80 ft bgs after approximately 4 ft of rotary drilling past the basalt contact.

2.3.2 Groundwater Conditions

Depth to groundwater was not determined in the mud rotary borings. Pore pressure dissipation tests indicate depth to groundwater on the order of 16 to 23 ft bgs. The groundwater information reported herein and on the summary logs in Appendix A is for the specific locations and dates indicated and may not be representative of other locations and/or dates. Groundwater conditions will vary depending on local subsurface conditions, weather conditions, and other factors. Groundwater depth is likely influenced by the water surface elevation in the Willamette River. For engineering analyses and liquefaction calculations, Landau assumed a groundwater depth of 13 ft based on the site elevation relative to typical river elevation.

3.0 SEISMIC DESIGN CONDITIONS

The following sections outline seismic design considerations for the project. We understand that McCall may seek a code exemption to seismic design requirements for at-grade pipin, mechanical ancillaries, and incidentally occupied enclosures.

Based on the results of the seismic analysis for the site and the requirements in American Society of Civil Engineers (ASCE) 7-16, Landau recommends delineating the site into two zones (Zone A and Zone B; see Figure 2) based on estimated lateral spreading.

Foundations in Zone A are estimated to be subject to more than 1 ft of lateral spreading and therefore deep foundations or ground improvements are required (sections 12.13.9 and 15.4.10 of ASCE 7-16) if a code exemption is not applicable.

Lateral spreading in Zone B is estimated to be less than 1 ft; therefore, foundations may consist of shallow foundations provided that the estimated seismic differential settlement is tolerable.

3.1 Seismic Hazards

The site is located in the seismically active Pacific Northwest and could be subject to ground shaking from a moderate to major earthquake. Earthquake shaking is anticipated during the design life of the proposed improvements; structures should be designed to resist earthquake loading. Additionally, site subsurface soils could liquefy during a design-level earthquake.

The saturated soils are prone to seismic liquefaction. A design-level earthquake is estimated to result in less than 1 ft to 15 ft of lateral spreading, depending on proximity to the shoreline, and up to 24 inches of liquefaction-induced settlement.

3.2 Seismic Design Parameters

The seismic design parameters summarized in Table 1 were determined in accordance with ASCE 7-16 and the 2022 Oregon Structural Specialty Code.

Table 1. 2022 Oregon Structural Specialty Code Seismic Design Parameters

Spectral response acceleration at short periods (S_s) = 0.891g
Spectral response acceleration at 1-second periods (S_1) = 0.404g
Site class = D
Site coefficient (F_a) = 1.144
Site coefficient (F_v) = 1.896$^{\text{Note 1}}$
Site modified peak ground acceleration (PGA × F_{PGA}) = 0.482g
Mean moment magnitude (M) = 7.82

Notes:
1. A site-specific ground motion analysis (Chapter 21 of the ASCE *Minimum Design Loads and Associated Criteria for Buildings and Other Structures [ASCE/SEI 7-16]*) may be required to determine F_V in areas with a Site Class D designation and S_1 values greater than or equal to 0.2g. However, the value provided above may be used to compute T_S and Exception No. 2 of Section 11.4.8 of ASCE 7-16 may be used in lieu of a site-specific ground motion analysis. Landau should be notified to perform a site-specific ground motion analysis if the exception is not applicable.
2. Site is classified as "F" due to liquefaction. However, for structures with fundamental periods less than 0.5 second, ASCE 7-16 permits design of structures considering the site class if liquefaction were not to occur. Contact Landau if proposed structures have a fundamental period greater than 0.5 second.

g = force of gravity

3.3 Liquefaction and Cyclic Softening

Liquefaction is defined as a significant rise in pore water pressure within a soil mass caused by earthquake-induced cyclic shaking. The shear strength of liquefiable soil is reduced during large and/or long-duration earthquakes as soil consistency approaches that of a semi-solid slurry. If not properly mitigated, loss of soil shear strength can result in significant, widespread structural damage. Deposits of loose, granular soil below the water table are most susceptible to liquefaction. Damage caused by foundation rotation, slope failure, seismically induced settlement, and reduced bearing capacity is frequently observed in areas where liquefaction has occurred.

Cyclic softening is the shear strength degradation of very soft to soft plastic silts and clays. Cyclic softening can reduce soil strength and cause damage similar to – though typically less severe than – that caused by liquefaction.

Landau completed a liquefaction susceptibility screening based on Bray and Sancio (2006) for the soil borings and using an I_c (Robertson 1990) cutoff value of 3.0 for the CPT data (I_c > 3.0 not susceptible to liquefaction). Landau selected the I_c cutoff value based on comparison to site-specific Atterberg limits and grain size analyses.

The factor of safety (FS) against liquefaction for layers of soil determined to be susceptible to liquefaction was performed using the software CLiq Version 3.0 (Geologismiki 2022) on the CPT soundings, and LiqSVs Version 2 (Geologismiki 2022) for the standard penetration tests. FSs were calculated in accordance with Boulanger and Idriss (2014).

The results of Landau's analysis indicate that saturated portions of fill and the interbedded layers of the alluvium (between 13 and approximately 80 ft bgs) have an FS (against liquefaction/cyclic softening) of less than 1.2 and would be susceptible to liquefaction or cyclic softening during a design-level event. Of these layers, most appear susceptible to liquefaction.

3.4 Liquefaction-Induced Settlement

Considering the cumulative effect of all potentially liquefiable soil layers, liquefaction-induced, free-field ground settlement could approach 24 inches. Shallow foundations installed on potentially liquefiable soil layers are estimated to settle by as much as 24 inches, with up to 18 inches of differential settlement over 50 ft.

3.5 Lateral Spreading

Lateral spreading is the finite, lateral movement of gently to steeply sloping, saturated soil caused by seismically-induced liquefaction. Lateral spreading can also occur on level ground near shoreline slopes. The project footprint is located along the banks of the Willamette River

Landau estimated lateral spread displacements using the Youd et al. (2002) and Zhang et al (2004) empirical methods. Based on these estimates, Landau concludes that areas of the site with L/H ratios (ratio of distance from free face to point of interest [L] to height of free face [H]) greater than 20, or more than 1,000 ft from the free face, have a negligible risk of lateral spreading (i.e., less than 1 ft). This is the basis for the delineation between Zone A and Zone B is shown on Figure 2. Areas closer to the shoreline (i.e., in Zone A) are estimated to undergo between 1 and 15 ft of lateral deformation, with the higher estimate occurring near the shoreline.

4.0 CONCLUSIONS AND RECOMMENDATIONS

Based on the conditions encountered in the subsurface explorations and the preceding discussion in Section 3 about seismic design considerations, the key considerations when planning the project are as follows:

- For structures in Zone A and anywhere a seismic design code exemption is not applicable, deep foundations or ground improvement may be required.
- For structures in Zone B, shallow foundations may be feasible if the estimated liquefaction-induced settlement is compatible with the structure. Landau understands that structures will be supported on shallow foundations and designed to accommodate the liquefaction-induced differential settlement.

4.1 Foundation Selection

Landau understands that structures in Zone A will receive an exemption regarding foundation design requirements for liquefaction settlement and lateral spreading. As such, structures will be supported on shallow foundations. Landau understands shallow foundations will be used for Zone B structures.

4.2 Shallow Foundations

Shallow foundations can be designed with the following parameters:

Table 2. Shallow Foundation Design Parameters

Footing Width, B	B ≤ 4 ft	4 ft < B ≤ 8 ft	8 ft < B ≤ 12 ft	12 ft < B ≤ 16 ft
Max. Footing Area[1]	16 ft^2	64 ft^2	144 ft^2	256 ft^2
Static Allowable Bearing Pressure[2]	3 ksf	2.5 ksf	1.5 ksf	0.75 ksf
Seismic Allowable Bearing Pressure[3]	4 ksf	3.3 ksf	2 ksf	1 ksf

Notes:
ksf = kips per square foot
Allowable base friction coefficient: 0.35
Minimum footing depth: 2 ft
Minimum footing width: 2 ft

1. For footings larger than 256 ft^2, Landau recommends using a subgrade modulus as described in Section 4.4.
2. Pressures for 1 inch total settlement, ½ inch differential over 50 ft.
3. Shallow foundations will still experience the estimated liquefaction settlement: up to 24 inches total and 18 inches differential over 50 ft.

Shallow foundations should be found on 1 ft of compacted structural fill. Landau should observe the subgrade conditions prior to placement of structural fill. Additional over-excavation may be required if soft or unsuitable materials are encountered.

4.3 Cast-in-drilled-hole Pipe Supports

Landau understands that at-grade piping will be supported on cast-in-drilled-hole (CIDH) foundations. CIDH foundations are anticipated to perform similarly to shallow foundations, and the seismic performance discussion in Section 4.2 also applies to CIDH foundations.

Landau recommends a minimum CIDH diameter of 18 inches, a minimum depth of 5 ft, and a maximum depth of 10 ft. Soils encountered in the top 10 ft are anticipated to consist of loose to medium-dense sand or silty sand and may be prone to caving. The contractor should be prepared to use temporary casing or drilling mud to maintain borehole stability. At the completion of drilling, the bottom of the hole should be cleaned out to remove loose drill slough with a cleanout bucket or similar approach. Concrete should be tremied in at the bottom of the hole after placement of the reinforcement.

The following parameters may be used to size CIDH foundations:

Table 3. Allowable Unit Resistances

End Bearing	3 ksf
Side friction*	0.35 ksf

*Ignore 0 to 5 ft bgs; assumes no permanent casing/formwork.

Table 4. CIDH Foundation Lateral Resistance (assumes shear load applied 3 ft above grade or less and no applied moment)

Diameter*	Depth = 5 ft*	Depth = 10 ft*
18 inches	2.2 kips	10 kips
24 inches	2.5 kips	15 kips
36 inches	3 kips	22 kips

*Designer may interpolate for intermediate depths or diameters. Contact Landau for other loading considerations.

4.4 Slabs-on-grade

Soils beneath slabs-on-grade should be excavated approximately 2 ft below the bottom of the slab and 2 lateral feet from the edge of the slab. The subgrade should be compacted to a firm and unyielding condition and observed by Landau prior to structural fill placement. Additional over-excavation may be required if soft or unsuitable soils are encountered. Excavated soils should be replaced with import structural fill.

Assuming subgrade preparation as described above, a modulus of subgrade reaction of 100 pci for static conditions and 130 pci for seismic conditions should be used to design the slab. This modulus is for a 1-ft x 1-ft loaded area and should be adjusted for the actual slab size. The seismic subgrade modulus does not account for the estimated liquefaction-induced total settlement of up to 24 inches and up to 18 inches differential over 50 ft.

4.5 Settlement Considerations

Shallow foundations, CIDH pipe supports, and slabs-on-grade are anticipated to experience up to 24 inches of total liquefaction-induced settlement and 18 inches over 50 ft of differential liquefaction-induced settlement because of a design-level earthquake.

Shallow foundations, CIDH pipe supports, and slabs-on-grade may experience limited static settlement due to the presence of soft soils at depths below the foundation. The cost to remedy this potential issue is likely impractical, provided McCall can accept some settlement of structures. Landau anticipates that new structures found on footings, CIDH foundations, or slabs-on-grade will perform similarly to previously constructed infrastructure at the site that is not supported on deep foundations. Landau estimates that long-term settlements are unlikely to exceed 1 to 2 inches over the lifespan of the improvements.

4.6 Earthwork and Construction Considerations

Landau understands that earthwork for the project will be limited to excavation for foundations and rail spur construction. Rail spur recommendations are contained separately in Section 4.7. Based on this understanding, the following recommendations should be incorporated into the project design:

- Import structural fill should meet the requirements of 2-inch-minus, dense-graded aggregate conforming to the requirements of Section 02630 of the 2021 Oregon Department of Standard Specifications and be compacted to at least 95 percent of the maximum dry density as determined by the modified Proctor.
- Subgrades for shallow foundations and slabs-on-grade should be observed by Landau prior to structural fill placement.
- Structural fill density should be tested by the contractor’s third-party quality control (QC) testing firm under the direction of an Oregon registered professional engineer. The testing firm should submit a stamped report documenting that structural fill compaction has met the requirements of the project. Testing results should be submitted to Norwest or Landau for review prior to pouring foundations.

4.7 Rail Spurs – Track Support

Based on the subsurface conditions observed in Landau’s explorations, site soils are anticipated to provide adequate support for the new single-track rail spur, provided the soils are prepared as recommended herein. Some long-term settlement of ballast could occur if spurs are constructed on raised grades. Landau understands that this settlement will be mitigated by releveling track as needed.

4.7.1 Rail Earthwork

All earthwork should be completed in general accordance with Chapter 1, Section 1.3.6, of the American Railway Engineering and Maintenance-of-Way Association’s (AREMA) 2019 *Manual for*

Railway Engineering. The following supplemental recommendations should be observed during earthwork construction.

4.7.2 Subgrade Preparation

The prepared subgrade should provide a foundation for the track roadbed or embankment fill that results in acceptable amounts of settlement and track deflection. In general, subsurface soils along the proposed rail alignments consist of loose to medium-dense sand or silty sand. The native soil will provide suitable subgrade support.

Unsuitable material (organic-rich soil, pavement, debris, foundations, etc.) should be removed from all cut, fill, and at-grade areas. Subgrade preparation should extend 3 horizontal feet beyond the edge of the embankment fill or sub-ballast, and 1 vertical foot below the fill or sub-ballast.

Suitable embankment fill material consists of import structural fill as described in Section 4.6 of this report.

4.7.3 Embankment Fill

Embankment fill is fill placed on prepared *in situ* soil to raise subgrade elevations. Landau assumes that embankment fill will be minimal, given the relatively flat site topography and the configuration of existing rail spurs. Moisture-conditioned embankment fill should be placed in 8-inch loose lifts and compacted to at least 95 percent of maximum dry density (MDD).

4.7.4 Maximum Slopes

Embankment fill slopes should be 2H:1V or flatter. When preparing this ratio, Landau assumed that fill slopes would be less than 4 ft tall and would not require seismic design. All cut and fill slopes should be protected from erosion in accordance with Section 1.4.5 of the AREMA 2019 *Manual for Railway Engineering.*

4.7.5 Sub-ballast and Ballast Requirements

The sub-ballast provides separation between the subgrade and ballast and is primarily used in construction of new tracks. The combined sub-ballast and ballast thickness depends on the subgrade strength (allowable bearing pressure) and the uniformly distributed pressure over the rail tie face. An allowable subgrade soil bearing pressure of 2,500 pounds per square foot (psf) can be used to determine the minimum combined depth of ballast and sub-ballast, per Section 2.11.2.3 of the AREMA 2019 *Manual for Railway Engineering*. For the compacted ballast and sub-ballast, a minimum depth of 12 inches each (combined 24 inches) is recommended for standard gauge construction.

Ballast should consist of material graded to meet the requirements for AREMA No. 4 as specified in section 2.4 of the AREMA 2019 *Manual for Railway Engineering*. In general, ballast should conform to the requirements in Section 2.3.1 of the AREMA 2019 *Manual for Railway Engineering*. Sub-ballast should consist of granular material that meets the requirements of structural fill, as defined in Section 4.6 of this report.

5.0 DOCUMENT REVIEW AND CONSTRUCTION SUPPORT

Landau should review geotechnical portions of the plans and specifications in advance of project bidding to verify that the recommendations presented herein have been properly interpreted and implemented.

Monitoring, testing, and consultation should be provided during construction to confirm that subsurface conditions are consistent with those indicated by Landau’s explorations; to provide expedient recommendations should conditions differ from those anticipated; and to evaluate whether geotechnical activities comply with project plans, specifications, and the recommendations in this report.

6.0 USE OF THIS REPORT

Landau Associates, Inc. (Landau) has prepared this technical memorandum for the exclusive use of Norwest Engineering and its client, McCall Terminals LLC, for specific application to the McCall Terminal Upgrades project. No other party is entitled to rely on the information, conclusions, and recommendations included in this document without the express written consent of Landau. Reuse of the information, conclusions, and recommendations provided herein for extensions of the project or for any other project, without review and authorization by Landau, shall be at the user's sole risk. Landau warrants that, within the limitations of scope, schedule, and budget, its services have been provided in a manner consistent with that level of skill and care ordinarily exercised by members of the profession currently practicing in the same locality under similar conditions as this project. Landau makes no other warranty, either express or implied.

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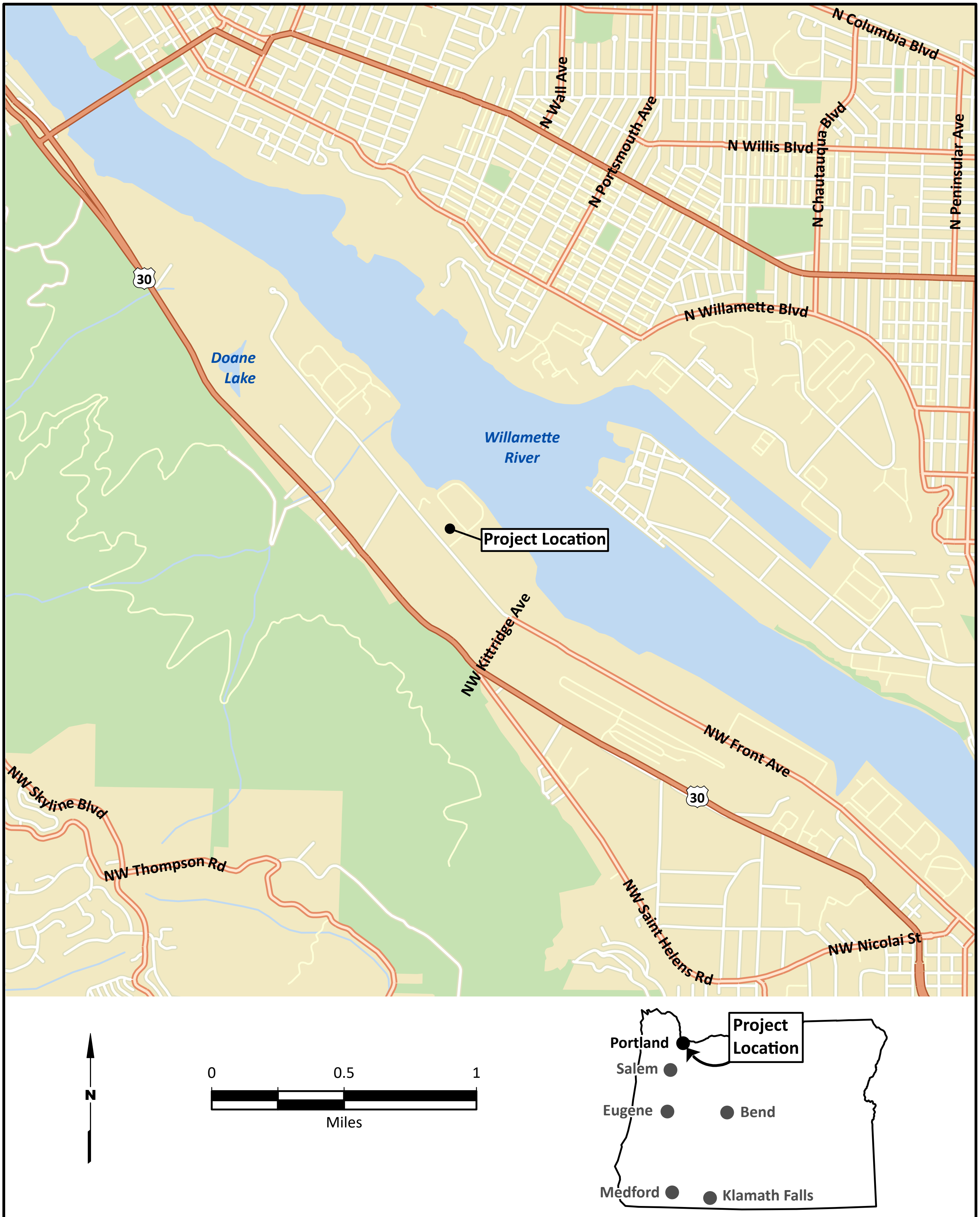

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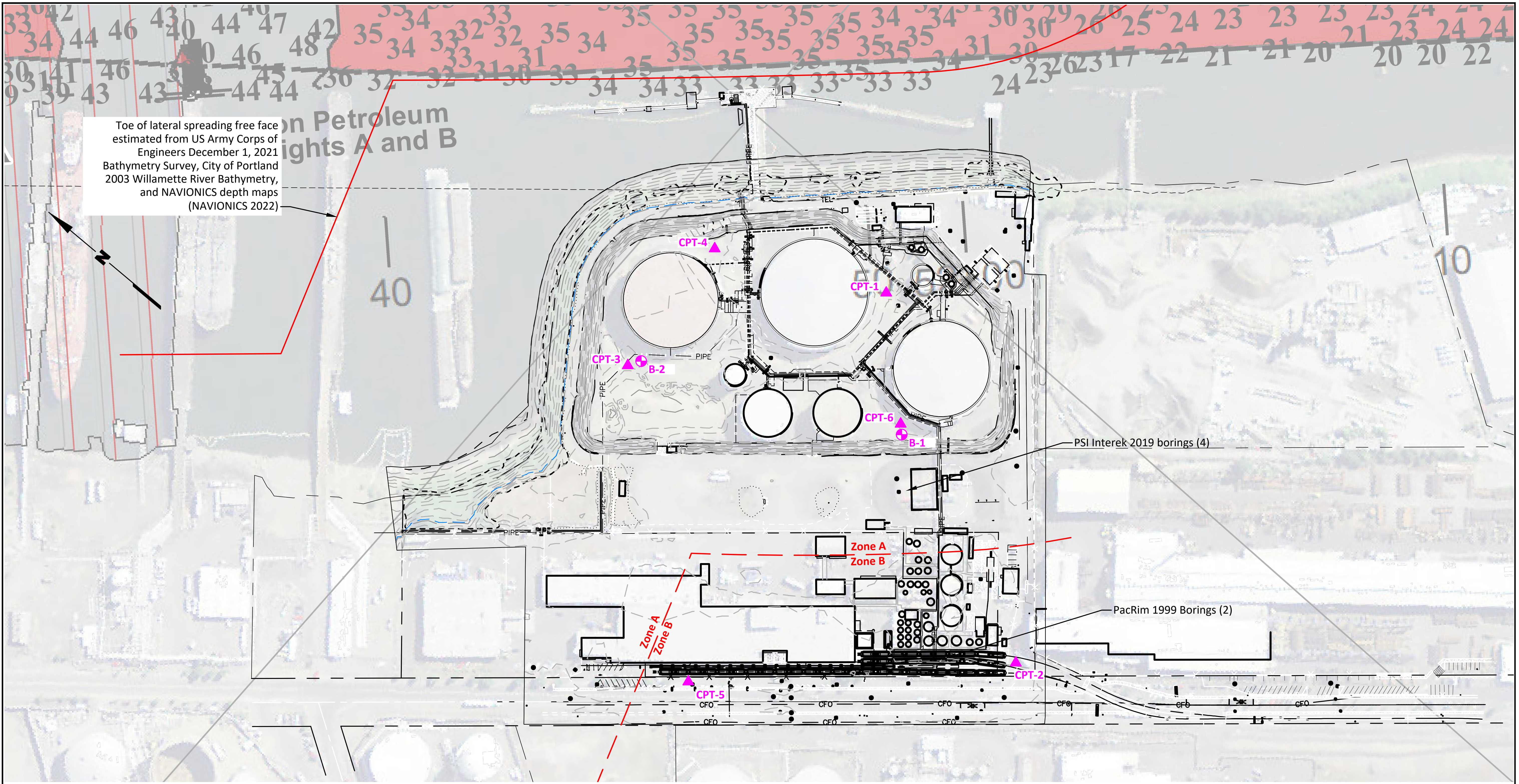
LANDAU ASSOCIATES

McCall Terminal Upgrades
Portland, Oregon

Vicinity Map

Figure
1

Landau Associates | O:\1374\022.010\T\CAD\1374022_site Plan.dwg | 9/28/2022 7:45 AM | dsimpson



Legend

B-1 ◑ Boring Location and Designation

CPT-1 ▲ Cone Penetration Test Location and Designation

Notes

1. Black and white reproduction of this color original may reduce its effectiveness and lead to incorrect interpretation.
2. River depth shown is depth in feet below Columbia River Datum, topography shown is City of Portland Datum.
3. Refer to report text for additional explanation.

Source: Topography (AKS Engineering & Forestry 2022), Imagery and River Depth (USACE 2021)

0 200 400
Scale in Feet

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McCall Terminal
Improvements Project
Portland, Oregon

Site and Exploration Plan

Figure
2

APPENDIX A

Field Explorations

APPENDIX A
FIELD EXPLORATIONS

Site subsurface conditions were explored on August 10 and 11, 2022. The exploration program included six cone penetration test (CPT) soundings (CPT-1 through CPT-6) and two geotechnical borings (B-1 and B-2) at the approximate locations shown on Figure 2. CPT-1 was advanced 85.3 feet (ft) below ground surface (bgs), CPT-2 78.6 ft bgs, CPT-3 47 ft bgs, CPT-3 76 ft bgs, CPT-4 83.5 ft bgs, and CPT-6 76.6 ft bgs.

B-1 and B-2 were advanced 91 ft bgs and 80 ft bgs respectively. The exploration locations were sited by measuring from existing infrastructure. Ground surface elevations at the exploration locations were not determined.

The CPT soundings were advanced by Oregon Geotechnical Explorations of Keizer, Oregon, subcontracted by Landau Associates, Inc. (Landau). The geotechnical borings were advanced by Holt Services and subcontracted by Landau.

The field exploration program was coordinated and monitored by Landau personnel, who also obtained representative soil and core samples, maintained a detailed record of the subsurface soil and groundwater conditions observed, and described both the soil and rock encountered by visual and textural examination. Each representative soil type was described using the soil classification system shown on Figure A1, in general accordance with ASTM International (ASTM) standard test method D2488, *Standard Recommended Practice for Description of Soils (Visual-Manual Procedure).* Landau personnel collected 15 ft of core run to determine the rock quality designation (RQD) of the core.

Samples collected in this manner were taken to Landau's soils laboratory for further examination and testing. A discussion of laboratory test procedures and the laboratory test results are included in Attachment 2.

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

Summary boring logs are provided on Figures A2 and A3. These logs represent Landau's interpretation of the subsurface conditions identified during the field exploration program. The stratigraphic contacts shown on the summary logs represent the approximate boundaries between soil types; actual transitions may be more gradual. The soil and groundwater conditions depicted are for the specific dates and locations reported and are not necessarily representative of other locations and dates.

Summary logs of the CPT soundings are included at the end of Appendix A.

Soil Classification System

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

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

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

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

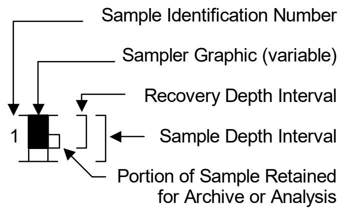
4. Soil density or consistency descriptions are based on judgement using a combination of sampler penetration blow counts, drilling or excavating conditions, field tests, and laboratory tests, as appropriate.

Drilling and Sampling Key

SAMPLER TYPE & METHOD

Graphic	Code	Description
☒	a	3.25-in OD, 2.42-in ID Split Spoon
☒	b	2.00-in OD, 1.50-in ID Split Spoon
◨	c	Shelby Tube
G	d	Grab Sample
◨	e	Single-Tube Core Barrel
◨	f	Double-Tube Core Barrel
☒	g	2.50-in OD, 2.00-in ID WSDOT
☒	h	3.00-in OD, 2.37-in ID Mod. Calif.
☒	i	Other - See text if applicable
	1	300-lb Hammer, 30-inch Drop
	2	140-lb Hammer, 30-inch Drop
	3	Pushed Sample
	4	Vibrocore (Rotosonic/Geoprobe)
	5	Other - See text if applicable
	6	Piston Extraction

SAMPLE NUMBER & INTERVAL



Field and Lab Test Data

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

Groundwater

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

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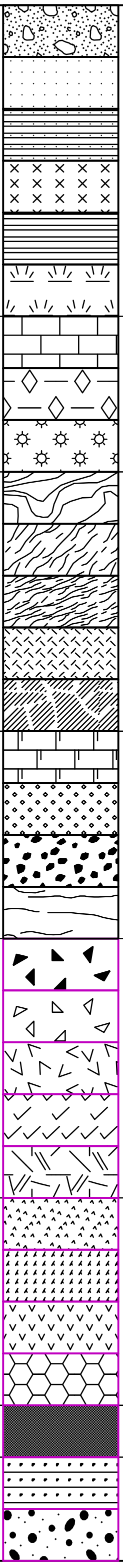
McCall Terminal
Upgrade Project
Portland, Oregon

Soil Classification
System and Key

Figure
A1.1

Rock Classification System

Primary Rock Types



Sedimentary	Clastic	Conglomerate
		Sandstone/ Sedimentary Quartzite
		Siltstone/Graywacke
		Claystone/Mudstone
		Shale
		Coal
	Crystalline	Limestone/Dolomite
		Gypsum/Halite/ Anhydride
		Chert
Metamorphic	Foliated	Gneiss/Schist
		Schist/Talc
		Phyllite
		Slate
		Mylonite
	Massive	Marble
		Quartzite
		Hornfels
		Serpentine/Soapstone/ Greenstone
Igneous	Phaneritic (Individual crystals distinguishable with unaided eye.)	Granite
		Monzonite/ Quartz Monzonite
		Granodiorite/ Diorite Quartz Diorite
		Gabbro
		Diabase/Perioite
	Aphanitic (Crystals not distinguishable or visible crystals in a fine-grained groundmass.)	Rhyolite
		Latite/Quartz Latite
		Dacite/Andesite
		Basalt
	Glassy	Obsidian/ Pumice/Scoria
	Pyroclastic	Agglomerate/ Breccia/Tuff
		Bombs/Blocks/ Cinders/Ash

Relative Hardness

Term	Designation	Approx. Unconfined Compressive Strength	Field Identification
Extremely Soft	R0	< 100 psi	Moldable or friable with finger pressure.
Very Soft	R1	100 - 1,000 psi	Peeled by knife with ease. Crumbles under firm blows with point of a geology pick.
Soft	R2	1,000 - 4,000 psi	Peeled by knife with difficulty. Shallow indentation made by firm blow of geology pick.
Medium Hard	R3	4,000 - 8,000 psi	Scratched by knife with ease. Fractured with a single firm blow of hammer/geology pick.
Hard	R4	8,000 - 16,000 psi	Scratched by knife with difficulty. Several hard hammer blows required to fracture.
Very Hard	R5	> 16,000 psi	Cannot be scratched with knife. Many hard hammer blows required to fracture or chip.

Relative Weathering

Fresh	Crystals are bright; no discoloration in rock fabric
Slightly Weathered	Some discoloration in rock fabric; decomposition extends up to 1 inch
Moderately Weathered	Rock mass is decomposed 50 % or less
Predominately Decomposed	Rock mass is more than 50 % decomposed; can be excavated with pick
Decomposed	Completely decomposed; can be reduced to soil with hand pressure

Structural Descriptions

Spacing (in)	Bedding/Foliation	Joint/Shear/Fracture	Attitude and Angle
< 2	Very Thin	Very Close	Horizontal (0-5°)
2 - 12	Thin	Close	Shallow or Low Angle (5-35°)
12 - 36	Medium	Moderately Close	Moderately Dipping (35-55°)
36 - 120	Thick	Wide	Steep or High Angle (55-85°)
> 120	Very Thick	Very Wide	Vertical (85-90°)

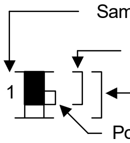
Core Recovery and Rock Quality Designation

$$\text{Core Recovery} = \frac{\textit{length of core recovered}}{\textit{total length of core run}} \times 100$$

$$\text{RQD} = \frac{\textit{total length of all pieces 4 inches or greater}}{\textit{total length of core run}} \times 100$$

Coring and Sampling Key

SAMPLE NUMBER & INTERVAL



SAMPLER TYPE

Code	Description
e	Other - See text if applicable
f	Single Tube Core Barrel
g	Double Tube Core Barrel
4	Other - See text if applicable
5	Air Rotary
6	Wash Rotary
7	Rotosonic

Field and Lab Test Data

Code	Description
W = 10	Moisture Content, %
D = 120	Dry Density, pcf
CS = 1.0	Compressive Strength, tsf
TS = 0.5	Tensile Strength, tsf
GT	Other Geotechnical Testing
CA	Chemical Analysis

Groundwater

∇ ATD	Approximate water elevation at time of drilling (ATD) or on date noted. Groundwater levels can fluctuate due to precipitation, seasonal conditions, and other factors.

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Portland, Oregon

Rock Classification
System and Key

Figure
A1.2

B-1

LAI Project No: 1374022.010

SAMPLE DATA / SOIL PROFILE

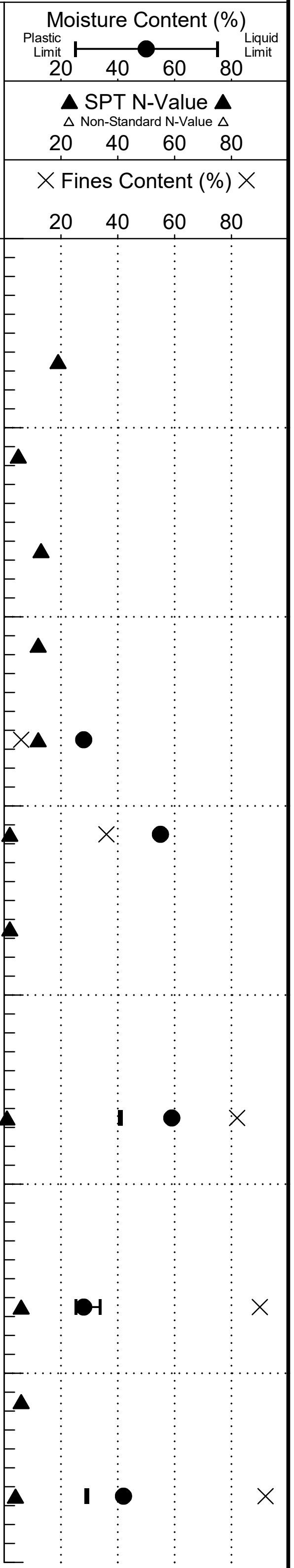
Drilling Method: Mud Rotary

Ground Elevation (ft): Not Measured

Drilled By: Holt

Logged By: BMD Date: 08/11/22

Depth (ft)	Sample Number & Interval	Sampler Type	Blows/Foot	Test Data	USCS Symbol	Description	Groundwater
0					SP	Dark gray fine SAND (medium dense, moist) **(FILL)**	Groundwater Not Determined
	S-1	b2	19				
5	S-2	b2	5				
	S-3	b2	13				
10	S-4	b2	12				
	S-5	b2	12	W = 28 -200 = 6			
15	S-6	b2	2	W = 55 -200 = 36		grades to very loose, silty	
	S-7	b2	2		ML	Gray mottled brown sandy SILT (soft, moist) **(ALLUVIUM)** 2-inch fine sandy lense	
20	S-8	c3					
	S-9	b2	1	W = 59 GS AL			
25	S-10	c3					
	S-11	b2	6	W = 28 GS AL		grades to medium stiff, with sand	
30	S-12	b2	6			increase in mottling and apparant weathering	
	S-13	b2	4	W = 42 GS AL			
35							



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

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McCall Terminal
Upgrade Project
Portland, Oregon

Log of Boring B-1

Figure
A2
(1 of 3)

B-1

LAI Project No: 1374022.010

SAMPLE DATA / SOIL PROFILE

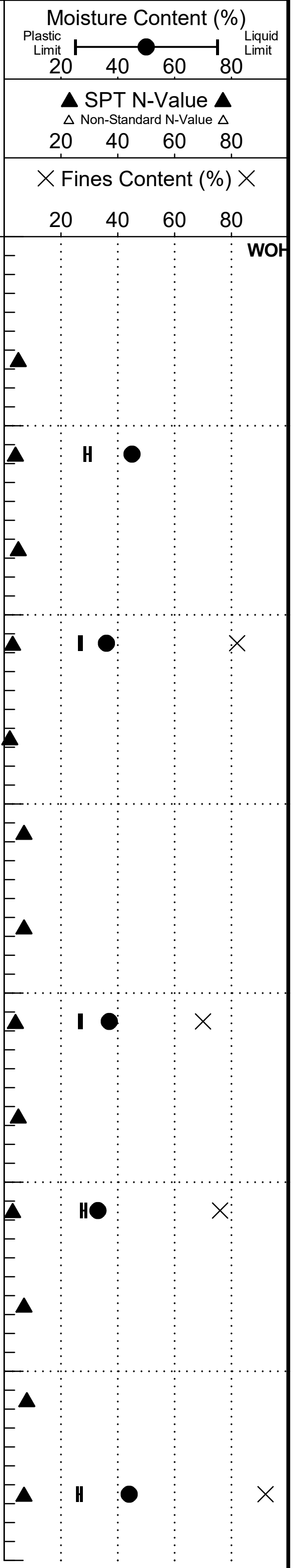
Drilling Method: Mud Rotary

Ground Elevation (ft): Not Measured

Drilled By: Holt

Logged By: BMD Date: 08/11/22

Depth (ft)	Sample Number & Interval	Sampler Type	Blows/Foot	Test Data	USCS Symbol	Soil Profile	Groundwater
35	S-14	b2	WOH		ML	Gray mottled brown sandy SILT (soft, moist) **(ALLUVIUM)**	Groundwater Not Determined
	S-15	b2	5				
40	S-16	b2	4	W = 45 AL		grades to soft brown stratified sandy SILT with fine sand layers (very loose to loose, wet)	
	S-17	b2	5				
45	S-18	b2	3	W = 36 -200 = 82 AL			
	S-19	b2	2				
50	S-20	b2	7			grades to medium stiff	
	S-21	b2	7				
55	S-22	b2	4	W = 37 GS AL			
	S-23	b2	5				
60	S-24	b2	3	W = 33 GS AL			
	S-25	b2	7				
65	S-26	b2	8				
	S-17	b2	7	W = 44 GS AL			
70							



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

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McCall Terminal
Upgrade Project
Portland, Oregon

Log of Boring B-1

Figure

A2
(2 of 3)

B-1

LAI Project No: 1374022.010

SAMPLE DATA — SOIL PROFILE

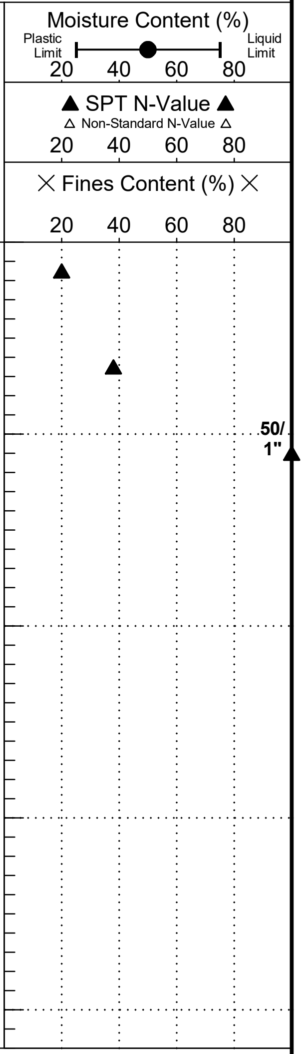
Drilling Method: Mud Rotary

Ground Elevation (ft): Not Measured

Drilled By: Holt

Logged By: BMD Date: 08/11/22

Depth (ft)	Sample Number & Interval	Sampler Type	Blows/Foot	Test Data	Graphic Symbol	USCS Symbol	Description	Groundwater
70	S-18	b2	20			ML	Gray mottled brown sandy SILT (soft, moist) **(ALLUVIUM)** grades to very dense, sharp increase in SPT blows	Groundwater Not Determined
	S-29	b2	38					
75	S-30	b2	50/ 1"				grades to hard	
						BST	Black BASALT; hard, fresh, massively bedded, fine grained with high angle joints and random fractures; CR=100%, RQD=52% (hard, damp) **(COLUMBIA FLOOD BASALT)**	
80							grades to blocky with minimal iron oxide staining along fractures decreases in blocky clast size with slight hydrochloric acid (HCl) reaction along calcarious infilled joints and fractures	
	S-31	f5					grades to massive texture with moderate HCl reaction along calcarious infilled joints 2-inch highly vesicular basalt inclusion	
85							fracture observed with moderate weathering and iron oxide staining	
						BST	Black BASALT; hard, fresh, massively bedded, vesicular; CR=100%, RQD=69% (hard, damp) grades to moderately vesicular with fine to medium sized vugs	
90							grades to highly vesicular with intermediate iron oxide staining and sulfuric mineralization grades to highly vesicular to scoriaceous, moderate to high iron oxide staining	
95								
100								
105								



Boring Completed 08/11/22
Total Depth of Boring = 91.0 ft.

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

B-2

LAI Project No: 1374022.010

Drilling Method: Mud Rotary

Ground Elevation (ft): Not Measured

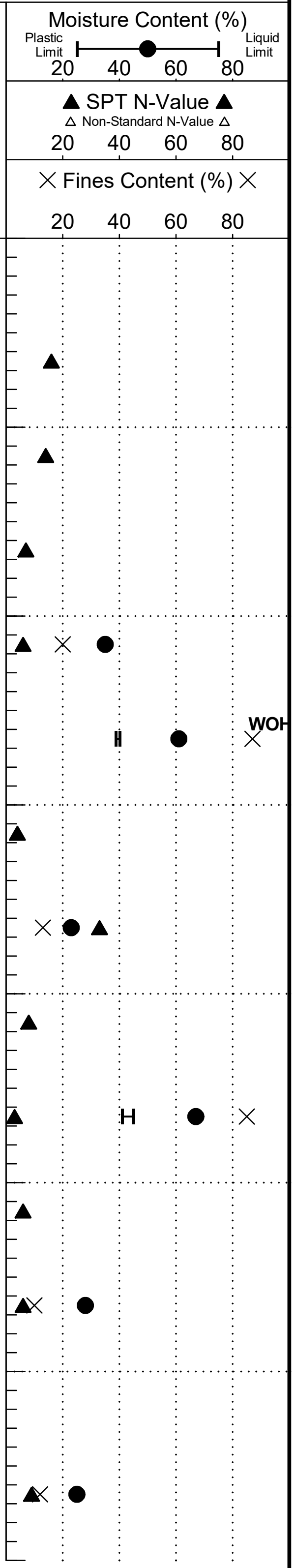
Drilled By: Holt

Logged By: BMD Date: 08/10/22

Groundwater: Groundwater not determined

Sample Number & Interval	Sampler Type	Blows/Foot	Test Data	USCS Symbol	Soil Profile
				SP	Brown fine SAND with silt (medium dense, moist) **(FILL)**
S-1	b2	16			
S-2	b2	14			
S-3	b2	7			
S-4	b2	6	W = 35 -200 = 20		grades to wet
S-5	b2	WOH	W = 61 GS AL	ML	Gray SILT with sand (soft, moist to wet) **(ALLUVIUM)**
S-6	b2	4			
S-7	b2	33	W = 23 -200 = 13	SM/ML	Brown very silty fine SAND with frequent 1 to 4-inch lenses of fine sandy SILT and scattered organic fibers (medium dense, moist to wet)
S-8	b2	8			lense of fine sandy SILT with trace organic fibers
S-9	b2	3	W = 67 GS AL	ML	grades to fine sandy SILT
S-10	b2	6			
S-11	b2	6	W = 28 -200 = 10		
S-12	c3				difficult to progress shelby tube, likely sand contact
S-13	b2	9	W = 25 -200 = 12		

Depth (ft): 0, 5, 10, 15, 20, 25, 30, 35



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

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McCall Terminal
Upgrade Project
Portland, Oregon

Log of Boring B-2

Figure
A3
(1 of 3)

B-2

LAI Project No: 1374022.010

Drilling Method: Mud Rotary

Ground Elevation (ft): Not Measured

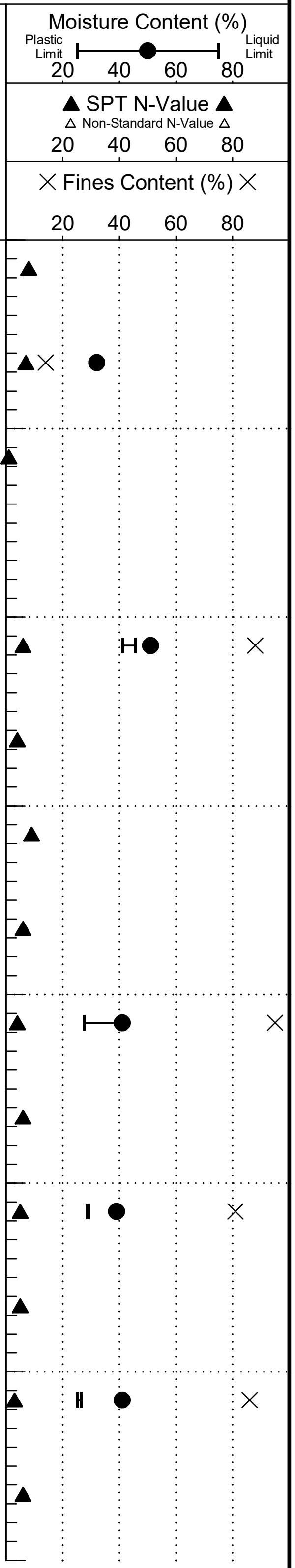
Drilled By: Holt

Logged By: BMD   Date: 08/10/22

Depth (ft)	Sample Number & Interval	Sampler Type	Blows/Foot	Test Data	Graphic Symbol	USCS Symbol	Soil Profile
35						ML	grades to fine sandy SILT
	S-14	b2	8				grades to fine silty SAND
	S-15	b2	7	W = 32 GS			
40							
	S-16	b2	1				grades to SILT with fine sands (soft, moist)
	S-17	c3					
45							
	S-18	b2	6	W = 51 -200 = 88 AL			
	S-19	b2	4				
50							
	S-20	b2	9				
	S-21	b2	6				
55							
	S-22	b2	4	W = 41 GS AL			
	S-23	b2	6				Brown/gray SILT (medium stiff, moist) increase in drill chatter towards bottom of sample
60							
	S-24	b2	5	W = 39 GS AL			
	S-25	b2	5				
65							
	S-26	b2	3	W = 41 GS AL			
	S-27	b2	6				
70							

Groundwater

Groundwater not determined



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

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McCall Terminal
Upgrade Project
Portland, Oregon

Log of Boring B-2

Figure

A3

B-2

LAI Project No: 1374022.010

SAMPLE DATA

SOIL PROFILE

Drilling Method: Mud Rotary

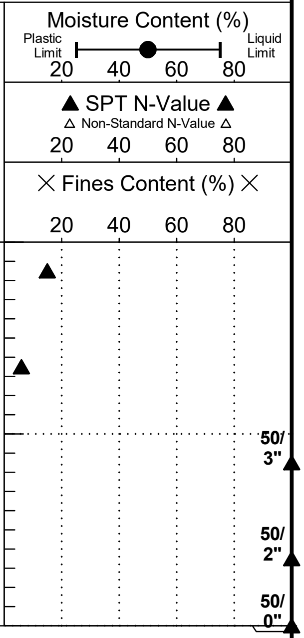
Ground Elevation (ft): Not Measured

Drilled By: Holt

Logged By: BMD   Date: 08/10/22

Depth (ft)	Sample Number & Interval	Sampler Type	Blows/Foot	Test Data
70	S-28	b2	15	
	S-29	b2	6	
75	S-30	b2	50/ 3"	
	S-31	b2	50/ 2"	
80	S-2	b2	50/ 0"	

USCS Symbol	Description	Groundwater
ML	grades to fine sandy SILT	Groundwater not determined
BST	Black BASALT; hard, fresh, fine grained and vesicular stuck in shoe of sampler. (hard, damp) **(COLUMBIA FLOOD BASALT)**	



Boring Completed 08/10/22
Total Depth of Boring = 80.0 ft.

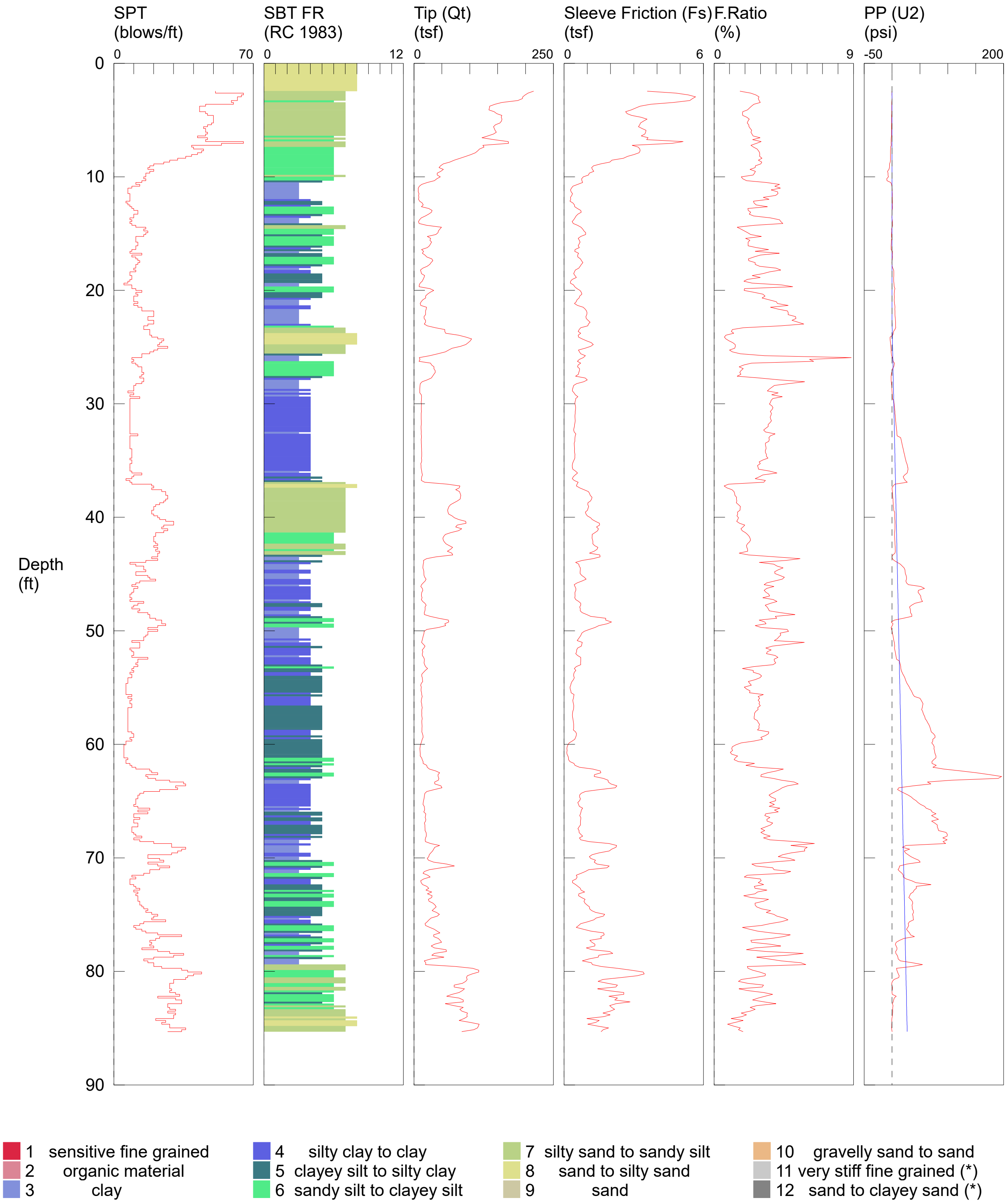
Notes:
1. Stratigraphic contacts are based on field interpretations and are approximate.
2. Reference to the text of this report is necessary for a proper understanding of subsurface conditions.
3. Refer to "Soil Classification System and Key" figure for explanation of graphics and symbols.

McCall Terminal
Upgrade Project
Portland, Oregon

Log of Boring B-2

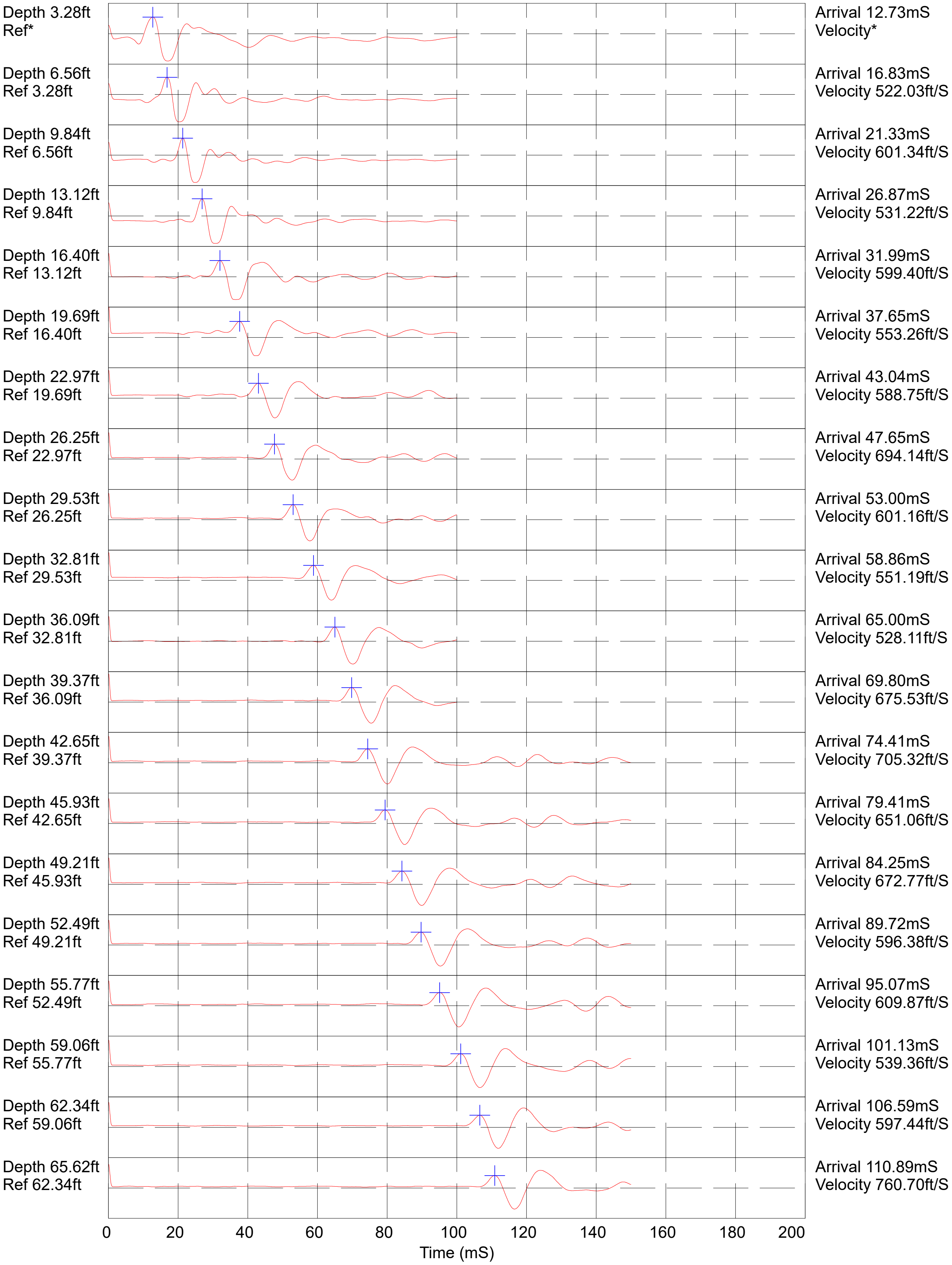
Landau / CPT-1 / 5480 NW Front Ave Portland

OPERATOR: OGE BAK
CONE ID: DDG1532
HOLE NUMBER: CPT-1
TEST DATE: 8/10/2022 1:53:16 PM
TOTAL DEPTH: 85.302 ft

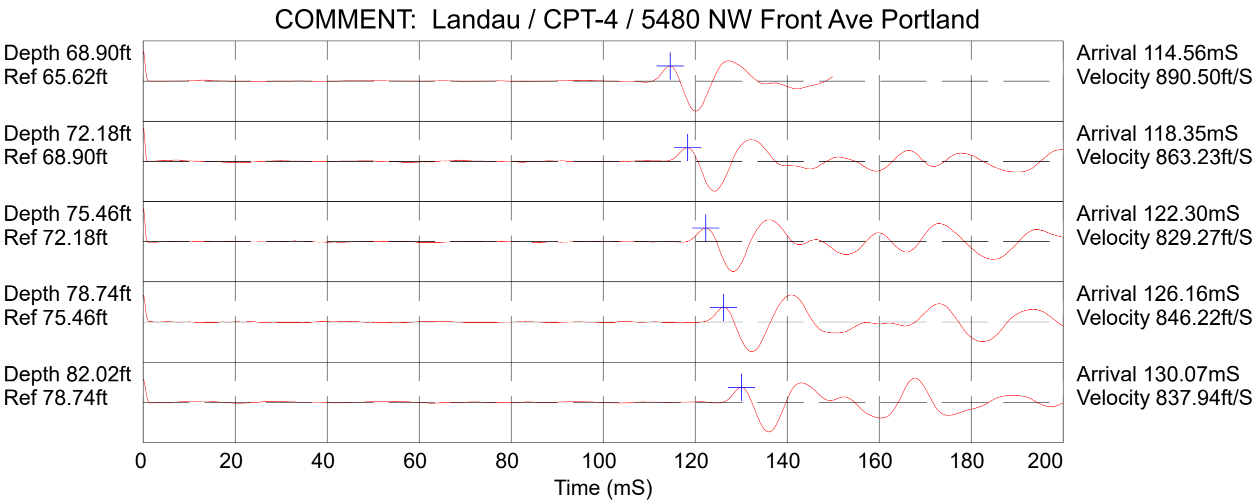


*SBT/SPT CORRELATION: UBC-1983

COMMENT: Landau / CPT-4 / 5480 NW Front Ave Portland



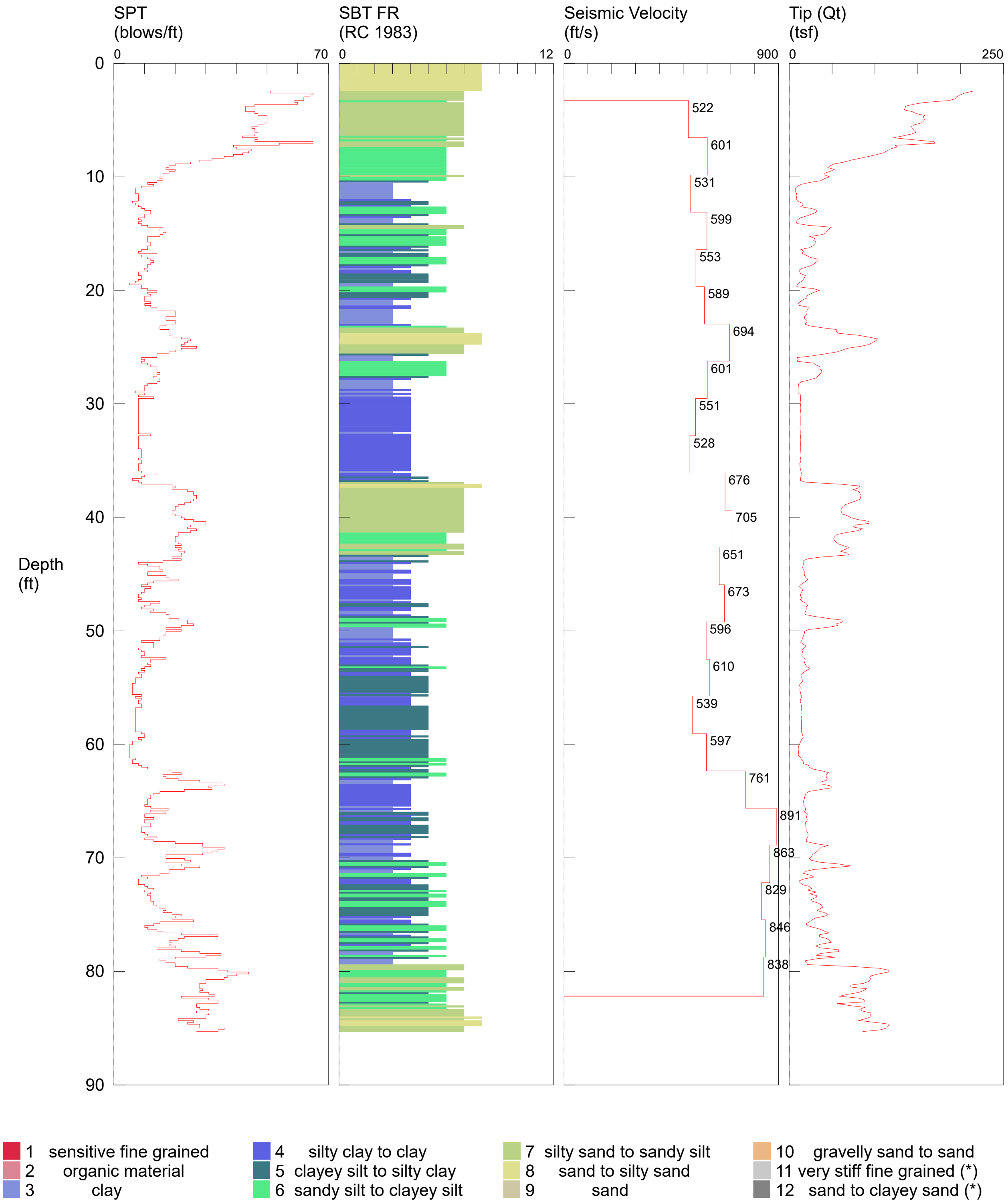
Hammer to Rod String Distance (ft): 5.58
* = Not Determined



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

Landau / CPT-1 / 5480 NW Front Ave Portland

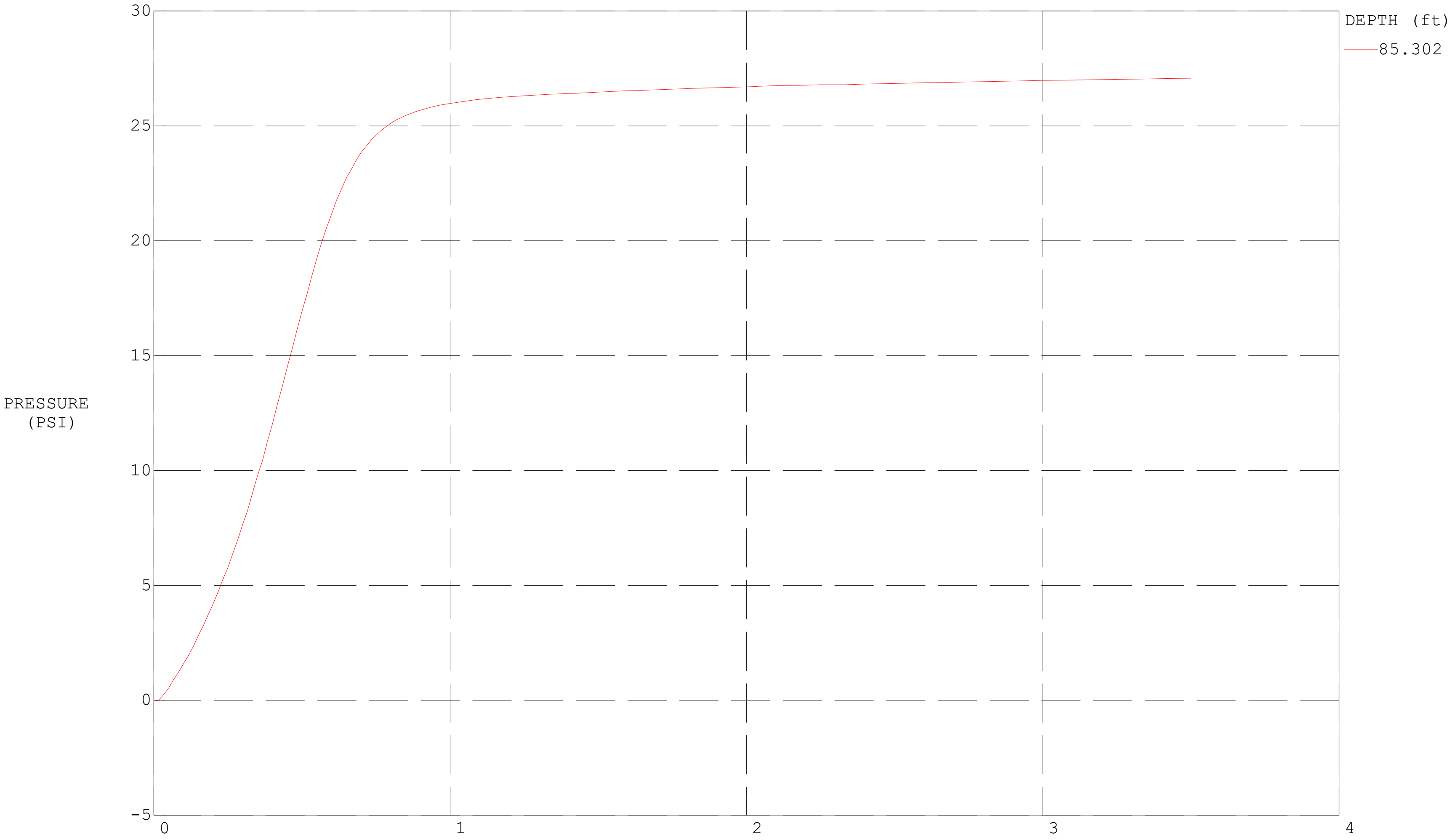
OPERATOR: OGE BAK
CONE ID: DDG1532
HOLE NUMBER: CPT-1
TEST DATE: 8/10/2022 1:53:16 PM
TOTAL DEPTH: 85.302 ft



*SBT/SPT CORRELATION: UBC-1983

COMMENT: Landau / CPT-1 / 5480 NW Front Ave Portland

TEST DATE: 8/10/2022 1:53:16 PM



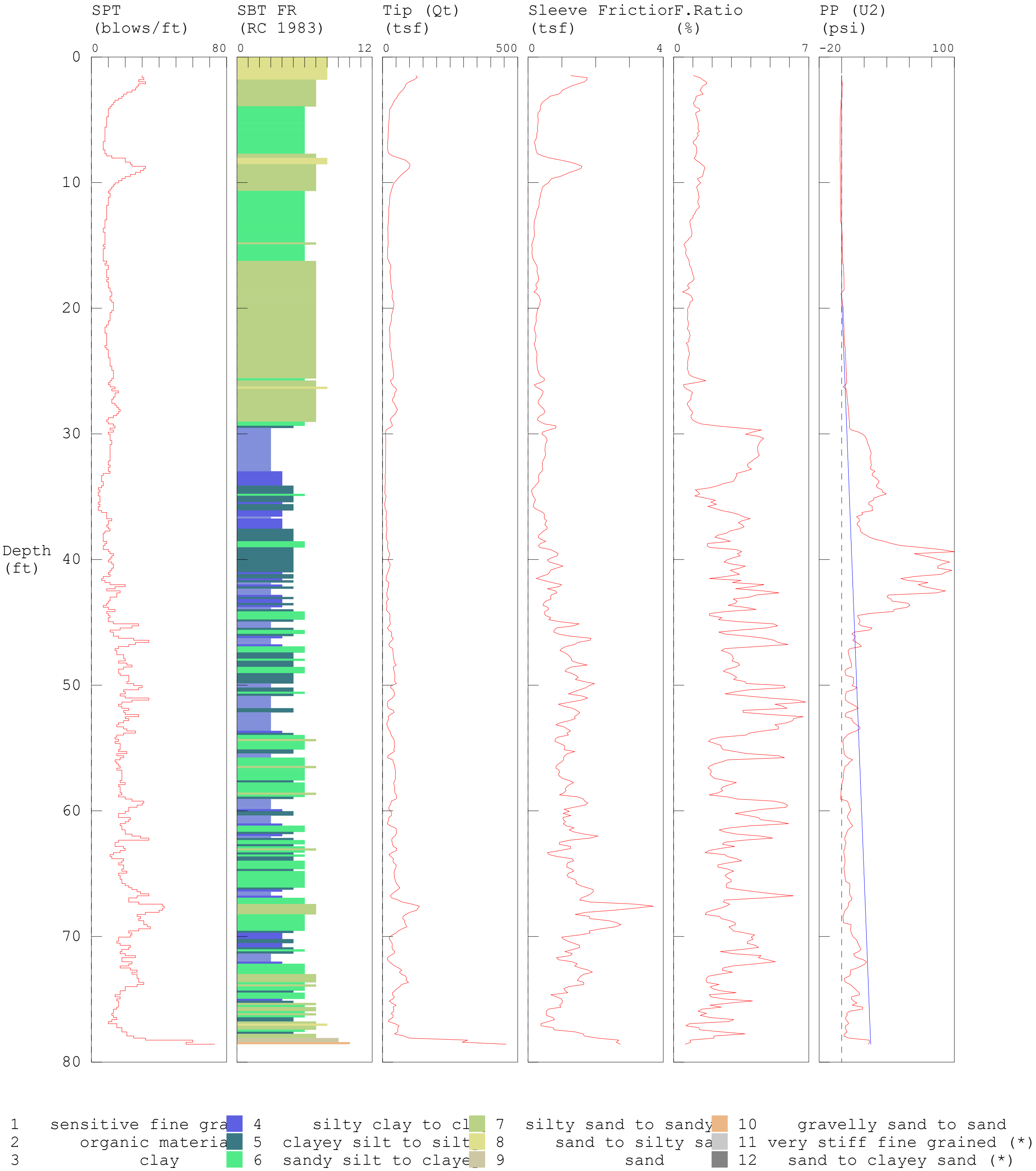
MAXIMUM PRESSURE = 27.072 (PSI)

TIME: (MINUTES)

HYDROSTATIC PRESSURE = 27.101 (PSI), WATER TABLE: 22.77 ft

Landau / CPT-2 / 5480 NW Front Ave Portland

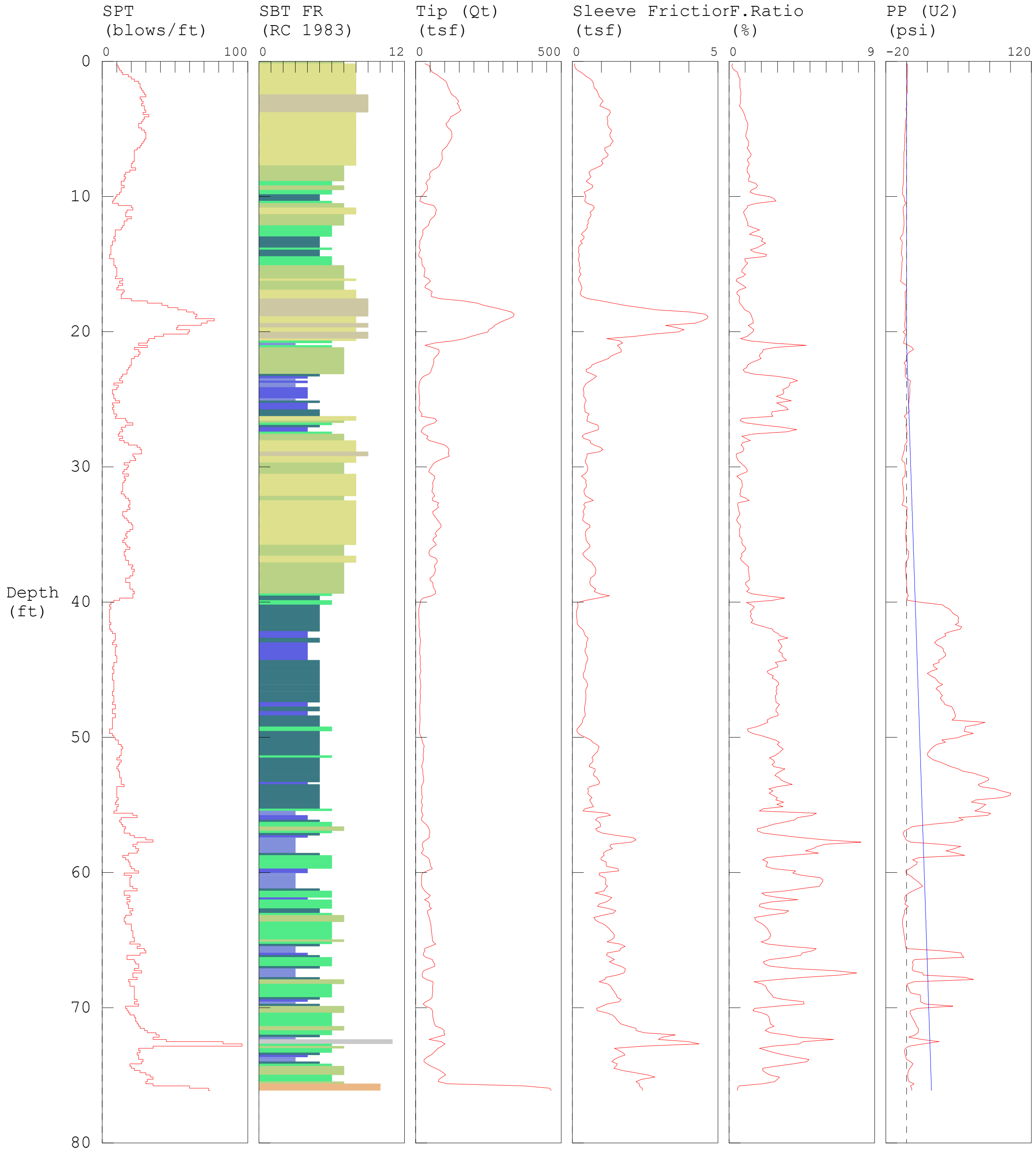
OPERATOR: OGE BAK
CONE ID: DDG1532
HOLE NUMBER: CPT-2
TEST DATE: 8/11/2022 11:28:20 AM
TOTAL DEPTH: 78.576 ft



*SBT/SPT CORRELATION: UBC-1983

Landau / CPT-3 / 5480 NW Front Ave Portland

OPERATOR: OGE BAK
CONE ID: DDG1532
HOLE NUMBER: CPT-3
TEST DATE: 8/11/2022 8:47:20 AM
TOTAL DEPTH: 76.115 ft

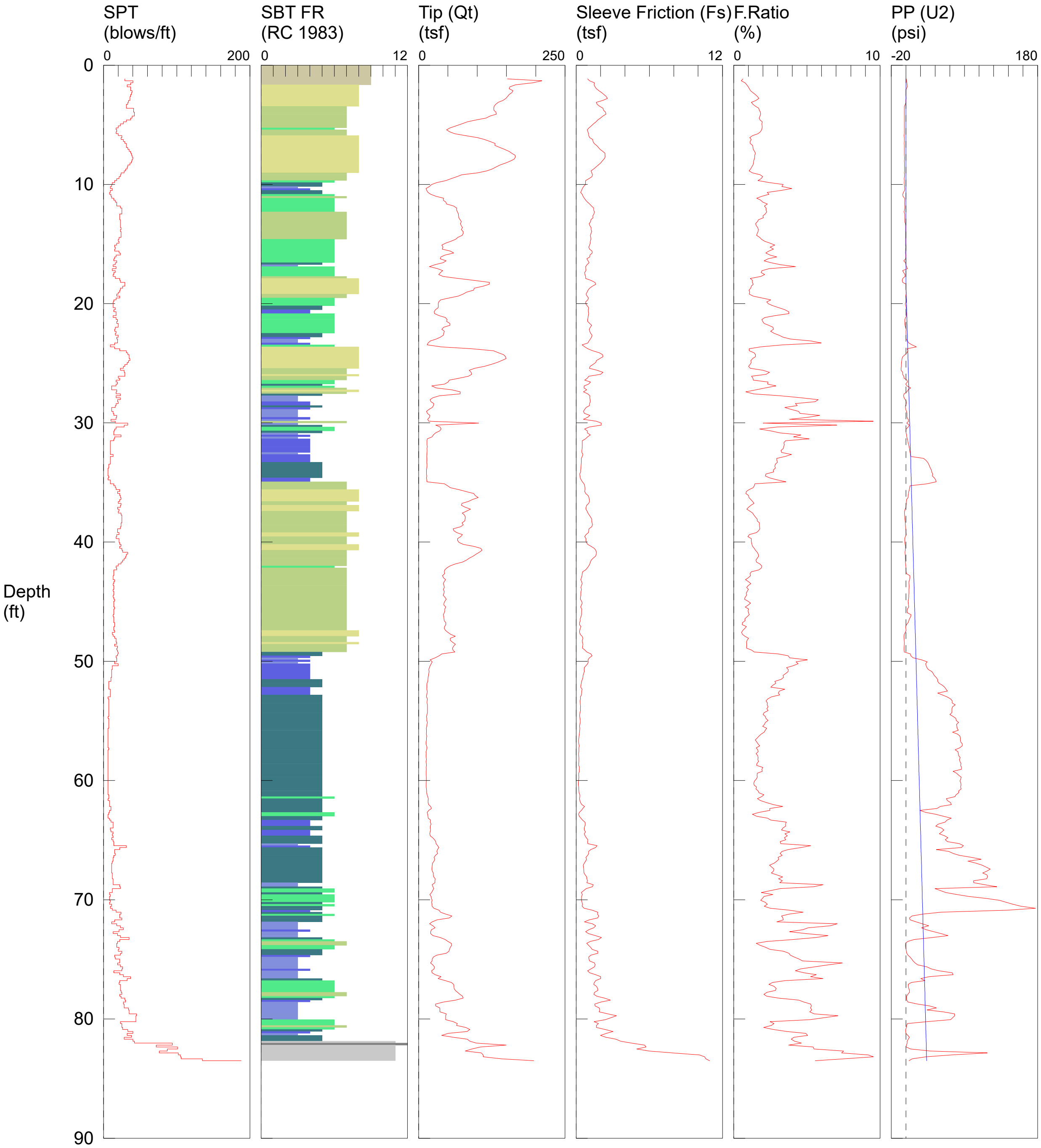


🟥 1	sensitive fine gra	🟦 4	silty clay to cl	🟩 7	silty sand to sandy	🟧 10	gravelly sand to sand
🟥 2	organic materia	🟦 5	clayey silt to silt	🟨 8	sand to silty sa	⬜ 11	very stiff fine grained (*)
🟦 3	clay	🟩 6	sandy silt to claye	🟫 9	sand	⬛ 12	sand to clayey sand (*)

*SBT/SPT CORRELATION: UBC-1983

Landau / CPT-4 / 5480 NW Front Ave Portland

OPERATOR: OGE BAK
CONE ID: DDG1532
HOLE NUMBER: CPT-4
TEST DATE: 8/10/2022 9:00:39 AM
TOTAL DEPTH: 83.497 ft

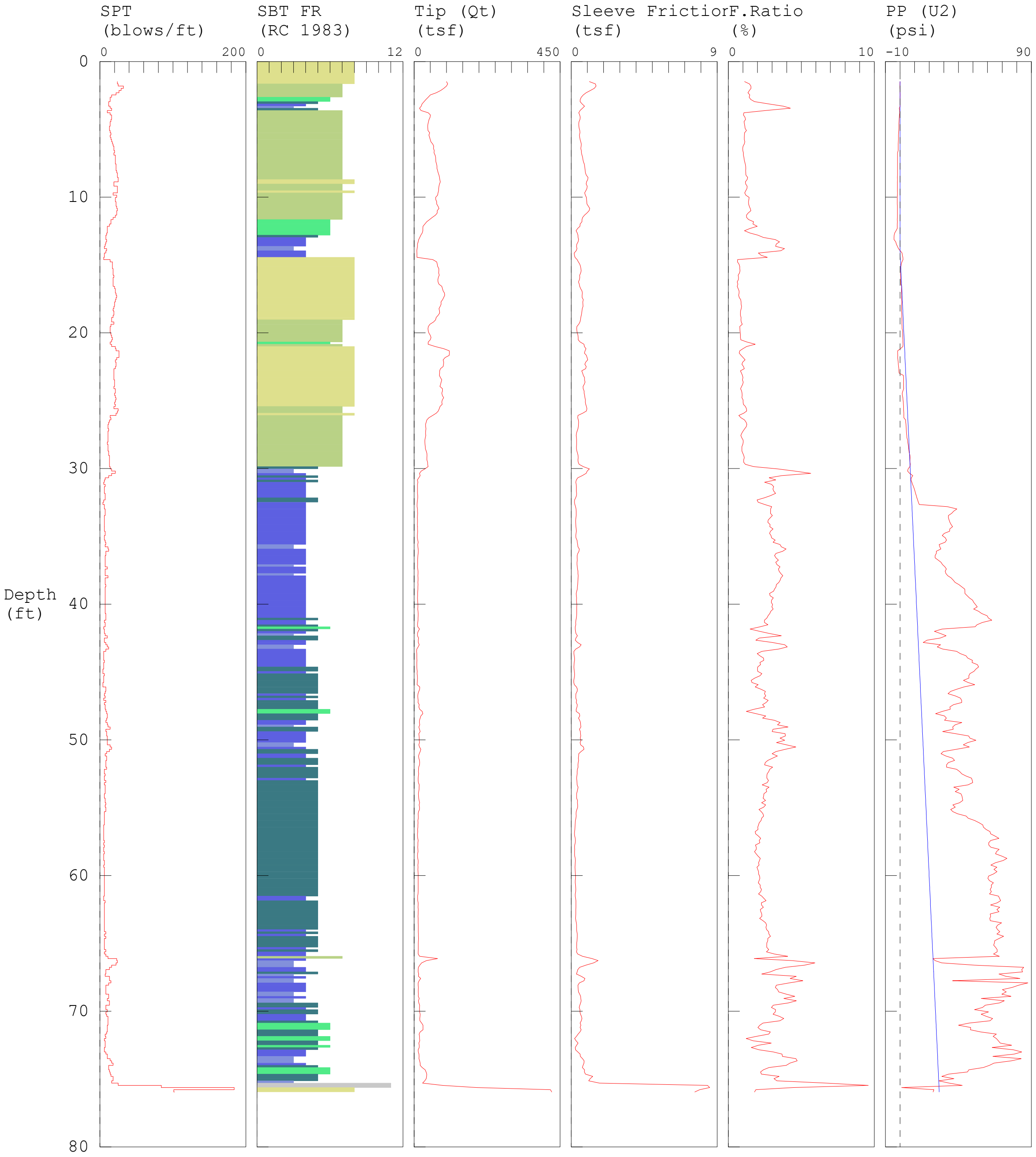


🟥	1	sensitive fine grained	🟦	4	silty clay to clay	🟩	7	silty sand to sandy silt	🟧	10	gravelly sand to sand
🟥	2	organic material	🟦	5	clayey silt to silty clay	🟨	8	sand to silty sand	⬜	11	very stiff fine grained (*)
🟦	3	clay	🟩	6	sandy silt to clayey silt	🟫	9	sand	⬛	12	sand to clayey sand (*)

*SBT/SPT CORRELATION: UBC-1983

Landau / CPT-5 / 5480 NW Front Ave Portland

OPERATOR: OGE BAK
CONE ID: DDG1532
HOLE NUMBER: CPT-5
TEST DATE: 8/11/2022 10:10:41 AM
TOTAL DEPTH: 75.951 ft

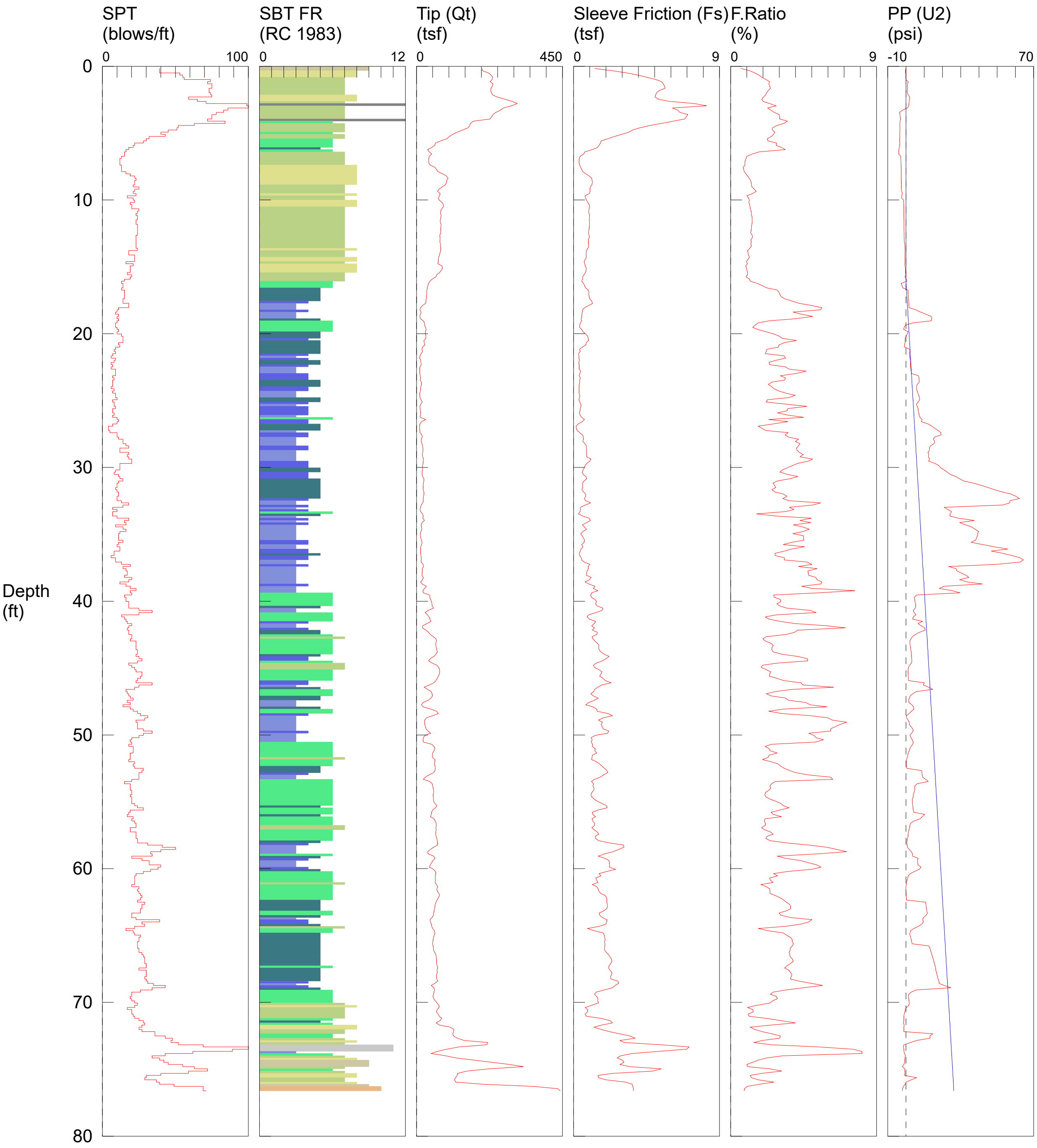


🟥	1	sensitive fine gra	🟦	4	silty clay to cl	🟩	7	silty sand to sandy	🟧	10	gravelly sand to sand
🟥	2	organic materia	🟦	5	clayey silt to silt_	🟨	8	sand to silty sa	⬜	11	very stiff fine grained (*)
🟦	3	clay	🟩	6	sandy silt to claye_	🟫	9	sand	⬛	12	sand to clayey sand (*)

*SBT/SPT CORRELATION: UBC-1983

Landau / CPT-6 / 5480 NW Front Ave Portland

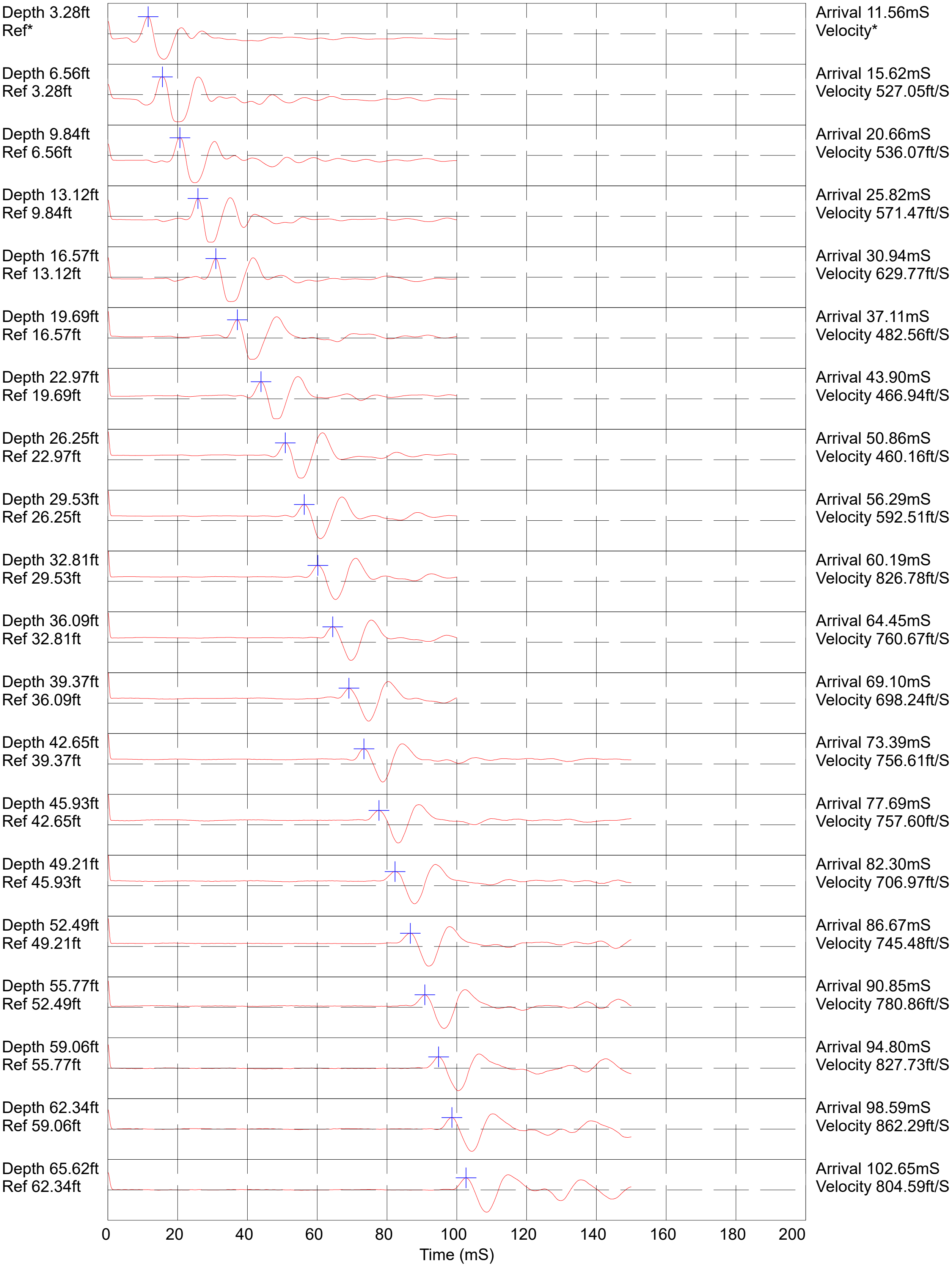
OPERATOR: OGE BAK
CONE ID: DDG1532
HOLE NUMBER: CPT-6
TEST DATE: 8/10/2022 11:42:05 AM
TOTAL DEPTH: 76.608 ft



🟥 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 (*)

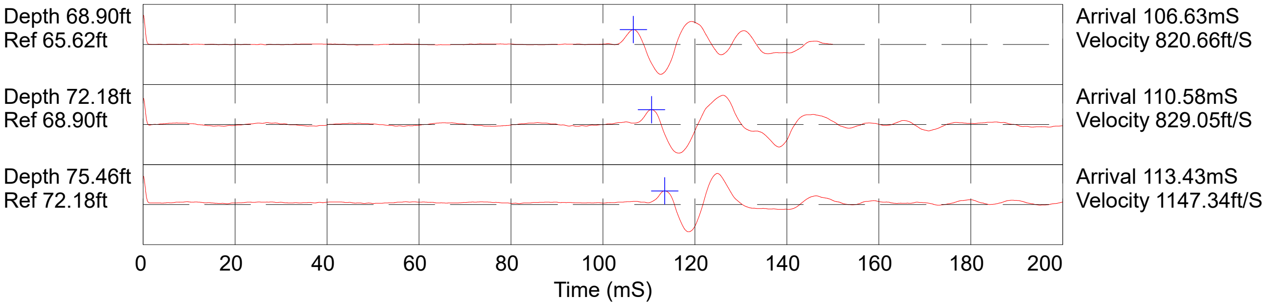
*SBT/SPT CORRELATION: UBC-1983

COMMENT: Landau / CPT-6 / 5480 NW Front Ave Portland



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

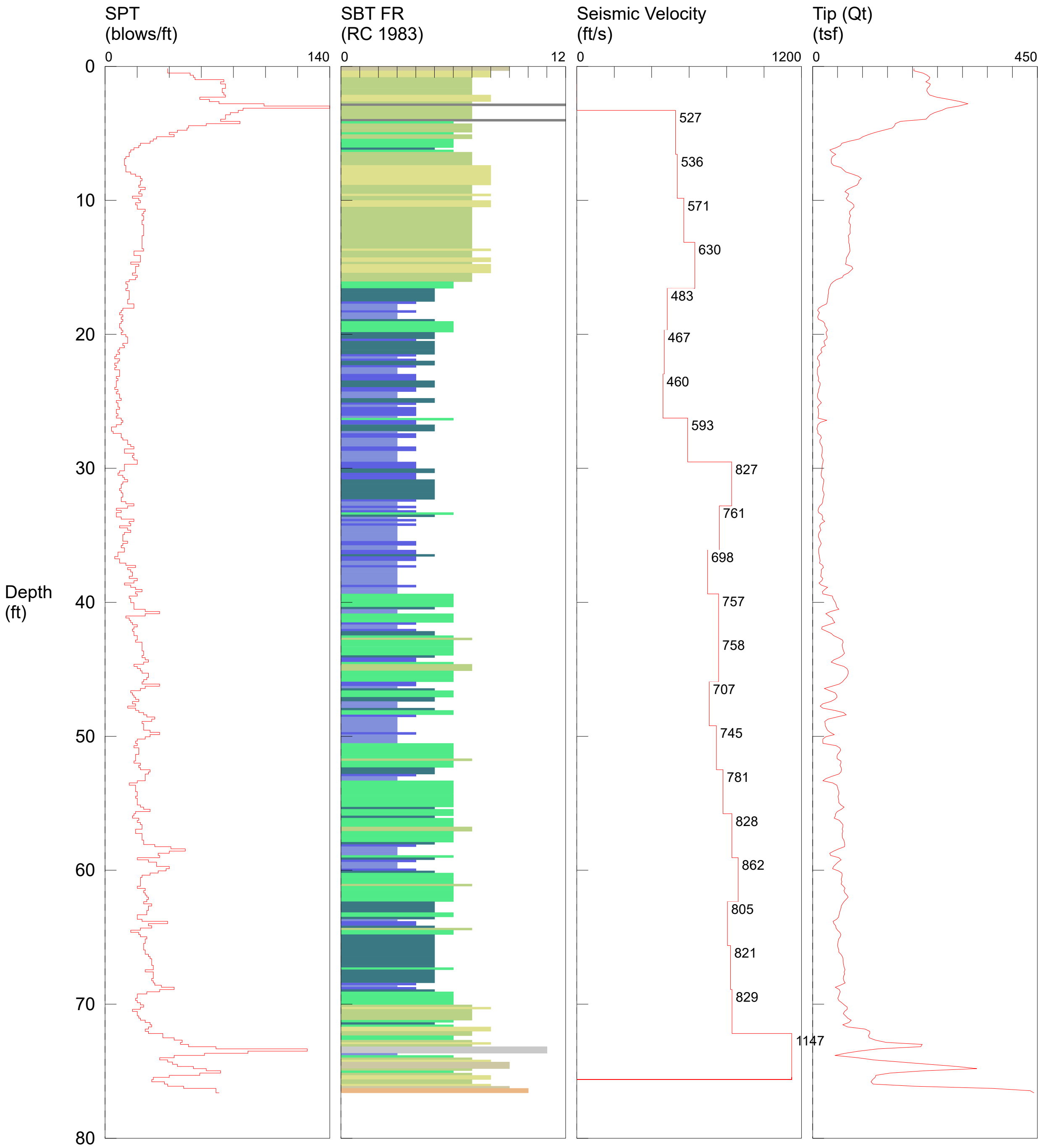
COMMENT: Landau / CPT-6 / 5480 NW Front Ave Portland



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

Landau / CPT-6 / 5480 NW Front Ave Portland

OPERATOR: OGE BAK
CONE ID: DDG1532
HOLE NUMBER: CPT-6
TEST DATE: 8/10/2022 11:42:05 AM
TOTAL DEPTH: 76.608 ft

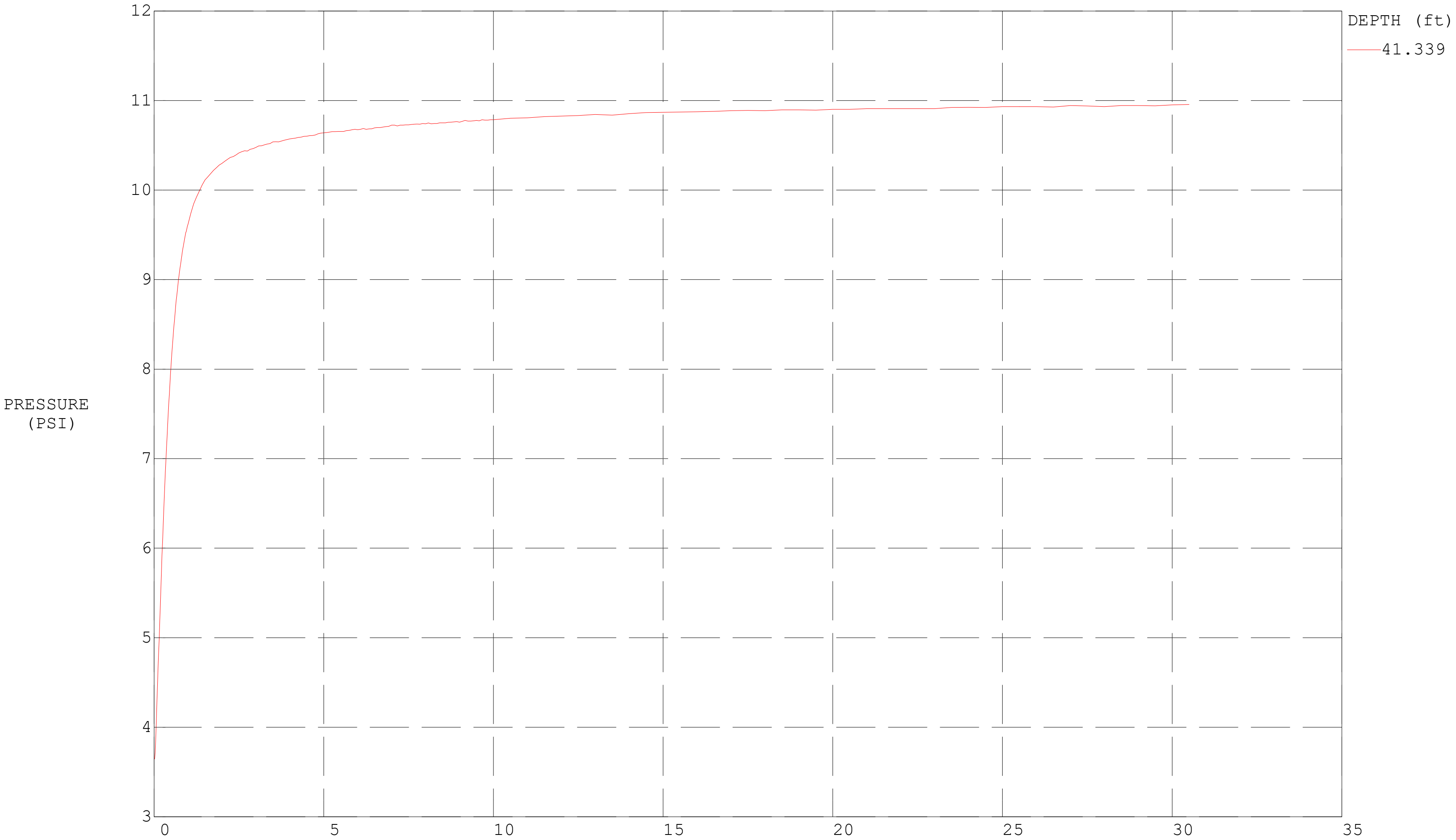


- 🟥 1 sensitive fine grained
- 🟥 2 organic material
- 🟦 3 clay
- 🟦 4 silty clay to clay
- 🟩 5 clayey silt to silty clay
- 🟩 6 sandy silt to clayey silt
- 🟩 7 silty sand to sandy silt
- 🟨 8 sand to silty sand
- 🟫 9 sand
- 🟧 10 gravelly sand to sand
- ⬜ 11 very stiff fine grained (*)
- ⬛ 12 sand to clayey sand (*)

*SBT/SPT CORRELATION: UBC-1983

COMMENT: Landau / CPT-6 / 5480 NW Front Ave Portland

TEST DATE: 8/10/2022 11:42:05 AM



MAXIMUM PRESSURE = 10.957 (PSI)

TIME: (MINUTES)

HYDROSTATIC PRESSURE = 10.968 (PSI), WATER TABLE: 16.03 ft

APPENDIX B

Laboratory Soil Testing

APPENDIX B
LABORATORY SOIL TESTING

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

Natural Moisture Content

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

Grain Size Analysis

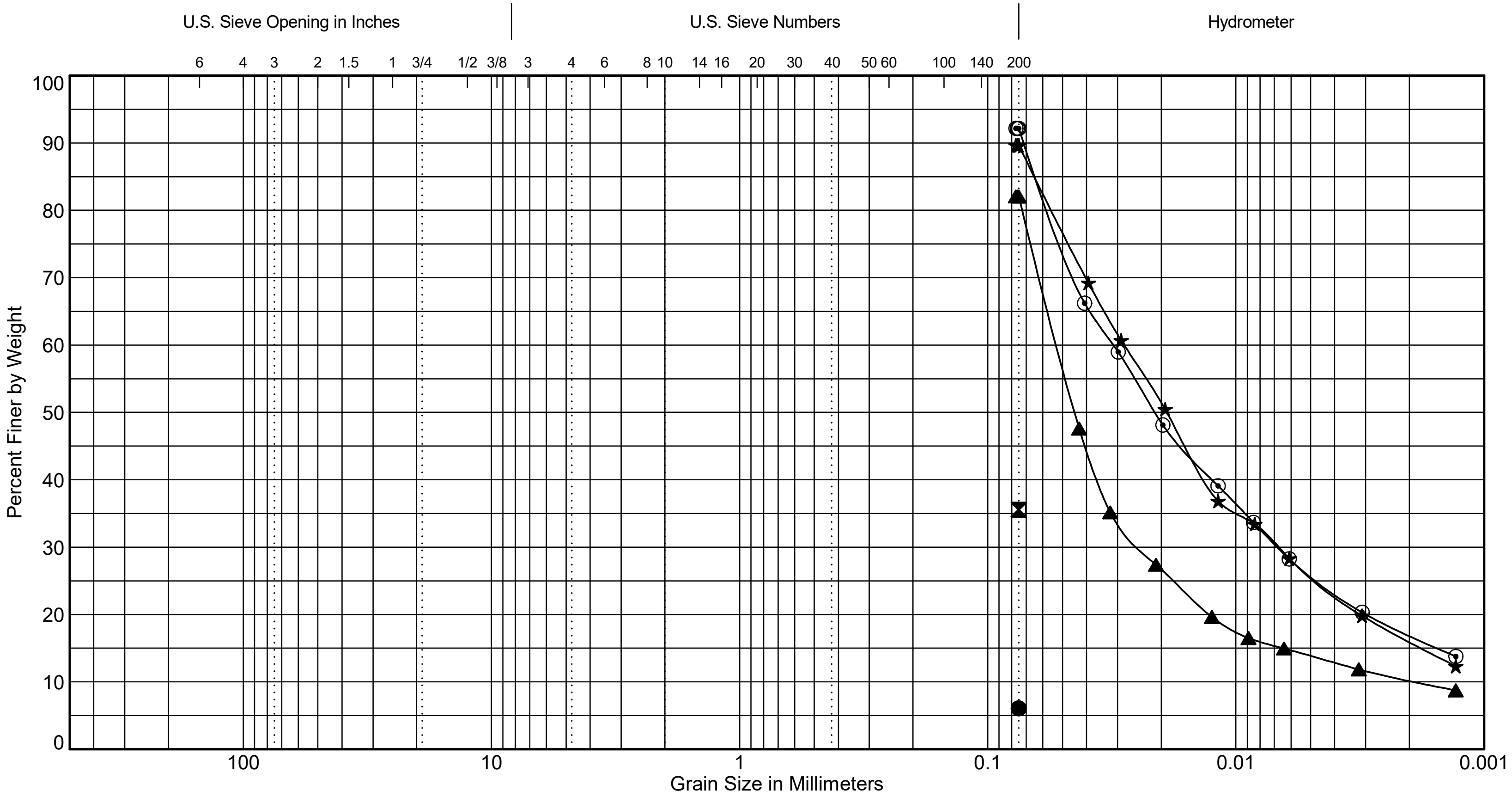
To provide an indication of the grain size distribution of site soils, grain size analyses were performed in accordance with ASTM test method D422. Samples selected for grain size analysis are designated with a "GS" in the column labeled "Test Data" on the summary boring logs in Appendix A. The results of the grain size analyses are presented in the form of grain size distribution curves on Figures B1 through B3.

Atterberg Limit Determination

To assess the plasticity of fine-grained site soils, Atterberg limit tests were performed in general accordance with ASTM test method D4318. Samples selected for Atterberg limit tests are designated with an "AL" in the column labeled "Test Data" on the summary boring logs in Appendix A. The results of the Atterberg limit tests are presented in graphical and tabular form on Figures B4 and B5.

Core Recovery and Rock Quality Designation

Fifteen feet (ft) of basalt core was collected for the purpose of determining the rock quality designation (RDQ) as well as the total core recovery (CR) as a standard parameter in general accordance with ASTM method D6032/D6032M-17. RQD and CR values are located on the summary boring logs in Appendix A.



Cobbles	Gravel		Sand			Silt or Clay
	Coarse	Fine	Coarse	Medium	Fine	

Symbol	Exploration Number	Sample Number	Depth (ft)	Natural Moisture (%)	Soil Description	Unified Soil Classification
●	B-1	S-5	12.5	28		
⧗	B-1	S-6	15.0	55		
▲	B-1	S-9	22.5	59	Sandy SILT	ML
★	B-1	S-11	27.5	28	SILT with sand	ML
⊙	B-1	S-13	32.5	42	SILT with sand	ML

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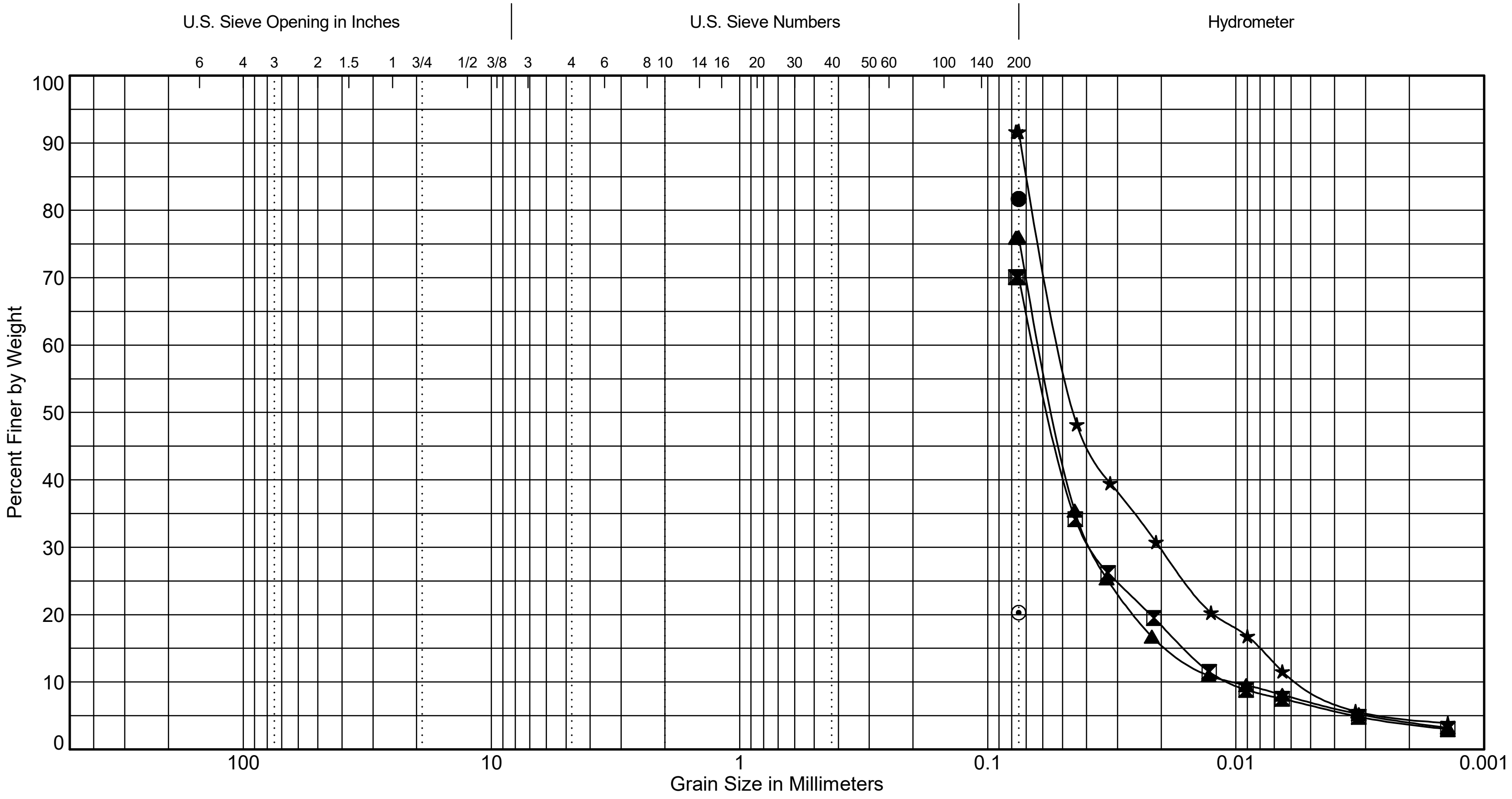
Portland, Oregon

Grain Size Distribution

Figure
B1



<table>
<tr><td rowspan="2">Cobbles</td><td colspan="2">Gravel</td><td colspan="3">Sand</td><td rowspan="2">Silt or Clay</td></tr>
<tr><td>Coarse</td><td>Fine</td><td>Coarse</td><td>Medium</td><td>Fine</td></tr>
</table>

Symbol	Exploration Number	Sample Number	Depth (ft)	Natural Moisture (%)	Soil Description	Unified Soil Classification
●	B-1	S-18	45.0	36	Sandy SILT	ML
⧗	B-1	S-22	55.0	37	Sandy SILT	ML
▲	B-1	S-24	60.0	33	Sandy SILT	ML
★	B-1	S-27	67.5	44	SILT with sand	ML
⊙	B-2	S-4	10.0	35		

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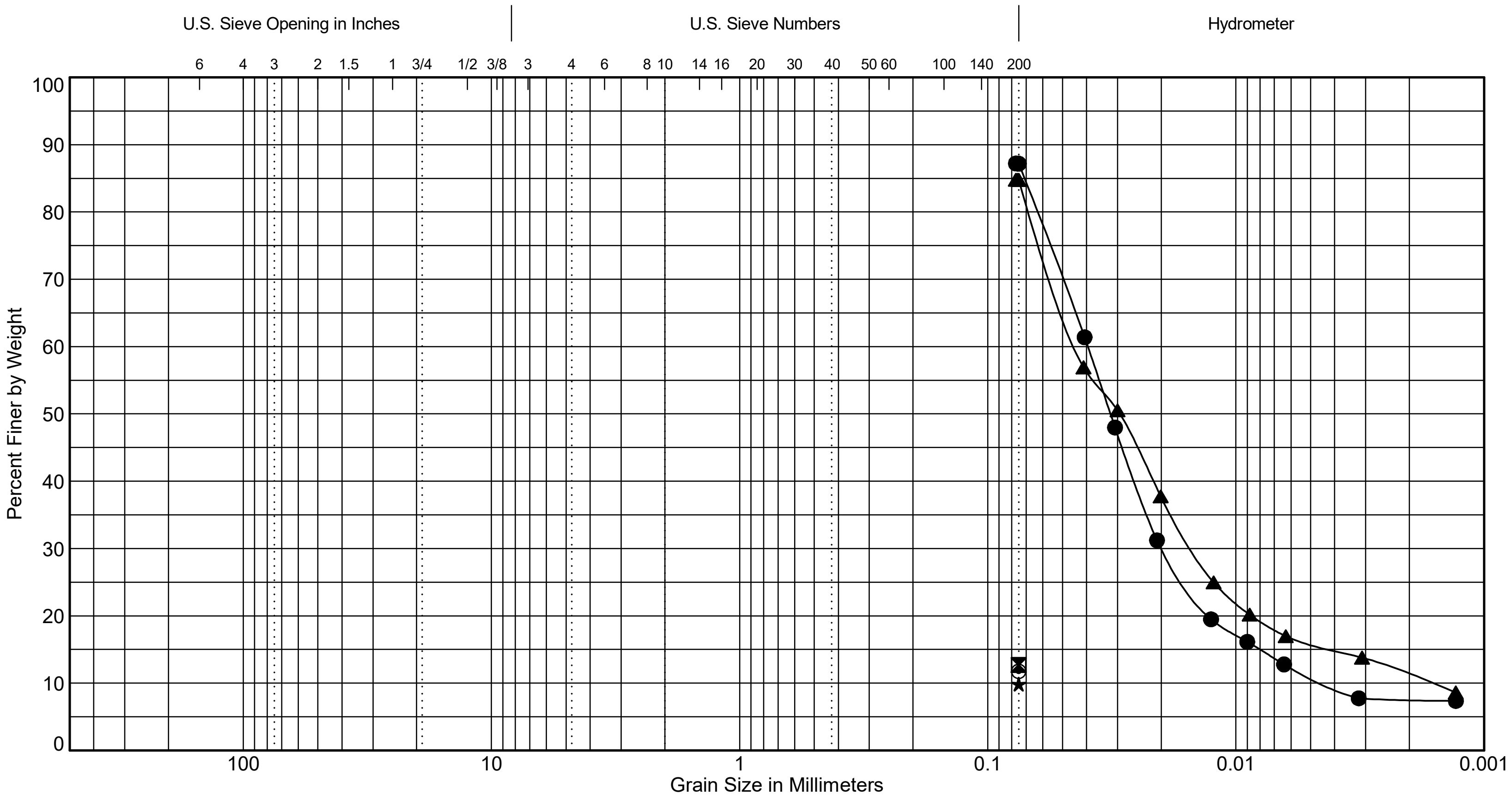
Portland, Oregon

Grain Size Distribution

Figure
B2



Cobbles	Gravel		Sand			Silt or Clay
	Coarse	Fine	Coarse	Medium	Fine	

Symbol	Exploration Number	Sample Number	Depth (ft)	Natural Moisture (%)	Soil Description	Unified Soil Classification
●	B-2	S-5	12.5	61	SILT with sand	ML
⧗	B-2	S-7	17.5	23		
▲	B-2	S-9	22.5	67	Sandy SILT	ML
★	B-2	S-11	27.5	28		
⊙	B-2	S-13	32.5	25		

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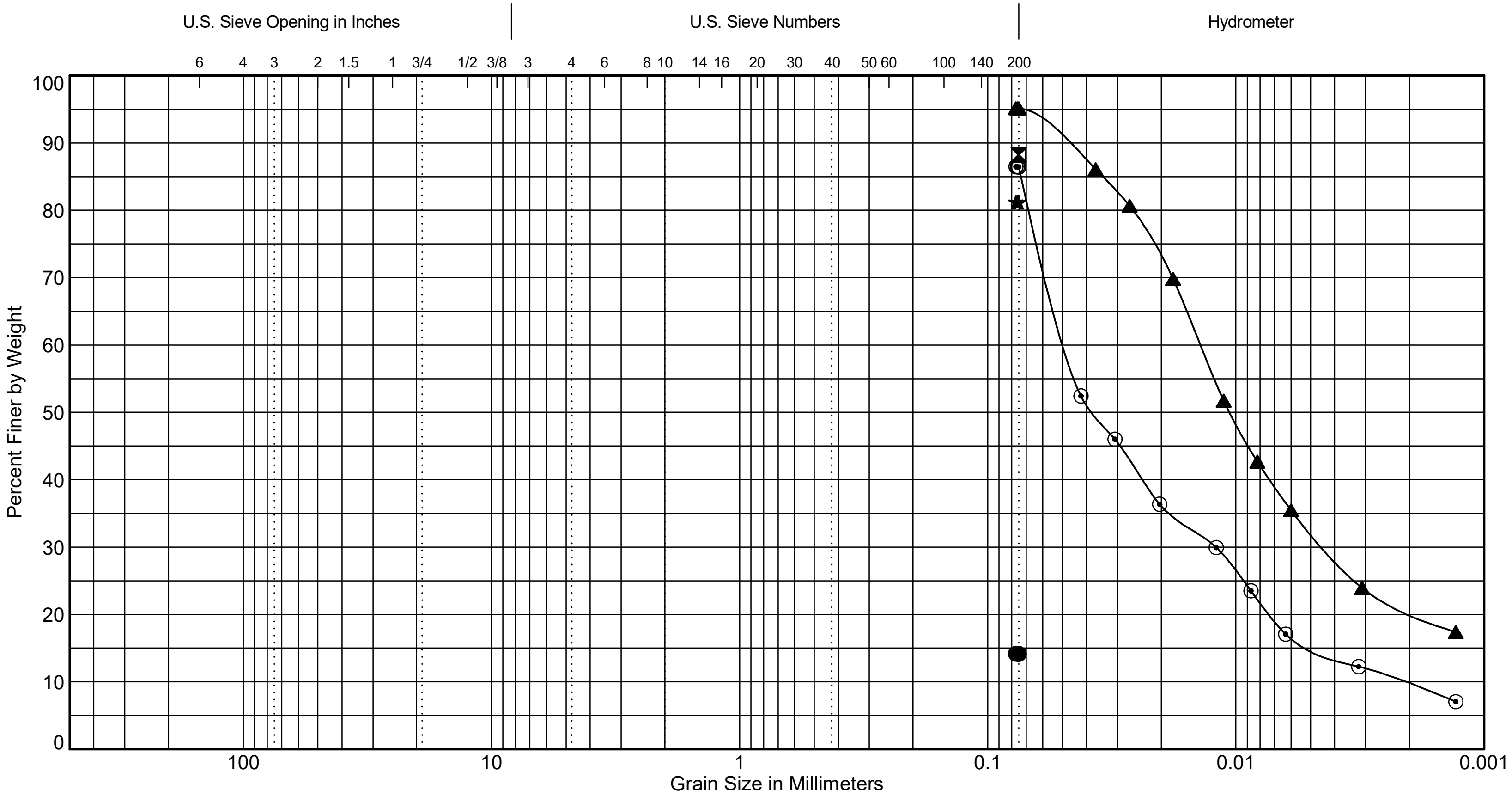
Portland, Oregon

Grain Size Distribution

Figure
B3



Cobbles	Gravel		Sand			Silt or Clay
	Coarse	Fine	Coarse	Medium	Fine	

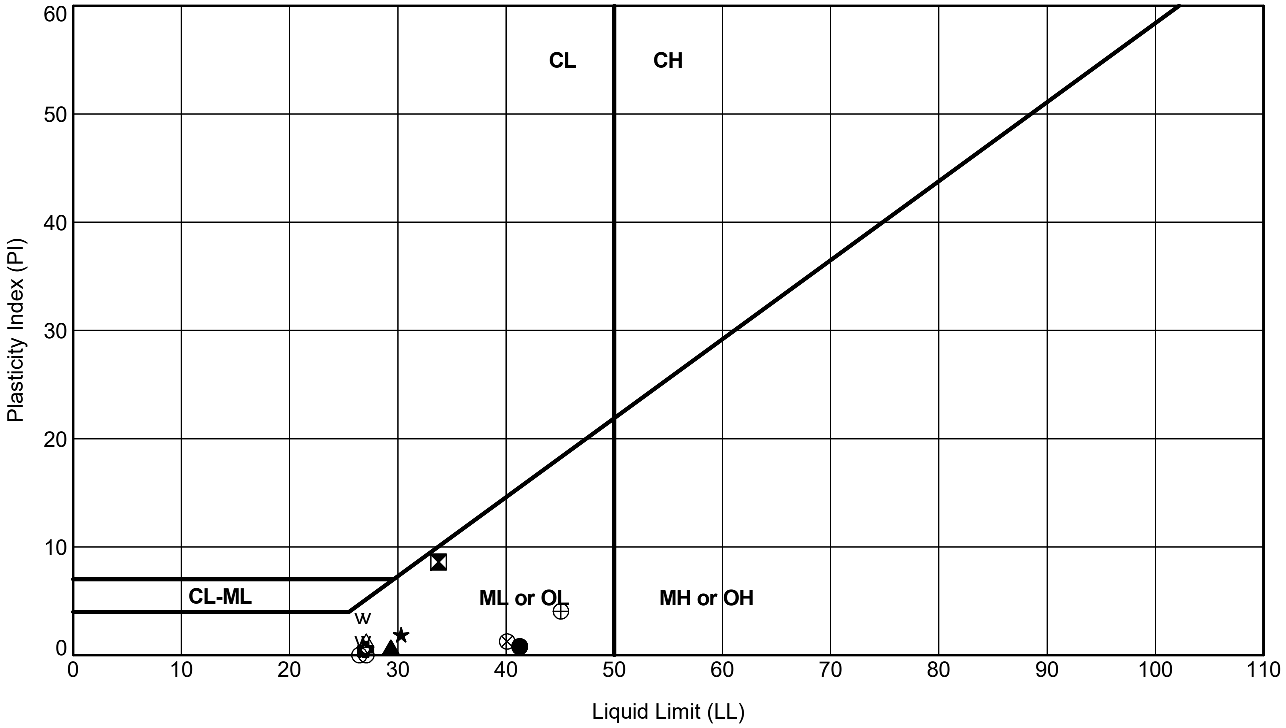
Symbol	Exploration Number	Sample Number	Depth (ft)	Natural Moisture (%)	Soil Description	Unified Soil Classification
●	B-2	S-15	37.5	32		
⧗	B-2	S-18	45.0	51	SILT with sand	ML
▲	B-2	S-22	55.0	41	SILT	ML
★	B-2	S-24	60.0	39	Sandy SILT	ML
⊙	B-2	S-26	65.0	41	SILT with sand	ML

LANDAU
ASSOCIATES

McCall Terminal
Upgrade Project
Portland, Oregon

Grain Size Distribution

Figure
B4



ATTERBERG LIMIT TEST RESULTS

Symbol	Exploration Number	Sample Number	Depth (ft)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Natural Moisture (%)	Soil Description	Unified Soil Classification
●	B-1	S-9	22.5	41	40	1	59	Sandy SILT	ML
⧗	B-1	S-11	27.5	34	25	9	28	SILT with sand	ML
▲	B-1	S-13	32.5	29	29	NP	42	SILT with sand	ML
★	B-1	S-16	40.0	30	28	2	45	SILT with sand	ML
⊙	B-1	S-18	45.0	26	27	NP	36	Sandy SILT	ML
✿	B-1	S-22	55.0	27	26	1	37	Sandy SILT	ML
○	B-1	S-24	60.0	27	29	NP	33	Sandy SILT	ML
△	B-1	S-27	67.5	27	26	1	44	SILT with sand	ML
⊗	B-2	S-5	12.5	40	39	1	61	SILT with sand	ML
⊕	B-2	S-9	22.5	45	41	4	67	Sandy SILT	ML

ASTM D 4318 Test Method

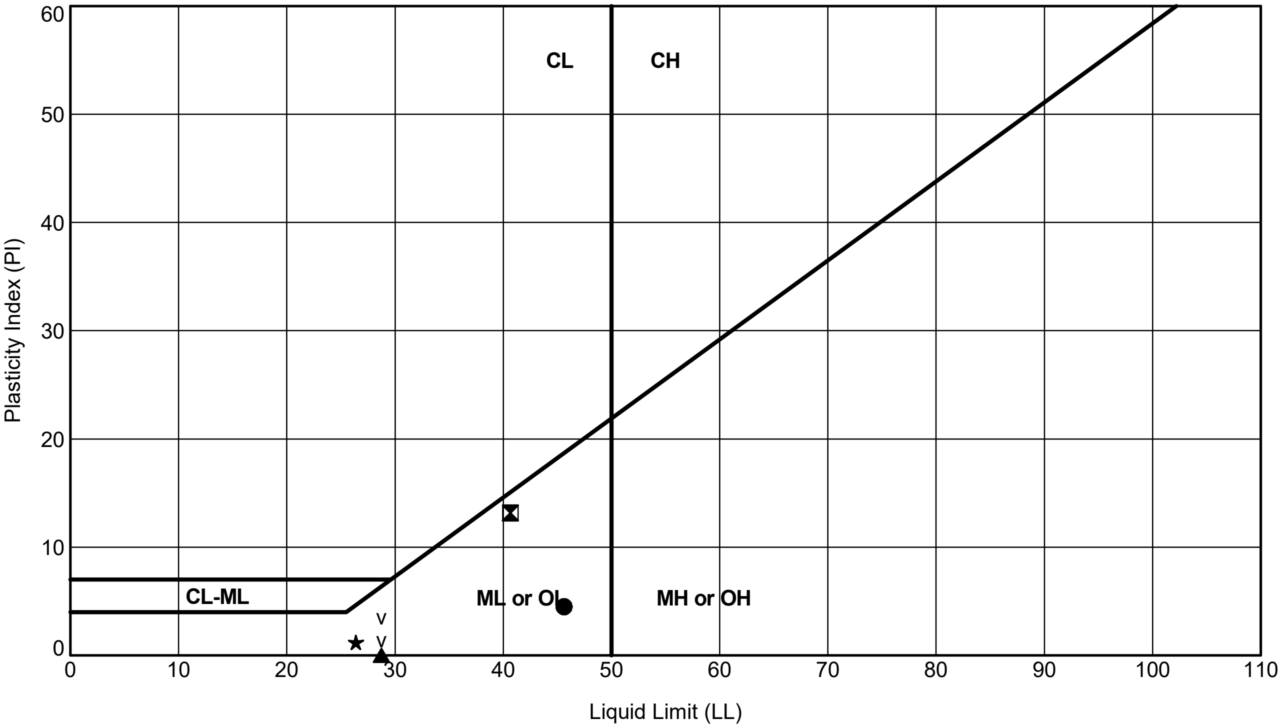
1374022.01 1/12/23 Y:\1374\022.010\T\GINT\1374022.010.GPJ ATTERBERG LIMITS FIGURE

LANDAU ASSOCIATES

McCall Terminal
Upgrade Project
Portland, Oregon

Plasticity Chart

Figure
B5



ATTERBERG LIMIT TEST RESULTS

Symbol	Exploration Number	Sample Number	Depth (ft)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Natural Moisture (%)	Soil Description	Unified Soil Classification
●	B-2	S-18	45.0	46	41	5	51	SILT with sand	ML
⧗	B-2	S-22	55.0	41	28	13	41	SILT	ML
▲	B-2	S-24	60.0	29	29	NP	39	Sandy SILT	ML
★	B-2	S-26	65.0	26	25	1	41	SILT with sand	ML

ASTM D 4318 Test Method

1374022.01 1/12/23 Y:\1374\022.010\T\GINT\1374022.010.GPJ ATTERBERG LIMITS FIGURE

LANDAU
ASSOCIATES

McCall Terminal
Upgrade Project
Portland, Oregon

Plasticity Chart

Figure
B6

McCall Oil and Chemical Corporation – PMA Project

5480 NW Front Ave
Portland, Oregon 97210

Prepared for

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November 21, 2019

PSI Project No. 07041274

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1 PROJECT INFORMATION

1.1 PROJECT AUTHORIZATION

This report presents the results of PSI's geotechnical investigation performed for the proposed polymer modified asphalt (PMA) expansion project at the McCall Oil & Chemical Corporation in Portland, Oregon. The project is located at the existing asphalt-paved petroleum and chemical storage and distribution terminal at 5480 NW Front Avenue in Portland, Oregon. A Vicinity Map of the site location is presented on Figure 1. This investigation was performed for Mr. John Deppa in general accordance with PSI proposal number 0704-290485, dated October 1, 2019.

1.2 PROJECT DESCRIPTION

Project information was provided by Mr. John Deppa of Alpha Technical Group Inc., in an email on September 11, 2019. The provided information included:

- A request for quotation (RFQ), entitled "RFQ-Geotechnical Investigation", and
- The PMA Plant Expansion vicinity map and enlarged plan

In addition, Mr. John Deppa provided the proposed loadings of the proposed tanks and building in an email on November 14, 2019.

Based on the provided information, PSI understands that Alpha Technical Group Inc. is planning the construction of the following items:

1. New asphalt storage tanks and foundations. Five new tanks will be welded steel construction with top mounted mixers and cone bottoms and set on multiple steel posts. Two of the tanks are approximately 14-feet in diameter by approximately 36-feet in height, and three of the tanks are approximately 12 feet in diameter by approximately 24-feet in height.

2. A new PMA equipment storage building.

3. A foundation for the new PMA equipment which is to be mounted on steel skid.

4. A new product piping system along with minor pipe support structures and foundations. This includes minor modifications to existing truck load rack structure and hot oil piping system.

Mr. John Deppa indicated that the new foundations (including tanks) are currently planned to be concrete spread footings unless ground conditions deem them infeasible.

Should any of the above information or design basis 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. PSI will not be held responsible for changes to the project if not provided the opportunity to review the information and provide modifications to our recommendations.

1.3 PURPOSE AND SCOPE-OF-SERVICES

Based on correspondence with Mr. John Deppa, and PSI Proposal # 0704-290485, the purpose of this exploration was to evaluate and understand the subsurface geologic conditions at the site and to develop geotechnical foundation design criteria for support of the five proposed asphalt storage tanks, and the proposed PMA equipment building, and ancillary structures

The scope of the exploration included a reconnaissance of the project site, completion of two soil Cone Penetrometer Tests (CPT) in the truck-accessible gravel area northeast of the proposed tank locations, and two geoprobes in the proposed footprint of the equipment storage building. The project analysis included laboratory testing of samples collected from the geoprobe, an engineering analysis and evaluation of the subsurface materials encountered, and preparation of this report.

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. The client acknowledges that mold is ubiquitous to the environment with mold amplification occurring when building materials are impacted by moisture. The 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.

1.4 FIELD EXPLORATION PROGRAM

PSI completed our field exploration of the project site on October 24, 2019. The scope of the exploration included completion of two soil Cone Penetrometer Tests (CPTu) in the truck-accessible gravel area northeast of the proposed tank locations, and two geoprobes in the proposed footprint of the equipment storage building. The CPTu soundings were designated CPT1 and CPT2 and the geoprobes was designated GP1 and GP2. The exploration location was determined and marked in the field by PSI prior to our exploration, and are represented on Figure 2, *Boring Location Map*. PSI notified Oregon's Utility Notification Center and a private utility locator to locate underground utilities in the vicinity of the proposed exploration locations prior to commencing the field activities. The CPTus were proposed to extend to a depth of 100 feet below ground surface (bgs), but cone refusal was encountered before this depth in both tests. CPT1 was pushed to the depth of cone refusal at 77.5 feet bgs and CPT2 was pushed to the depth of cone refusal at 76 feet bgs. Though one geoprobe to 35 feet bgs was proposed, no soil recovery after 15 feet bgs in geoprobe GP1 was possible. Thus, PSI performed a second geoprobe, GP2, which extended to the proposed depth of 35 feet bgs.

The CPT soundings were performed to observe the stratigraphy, density, and variability of subsurface soil conditions, based on the Soil Behavior Type (SBT) classification. Physical soil samples were recovered from the geoprobes and transported to PSI's laboratory for additional examination and testing. A representative of PSI's geotechnical staff was present during the explorations to record soil and groundwater conditions encountered in the exploration and to obtain soil samples for laboratory testing.

Individual logs of the CPTs and geoprobes are presented in Appendix A. It should be noted that the subsurface conditions presented on the log is representative of the conditions at the specific locations drilled. Variations may occur and should be expected across the site. The soil morphology represents the approximate boundary between subsurface materials and the transitions may be gradual and indistinct. Water level information obtained during our field operations is also shown on the boring logs. Elevations referenced were obtained via Google Earth and should be considered approximations.

Sampling Procedures

Throughout the drilling operations, soil samples were obtained from the boring using a 2.3-inch DT 22 geoprobe sampler. The samplers were driven into the soil a total distance of 60 inches using a hydraulic percussion hammer. The rig used to drive the geoprobe was a 20CPT Press. All sampling methods were performed in accordance with the current standard of practice for sampling with geoprobes.

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. The CPTu records are presented in Appendix A.

During a CPTu, the electric cone (tip angle 60°, section area 10 cm^2) and the sounding rods are pushed continuously into the ground. Intermittent measurements of the cone resistance (qc), sleeve friction (fs), and pore pressure (u_2) 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:

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

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.

Field Classification

Soil samples were initially classified visually in the field. Consistency, color, relative moisture, degree of plasticity, peculiar odors, and other distinguishing characteristics of the soil samples were noted. The terminology used in the soil classifications and other modifiers are depicted in the General Notes and Soil Classification Chart in Appendix A.

1.5 LABORATORY TESTING PROGRAM AND PROCEDURES

Soil samples obtained during the field explorations were examined in our laboratory. The physical characteristics of the samples were noted, and the field classifications were modified, where necessary. Representative samples were selected during the course of the examination for further testing. The laboratory test procedures are summarized below, and test data is provided on the boring logs in Appendix A and in the lab test data in Appendix B.

Moisture Content

Natural moisture content determinations were made on selected soil samples. The natural moisture content is defined as the ratio of the weight of water to the dry weight of soil, expressed as a percentage.

Visual-Manual Classification

The soil samples were classified in general accordance with guidelines presented in ASTM D2488. Certain terminology incorporating current local engineering practice, as provided in the Soil Classification Chart, included with, or in lieu of, ASTM terminology. The term which best described the major portion of the sample was used in determining the soil type (i.e., gravel, sand, silt or clay).

Sieve Analysis by Washing

The determination of the amount of material finer than the U.S. Standard No. 200 (75-µm) sieve was made on selected soil samples in general accordance with guidelines presented in ASTM C117. In general, the sample is dried in an oven and then washed with water over the No. 200 sieve. The mass retained on the No. 200 sieve is dried in an oven, and the dry weight recorded. Results from this test procedure assist in determining the fraction, by weight, of coarse-grained and fine-grained soils in the sample.

The determination of the gradation curve of the coarse-grained material was made on selected soil samples in general accordance with guidelines presented in ASTM C136. In general, the oven dried mass retained on the No. 200 sieve is passed over progressively smaller sieve openings, by agitating the sieves by hand or by a mechanical apparatus. The mass retained on each sieve is recorded as a fraction of the total sample, including the percent passing the No. 200 sieve.

Atterberg Limits

The Atterberg Limits (ASTM D-4318) are defined by the liquid limit (LL) and plastic limit (PL) states of a given soil. These limits are used to determine the moisture content limits where the soil characteristics change from behaving more like a fluid on the liquid limit end to where the soil behaves more like individual soil particles on the plastic limit end. The plasticity index (PI) is the difference between the liquid limit and the plastic limit. The plasticity index is used in conjunction with the liquid limit to assess if the material will behave like a silt or clay. The results of the Atterberg Limit tests, which include liquid and plastic limits, are plotted on the boring logs.

2 SITE AND SUBSURFACE CONDITIONS

2.1 SITE DESCRIPTION

The existing asphalt, petroleum and chemical storage and distribution terminal of the McCall Oil & Chemical Corporation is located at 5480 NW Front Ave in Portland, Oregon. Based on a site visit conducted by PSI and Alpha Technical Group Inc. on September 30, 2019, the proposed project location is in a heavily industrialized area near the banks of the Willamette River. The polymer modified asphalt (PMA) expansion project will consist of:

- Constructing new asphalt storage tanks and foundations
- A new PMA equipment storage building and footings
- A new product piping system with minor pipe support structures and footings

2.2 TOPOGRAPHY

Based on the available topographic information on Google Earth, the project site is relatively flat in the area of proposed construction with elevations varying between approximately 25 to 28 feet, mean sea level (msl).

2.3 GEOLOGY

Based on a review of available geologic information, the site geology at the McCall Oil & Chemical Corporation terminal consists of predominately Quaternary surficial deposits. These deposits can include alluvium, colluvium, river and coastal terrace, landslide, glacial, eolian, beach, lacustrine, playa and pluvial lake deposits, and outburst flood deposits left by the Missoula and Bonneville floods. Man-made fill deposits consisting of mixed grained sediments are also present in the area.

2.3.1 LOCAL FAULTING AND SEISMIC DESIGN PARAMETERS

PSI has reviewed the USGS Quaternary Fault and Fold Database of the United States. Table 1 summarizes distance and names of the closest mapped faults within about 25 miles of the project site.

Table 1 - Summary of Published, Nearby Faults

Fault Name	Approximate Distance (miles) and Direction from the Site
Portland Hills Fault	0.32, west
East Bank Fault	1.02, east
Oatfield Fault	2.87, west
Beaverton Fault Zone	7.04, southeast
Helvatia Fault	9.12, west
Canby-Molalla Fault	9.55, south
Damascus-Tickle Creek Fault Zone	11, southeast
Grant Butte Fault	11.14, southeast
Lacamas Lake Fault	13.59, northeast
Gales Creek Fault Zone	19.95, west
Nerberg Fault	23.21, southwest

As part of the procedure to evaluate seismic forces, the 2019 OSSC requires the evaluation of the Seismic Site Class, which categorizes the site based upon the characteristics of the subsurface profile within the upper 100 feet of the ground surface. Based on the obtained soil data of the site, the seismic site class classifies as a site class "D" soil; however since more than 10 feet of potentially liquefiable soil exists on the site, the site classifies as a Site Class "F" as defined in Table 20.3-1 of ASCE 07-16. However, the exception in Section 20.3.1 of ASCE 07-16 permits the Site Class to be determined in accordance with Section 20.3 and the corresponding values of Fa and Fv determined from Tables 11.4-1 and 11.4-2 provided the fundamental period of the structure is less than ½ second. Based on this exception, Site Class D seismic design coefficients can be used and are provided below. The structural engineer should confirm this exception is applicable. The associated ASCE 7-16 probabilistic ground acceleration values and site coefficients for the general site area were obtained from the Structural Engineers Association California Seismic Design Map Webpage. The risk targeted seismic values and coefficient are presented in Table 2.

Table 2 - Seismic Design Parameters
(45.5629 °, -122.7353°) – SITE CLASS "D"

Period (seconds)	Mapped Spectral Acceleration Parameters (g)	Site Coefficients	Adjusted Spectral Acceleration Parameters (g)	Design Spectral Acceleration Parameters (g)	Period, T (sec)
0.0 (PGA)	PGA = 0.403	F_{PGA} = 1.197	PGA_M = 0.482	---	---
0.2 (S_s)	S_s = 0.891	F_a = 1.102	S_{ms} = 1.096	S_{Ds} = 0.731	T_0 = 0.124
1.0 (S_1)	S_1 = 0.405	F_v = 1.895	S_{m1} = 0.767	S_{D1} = 0.511	T_s = 0.699

Notes:
PGA_M = Maximum considered earthquake geometric mean peak ground acceleration adjusted for Site Class effects
FPGA = PGA site coefficient.
PGA_M = Maximum considered earthquake geometric mean peak ground acceleration adjusted for Site Class effects
S_S = Short period (0.2 second) Mapped Spectral Acceleration
S_1 = 1.0 second period Mapped Spectral Acceleration
S_{MS} = Spectral Response adjusted for site class effects for short period = $F_a \bullet S_S$
S_{M1} = Spectral Response adjusted for site class effects for 1-second period = $F_v \bullet S_1$
S_{DS} = Design Spectral Response Acceleration for short period = $2/3 \bullet S_{MS}$
S_{D1} = Design Spectral Response Acceleration for 1-second period =$2/3 \bullet S_{M1}$
F_a = Short Period Site Coefficients
F_v = Long Period Site Coefficients
$T_0 = 0.2 \bullet S_{D1} / S_{Ds}$
$T_s = S_{D1} / S_{Ds}$

In accordance with ASCE 7-16, Chapter 11, for a Site Class D site with a S_1 >0.1, the F_V value is governed by Section 11.4.8. Section 11.4.8. requires a ground motion hazard analysis be performed for structures with seismic isolation elements or dampening systems on all sites with S_1 >0.6 or Site Class D with S_1 >0.2. Also, in section 11.4.8 is the exception that the ground motion hazard analysis "is not required for structures other than seismically isolated structures and structures with dampening systems where:" the seismic response coefficient of the structure (chapter 12) meets specific requirements.

This report is based on no seismic isolation elements or dampening systems being installed. As such, the long period site coefficient, F_v values from Table 11.4-2 for the Code supplied S_1 values at this location are supplied in this report.

2.4 SUBSURFACE CONDITIONS

Gravel fill was encountered at the surface of both GP1 and GP2 and extended to a depth of approximately 8 inches bgs. This gravel was underlain with a layer of black sand with silt which extended to a depth of 1.5 feet bgs. This sand layer was underlain by a layer of dark gray lean clay which extended to a depth of 2.5 feet bgs. This clay layer was underlain by a layer of brown sand with silt in GP1, and silty sand in GP2. In both geoprobes, this sand layer extended to a depth of 14.5 feet bgs. GP1 was terminated at 15 feet bgs in a layer of black gravel with silt and sand because no recovery was possible after 15 feet bgs. GP2 extended into this silty gravel with sand layer, which extended to a depth of 20 feet bgs. This gravel layer was underlain by dark gray lean clay with sand, which extended to a depth of 34 feet bgs. GP2 was terminated in a layer of silty sand at depth of 35 feet bgs. The CPT Soil Behavior Type (SBT) and geoprobe physical samples generally matched to the termination depth of the geoprobes at 15 feet bgs for GP1 and 35 feet bgs for GP2. The CPT soil behavior type in both CPT1 and CPT2 suggest that clay and silty clay extend to the explored depth of 77.5 feet bgs. The logs indicate that this clay generally increases in stiffness with depth. Both CPTs were terminated due to cone refusal at depths of 77.5 and 76 in CPT1 and CPT2, respectively.

The subsurface profile described above is a generalized interpretation provided to highlight the major subsurface stratification features and material characteristics. The logs in Appendix A should be reviewed for more specific information. This record includes soil description, stratifications, penetration resistances, location of samples, and laboratory test data. The stratifications shown on the logs represent the conditions only at each of the exploration locations. The stratifications indicated on the logs represent the approximate boundary between subsurface materials. The actual transitions may be gradual. Subsurface soils and conditions may vary across relatively short distances at the site and may become apparent with additional explorations or excavation. If soil conditions are found to be different than those described herein, PSI should be allowed to reevaluate our recommendations, if necessary.

2.5 GROUNDWATER

Free groundwater was observed at the site and was measured at a depth of approximately 7 feet bgs in both GP1 and GP2 at the time of our field investigation. Pore pressure dissipation tests performed in CPT1 and CPT2 suggest the static ground water level is at a depth of approximately 7.5 feet bgs, generally matching the results of the geoprobes. PSI anticipates that the groundwater table fluctuates seasonally and in response to significant precipitation events. If groundwater flows during construction are encountered at shallower depths, PSI should be notified to assist in determining the proper course of action.

2.6 LIQUEFACTION POTENTIAL

In general, liquefaction is a condition where soils lose intergranular strength due to abrupt increases in pore water pressure. Pore water pressure increases typically occur during dynamic loading such as ground shaking during a seismic event. Liquefaction, should it occur on a site, can induce ground settlement and lateral spreading, which can result in damage to the structures. For liquefaction to occur, the following conditions must be present:

- The soil sediments must be in saturated or near-saturated conditions. At least 80-85 percent saturation is generally considered necessary for the liquefaction to occur.
- The soil must be predominately composed of non-plastic material such as sand or silt or low plasticity materials with a PI of Less than 18.
- The soil must be in a relatively loose state.
- The soil must be subjected to dynamic loading, such as an earthquake.

The site is mapped as having a high liquefaction potential, based on the Oregon Department of Geology and Mineral Industries (DOGAMI HazVu).

An estimated liquefaction settlement analysis has been performed based on conservative modeling equations and parameters utilizing CLiq v2.1.6.11. Results of our studies indicate that the soils from approximately surface grade to 18 to 20 feet bgs would liquefy under an earthquake of magnitude 7.44 based on the moment magnitude (MCEr) and peak ground acceleration (PGAm) of 0.48. The magnitude was based on the mean for all sources of the deaggregation of the USGS PSHA, for the Dynamic: Conterminous U.S., for an earthquake with a 2% occurrence in 50 years. The peak ground acceleration was based on the site modified peak ground acceleration (PGAm) from ASCE 7-16.

The analysis was performed in CLiq using 5 models including Robertson (NCEER 2001), Robertson (2009), Idriss and Boulanger (2008), Moss se al. (2006) and Boulanger and Idriss (2014), yielding liquefaction induced vertical settlements ranging from approximately 1.3 inches to 2.7 inches. PSI recommends using the Idriss and Boulanger (2008) model as a conservative estimate at approximately 2.7 inches vertical settlement. Appendix C shows the results of the Idriss and Boulanger (2008) liquefication assessment and outlines the evaluation procedures for the models discussed.

Based on our analysis of the soils encountered during our investigation, some of the soils encountered are susceptible to liquefaction, with a potential for liquefaction-induced settlement on the order of approximately 2½ inches with liquefaction predicted to occur from near the surface and extending down to a depth of approximately 20 feet bgs. Based on the subsurface data, collected from the CPT and geoprobe exploration locations, PSI anticipates differential liquefaction induced settlements to be on the order of approximately just under 1 inch over a 40-foot span for conventional spread footing foundations. The structural engineer should determine if the structure can adequately withstand this potential settlement and distortion.

2.7 HAZARD DISCUSSION

The following table presents a qualitative assessment of these issues considering the site class, the subsurface soil properties, the groundwater elevation, and probabilistic ground motions:

Table 3 – Qualitative Seismic Site Assessments

Liquefaction	High	The area is mapped as being in a zone with a High Liquification Hazard. PSI agrees with this assessment based on our liquefaction analysis.
Earthquake Shaking	Severe	The area is mapped as being in a zone of Severe Earthquake Shaking.
Slope Stability	Low	The site and neighboring properties are relatively flat and mapped in an area with low risk of landslide.
Surface Rupture	Moderate	No known active faults underlie the site, but the site is 1/3 mile from the Portland Hill Fault.
Flooding	Low	The site and neighboring properties are located outside the effective FEMA 100-Year Flood, but the boundary of the FEMA 100-Year Flood is within ¼ mile of the site.

From the Oregon HazVu: Statewide Geohazards websites (https://gis.dogami.oregon.gov/hazvu/)

3 CONCLUSIONS AND RECOMMENDATIONS

The following geotechnical recommendations have been developed based on the subsurface conditions encountered in the borings and PSI's understanding of the proposed site additions. In PSI's opinion, based on an evaluation of the data obtained from the soil borings, the proposed site is suitable for construction of the new additions, provided the geotechnical engineering recommendations in this report are followed.

3.1 SITE PREPARATION

PSI recommends that loose, soft, or otherwise unsuitable soils at the project site be stripped and removed from structural areas. Buried piping and utilities, if encountered, must be completely removed and rerouted from below proposed building foundations. Should below-grade pipes remain, a risk of seepage or underground soil erosion may occur in the future. Concrete structures and remnants of previous structures encountered during site excavation and site construction operations should be completely removed beneath planned foundations.

After the surficial materials have been stripped and completely removed from proposed development areas, PSI should observe the subgrade to identify any loose/soft or unsuitable areas. Any undocumented or uncontrolled fill should be completely removed, cleaned of any debris, and replaced as engineered fill. Where loose, soft or otherwise unsuitable soils are identified within structural areas of the project, these soils should be completely removed and replaced with structural fill. The Contractor should provide a contingency for the repair of loose, soft or otherwise unsuitable areas identified by the Geotechnical Engineer. Geotextile fabric or geotextile grid may be utilized to provide stabilization of the subgrade.

A proof roll using a fully loaded tandem-axle truck should be performed on finished subgrade elevations to identify any loose, soft or unsuitable areas of subgrade. Loose, soft or otherwise unsuitable soils in these areas should be over-excavated and replaced with properly placed and properly compacted structural fill.

3.2 WET WEATHER CONSTRUCTION

It has been PSI's experience that during warm, dry weather, the moisture content of the upper few feet of soil will decrease. However, below the upper 2 to 3 feet, the moisture content of the soil tends to remain relatively unchanged and often well above the optimum moisture content for compaction.

As a result, the subcontractor must use care to protect exposed subgrade from disturbance by construction traffic, particularly during wet weather. The Contractor 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 with the concurrent placement of granular work pads tends to minimize the potential for subgrade disturbance.

3.3 EXCAVATION CONSIDERATIONS

Open excavations exceeding four feet are not anticipated; however, if they do occur, excavations should be performed in accordance with OSHA regulations as stated in 29 CFR Part 1926. The contractor is solely responsible for designing and constructing stable, temporary excavations and should shore, slope, or bench the sides of the excavations as required to maintain stability of both the excavation sides and bottom. The contractor should evaluate the soil exposed in the excavations as part of the required safety procedures. In no case should slope height, slope inclination, or excavation depth, including utility trench excavation depth, exceed those specified by local, state, and federal safety regulations. PSI is providing this information solely as a service to our client. PSI does not assume responsibility for construction site safety or the contractor's or other parties' compliance with local, state, and federal safety or other regulations.

During wet weather, earthen berms or other methods should be used to prevent runoff water from entering the excavations. The bottom of the excavations should be sloped to a collection point. Collected water within the foundation and utility trench excavations should be discharged to a suitable location outside the construction limits.

3.4 STRUCTURAL FILL MATERIALS

PSI should observe the subgrade prior to placing structural fill or structures to document the subgrade condition and stability. 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.

Proper control of placement and compaction of new fills should be monitored by PSI. Structural fill should be placed in lifts not exceeding 1-foot for large compaction equipment such as vibratory rollers or hoe-packs, but thinner lifts may be necessary if small compaction equipment such as jumping jacks or plate compactors are to be used. 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, as determined by the geotechnical engineer, should be performed on each lift of fill.

Tested structural fill materials that do not achieve the required dry density 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.

Re-Use of Native Soils

The on-site lean clay and silt soils may be considered for re-use as structural fill provided they can be suitably moisture conditioned to meet compaction requirements. Onsite soils to be reused should be absent of deleterious materials (e.g., construction debris, organics) and have particle sizes of no greater than 3 inches. It has been our experience that when fine grained soils are outside of optimum moisture content, they may be difficult to properly moisture condition. Special care should be taken if these materials are to be re-used, especially during wet-weather conditions as they may become difficult, if not impossible, to compact.

During construction, the Geotechnical Engineer should confirm the acceptability of soils onsite for the re-use as structural fill.

Imported Structural Fill

If imported structural fill is to be utilized, it should consist of pit-run or quarry-run rock, crushed rock, crushed gravel, or sand. The material should be well-graded between coarse and fine material, angular, have a plasticity index of 8 or less, and have less than 10 percent by weight passing the U.S. Standard No. 200 Sieve (75-µm).

Drain Rock

Drain rock, or “free-draining” material should have less than 2% passing the No. 200 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.

3.5 SURFACE DRAINAGE AND EROSION CONTROL

Site grading should be carefully planned to promote positive drainage away from the structures and to divert surface water away from the site. Water should not be allowed to collect near the foundations either during or after construction.

3.6 FOUNDATIONS

Due to the presence of liquefiable soils below the proposed tanks and building, PSI recommends that each of the five tanks be supported by a singular mat foundation. PSI recommends that the building and ancillary structures be supported by either spread footings or a continuous footing.

3.6.1. Foundation System for Proposed Tanks

PSI believes a concrete mat foundation beneath each proposed tank could be used to support each proposed tank while mitigating anticipated liquefaction induced settlement. Mat foundations can be used distribute the entire footprint of a structure onto a single footing which can help limit differential settlements.

PSI recommends that the footing excavations be observed and documented by PSI's Geotechnical Engineer or designated technical representative prior to placement of structural fill, concrete, or reinforcing steel to verify their suitability for foundation support.

Allowable Bearing Pressure

The following calculations are based on each 12-foot diameter and 14-foot diameter tank being supported on separate 15-foot by 15-foot square mat foundations. The calculations are also based on the proposed tank loading of a 260-kip product load and a 2.5-kip dead load. The mats should be sufficiently reinforced so that they perform as rigid bodies. The mat foundation should be founded a minimum of 1.5 feet below the lowest adjacent subgrade prepared in accordance with Section 3.1 of this report, with at least one foot of overexcavation. The bearing capacity of mats are often not the governing criteria for design; the settlement of the mat usually governs the allowable load on the mats. Based on our calculations, bearing pressures of up to 1,500 psf may be utilized. Per ASCE 7, allowable bearing pressures can be increased by 33% for lateral loads, including wind and seismic loads. Based on this bearing pressure, static settlements on the order of ½-inch can be expected. Differential settlements for both static and seismic conditions combined will be on the order of ¾-inch over 20 feet. Please note that the use of a mat foundation will not reduce total settlements (combined static and seismic) and that flexible connections for utilities to the onsite structure are recommended to account for the anticipated settlements.

3.6.2. Foundation System for Proposed Building

PSI believes that either spread footings or continuous footings can be used to support the proposed PMA equipment storage building.

PSI recommends that the footing excavations be observed and documented by PSI's Geotechnical Engineer or designated technical representative prior to placement of structural fill, concrete, or reinforcing steel to verify their suitability for foundation support.

Allowable Bearing Pressure

For spread footings, the following calculations are based on 3-foot by 3-foot square spread footings at each column load. The calculations are also based on the proposed combined dead and roof live/snow column load of 25 kips. The spread footings should be founded a minimum of 1.5 feet below the lowest adjacent subgrade prepared in accordance with section 3.1 of this report. Based on our calculations, bearing pressures of up to 3,500 psf may be utilized. Per ASCE 7, allowable bearing pressures can be increased by 33% for lateral loads, including wind and seismic loads.

For continuous footings, the following calculations are based on a 1.5-foot-wide continuous footing around the perimeter of the building. The calculations are also based on the proposed combined dead and roof live/snow column load of 25 kips. The continuous footings should be founded a minimum of 1.5 feet below the lowest adjacent subgrade prepared in accordance with section 3.1 of this report. Based on our calculations, bearing pressures of up to 2,600 psf may be utilized. Per ASCE 7, allowable bearing pressures can be increased by 33% for lateral loads, including wind and seismic loads.

Based on these bearing pressures, static settlements on the order of 1-inch can be expected. Differential settlements for both static and seismic conditions combined will be on the order of ¾-inch over 20 feet. Please note that the use spread footings and continuous footings will not reduce total settlements (combined static and seismic) and that flexible connections for utilities to the onsite structure are recommended to account for the anticipated settlements.

The ancillary structures should be founded below frost depth, at a minimum of 1.5 feet and based on our calculations, bearing pressures on strip footings of up to 2,500 psf may be utilized. Per ASCE 7, allowable bearing pressures can be increased by 33% for lateral loads, including wind and seismic loads.

3.6.3. Lateral Earth Pressure

Resistance to lateral loads can be provided by passive earth pressure against the side of mat foundations, spread footings or continuous footings, and by friction at the base.

The following soil parameters are applicable for general design of foundation systems. Below-grade structures should be designed to resist lateral earth pressures. Lateral earth pressure is developed from the soils present within a wedge formed by the vertical below-grade structure and an imaginary line extending up and away from the bottom of the structure at an approximate 45° angle. The lateral earth pressures are determined by multiplying the vertical applied pressure by the appropriate lateral earth pressure coefficient K. If the structures are rigidly attached to the structure and not free to rotate or deflect at the top, PSI recommends designing the structures for the “at-rest” lateral earth pressure condition using K_o. Structures that are permitted to rotate and deflect at the top can be designed for the active lateral earth pressure condition using K_a. PSI understands that passive earth pressure is requested for design. Passive pressure can be determined using K_p, with a factor of safety of 2.0 to limit strain. Recommended parameters for use in below grade foundations are as follows:

Table 4 - Recommended Parameters for use in Foundation Design	
Material Type	Drained Friction Angle (ϕ')
Native Sand	32°
Parameters specific to soil type	**Native Lean Clay**
Friction Factor for Base	0.42
Coefficient of Active Pressure (K_a) **	0.31
Coefficient of Passive Pressure (K_p) **	3.25
Coefficient of At-Rest Pressure (K_o) **	0.47

** Earth pressure coefficients valid for level backfill conditions with no surcharge

Horizontal forces can be resisted partially or completely by frictional forces developed between the base of the spread footings and the underlying 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. We recommend assuming an ultimate coefficient of friction value of 0.41 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 assumes that backfill around footings will be placed as granular structural fill.

Friction should be applied to net dead normal load only. A minimum factor of safety of 1.5 and 1.1 should be used for sliding resistance for static and seismic cases, respectively. If passive pressure and friction are combined when evaluating the lateral resistance of a mat foundation, a factor of safety of 1.5 should be used to reduce the contribution from passive pressure.

3.7 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.

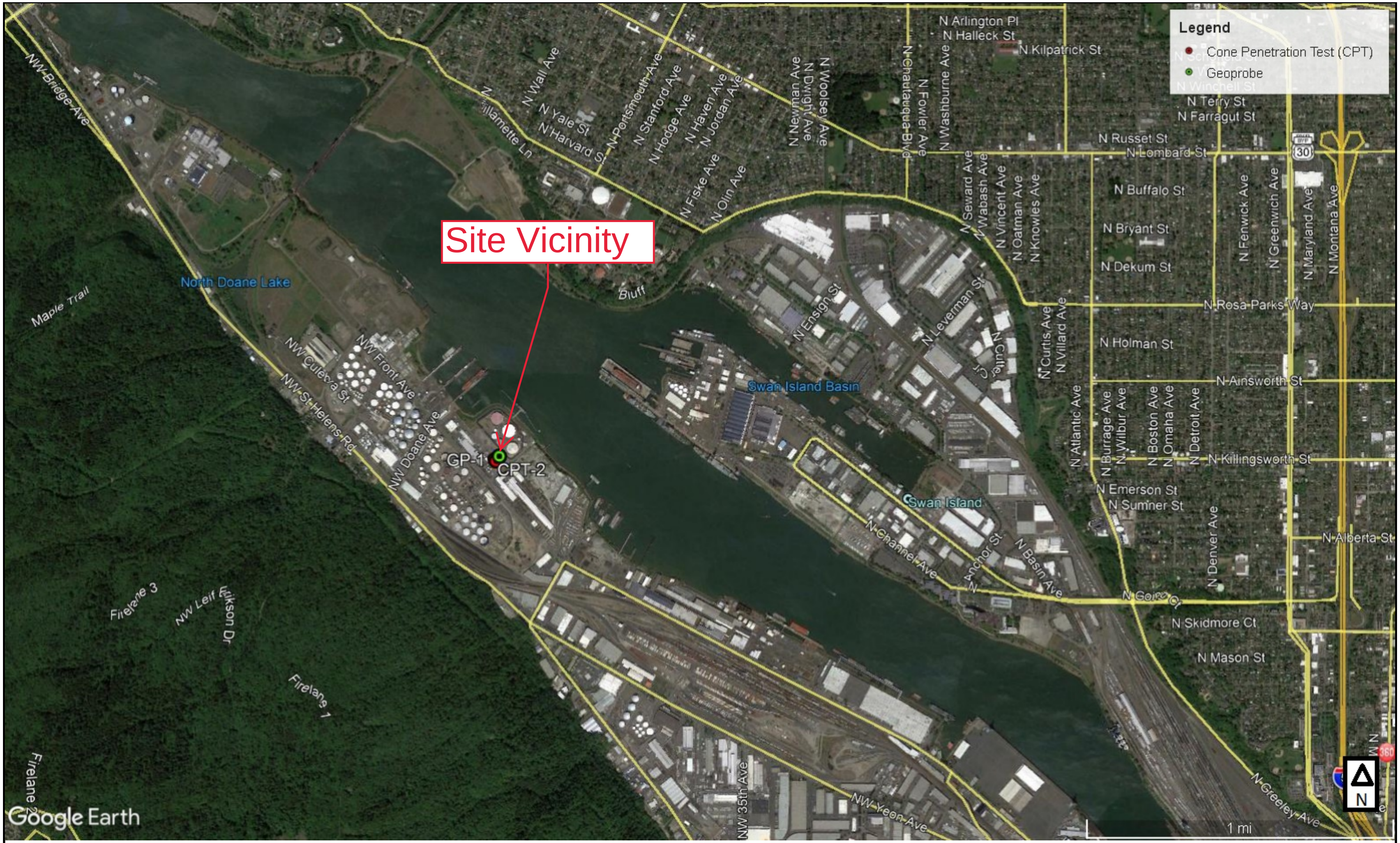
4 GEOTECHNICAL RISK AND REPORT LIMITATIONS

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 pavement section 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 pavement section 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 are based on the available subsurface information obtained by PSI, and information provided by Mr. John Deppa. 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 should be notified immediately to determine if changes in the 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 Geotechnical Engineer should be retained and provided the opportunity to review the final design plans and specifications to check that our engineering recommendations have been properly incorporated into the design documents. At that time, it may be necessary to submit supplementary recommendations. This report has been prepared for the exclusive use of Mr. John Deppa and his design consultants for the specific application to the proposed polymer modified asphalt (PMA) expansion project at the McCall Oil & Chemical Corporation located at 5480 NW Front Avenue in Portland, Oregon.

FIGURES



Taken From Google Earth

McCall Oil and Chemical Corporation - PMA Project
Portland, Oregon

JOB NO. 07041274

Site Vicinity Map

FIGURE NO. 1

Taken From Google Earth

intertek psi

McCall Oil and Chemical Corporation - PMA Project Portland, Oregon		JOB NO.	07041274
CPT and Geoprobe Locations		FIGURE NO.	2

APPENDIX A – SOIL INVESTIGATION LOGS, GENERAL NOTES, AND SOIL CLASSIFICATION CHART

GENERAL NOTES

SAMPLE IDENTIFICATION

The Unified Soil Classification System (USCS), AASHTO 1988 and ASTM designations D2487 and D-2488 are used to identify the encountered materials unless otherwise noted. Coarse-grained soils are defined as having more than 50% of their dry weight retained on a #200 sieve (0.075mm); they are described as: boulders, cobbles, gravel or sand. Fine-grained soils have less than 50% of their dry weight retained on a #200 sieve; they are defined as silts or clay depending on their Atterberg Limit attributes. Major constituents may be added as modifiers and minor constituents may be added according to the relative proportions based on grain size.

DRILLING AND SAMPLING SYMBOLS

SFA: Solid Flight Auger - typically 4" diameter flights, except where noted.
HSA: Hollow Stem Auger - typically 3¼" or 4¼ I.D. openings, except where noted.
M.R.: Mud Rotary - Uses a rotary head with Bentonite or Polymer Slurry
R.C.: Diamond Bit Core Sampler
H.A.: Hand Auger
P.A.: Power Auger - Handheld motorized auger

☒ SS: Split-Spoon - 1 3/8" I.D., 2" O.D., except where noted.
■ ST: Shelby Tube - 3" O.D., except where noted.
▯ RC: Rock Core
↓ TC: Texas Cone
✋ BS: Bulk Sample
▩ PM: Pressuremeter
CPT-U: Cone Penetrometer Testing with Pore-Pressure Readings

SOIL PROPERTY SYMBOLS

N: Standard "N" penetration: Blows per foot of a 140 pound hammer falling 30 inches on a 2-inch O.D. Split-Spoon.
N_{60}: A "N" penetration value corrected to an equivalent 60% hammer energy transfer efficiency (ETR)
Q_u: Unconfined compressive strength, TSF
Q_p: Pocket penetrometer value, unconfined compressive strength, TSF
w%: Moisture/water content, %
LL: Liquid Limit, %
PL: Plastic Limit, %
PI: Plasticity Index = (LL-PL),%
DD: Dry unit weight, pcf
▼,▽,▼ Apparent groundwater level at time noted

RELATIVE DENSITY OF COARSE-GRAINED SOILS

Relative Density	N - Blows/foot
Very Loose	0 - 4
Loose	4 - 10
Medium Dense	10 - 30
Dense	30 - 50
Very Dense	50 - 80
Extremely Dense	80+

ANGULARITY OF COARSE-GRAINED PARTICLES

Description	Criteria
Angular:	Particles have sharp edges and relatively plane sides with unpolished surfaces
Subangular:	Particles are similar to angular description, but have rounded edges
Subrounded:	Particles have nearly plane sides, but have well-rounded corners and edges
Rounded:	Particles have smoothly curved sides and no edges

GRAIN-SIZE TERMINOLOGY

Component	Size Range
Boulders:	Over 300 mm (>12 in.)
Cobbles:	75 mm to 300 mm (3 in. to 12 in.)
Coarse-Grained Gravel:	19 mm to 75 mm (¾ in. to 3 in.)
Fine-Grained Gravel:	4.75 mm to 19 mm (No.4 to ¾ in.)
Coarse-Grained Sand:	2 mm to 4.75 mm (No.10 to No.4)
Medium-Grained Sand:	0.42 mm to 2 mm (No.40 to No.10)
Fine-Grained Sand:	0.075 mm to 0.42 mm (No. 200 to No.40)
Silt:	0.005 mm to 0.075 mm
Clay:	<0.005 mm

PARTICLE SHAPE

Description	Criteria
Flat:	Particles with width/thickness ratio > 3
Elongated:	Particles with length/width ratio > 3
Flat & Elongated:	Particles meet criteria for both flat and elongated

RELATIVE PROPORTIONS OF FINES

Descriptive Term	% Dry Weight
Trace:	< 5%
With:	5% to 12%
Modifier:	>12%

GENERAL NOTES

(Continued)

CONSISTENCY OF FINE-GRAINED SOILS

Q_U - TSF	N - Blows/foot	Consistency
0 - 0.25	0 - 2	Very Soft
0.25 - 0.50	2 - 4	Soft
0.50 - 1.00	4 - 8	Firm (Medium Stiff)
1.00 - 2.00	8 - 15	Stiff
2.00 - 4.00	15 - 30	Very Stiff
4.00 - 8.00	30 - 50	Hard
8.00+	50+	Very Hard

MOISTURE CONDITION DESCRIPTION

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

RELATIVE PROPORTIONS OF SAND AND GRAVEL

Descriptive Term	% Dry Weight
Trace:	< 15%
With:	15% to 30%
Modifier:	>30%

STRUCTURE DESCRIPTION

Description	Criteria
Stratified:	Alternating layers of varying material or color with layers at least ¼-inch (6 mm) thick
Laminated:	Alternating layers of varying material or color with layers less than ¼-inch (6 mm) thick
Fissured:	Breaks along definite planes of fracture with little resistance to fracturing
Slickensided:	Fracture planes appear polished or glossy, sometimes striated

Description	Criteria
Blocky:	Cohesive soil that can be broken down into small angular lumps which resist further breakdown
Lensed:	Inclusion of small pockets of different soils
Layer:	Inclusion greater than 3 inches thick (75 mm)
Seam:	Inclusion 1/8-inch to 3 inches (3 to 75 mm) thick extending through the sample
Parting:	Inclusion less than 1/8-inch (3 mm) thick

SCALE OF RELATIVE ROCK HARDNESS

Q_U - TSF	Consistency
2.5 - 10	Extremely Soft
10 - 50	Very Soft
50 - 250	Soft
250 - 525	Medium Hard
525 - 1,050	Moderately Hard
1,050 - 2,600	Hard
>2,600	Very Hard

ROCK BEDDING THICKNESSES

Description	Criteria
Very Thick Bedded	Greater than 3-foot (>1.0 m)
Thick Bedded	1-foot to 3-foot (0.3 m to 1.0 m)
Medium Bedded	4-inch to 1-foot (0.1 m to 0.3 m)
Thin Bedded	1¼-inch to 4-inch (30 mm to 100 mm)
Very Thin Bedded	½-inch to 1¼-inch (10 mm to 30 mm)
Thickly Laminated	1/8-inch to ½-inch (3 mm to 10 mm)
Thinly Laminated	1/8-inch or less "paper thin" (<3 mm)

ROCK VOIDS

Voids	Void Diameter
Pit	<6 mm (<0.25 in)
Vug	6 mm to 50 mm (0.25 in to 2 in)
Cavity	50 mm to 600 mm (2 in to 24 in)
Cave	>600 mm (>24 in)

GRAIN-SIZED TERMINOLOGY

(Typically Sedimentary Rock)

Component	Size Range
Very Coarse Grained	>4.76 mm
Coarse Grained	2.0 mm - 4.76 mm
Medium Grained	0.42 mm - 2.0 mm
Fine Grained	0.075 mm - 0.42 mm
Very Fine Grained	<0.075 mm

ROCK QUALITY DESCRIPTION

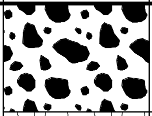
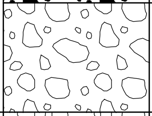
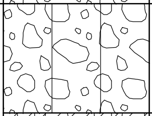
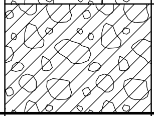
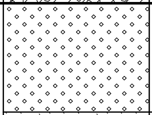
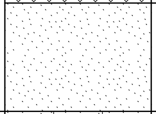
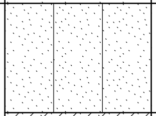
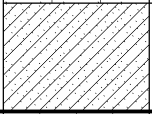
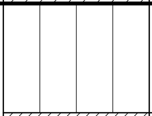
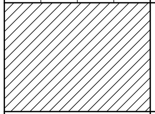
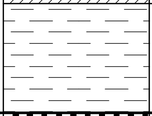
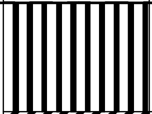
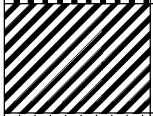
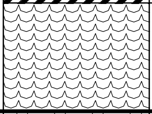
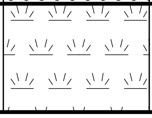
Rock Mass Description	RQD Value
Excellent	90 -100
Good	75 - 90
Fair	50 - 75
Poor	25 -50
Very Poor	Less than 25

DEGREE OF WEATHERING

Slightly Weathered:	Rock generally fresh, joints stained and discoloration extends into rock up to 25 mm (1 in), open joints may contain clay, core rings under hammer impact.
Weathered:	Rock mass is decomposed 50% or less, significant portions of the rock show discoloration and weathering effects, cores cannot be broken by hand or scraped by knife.
Highly Weathered:	Rock mass is more than 50% decomposed, complete discoloration of rock fabric, core may be extremely broken and gives clunk sound when struck by hammer, may be shaved with a knife.

SOIL CLASSIFICATION CHART

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

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

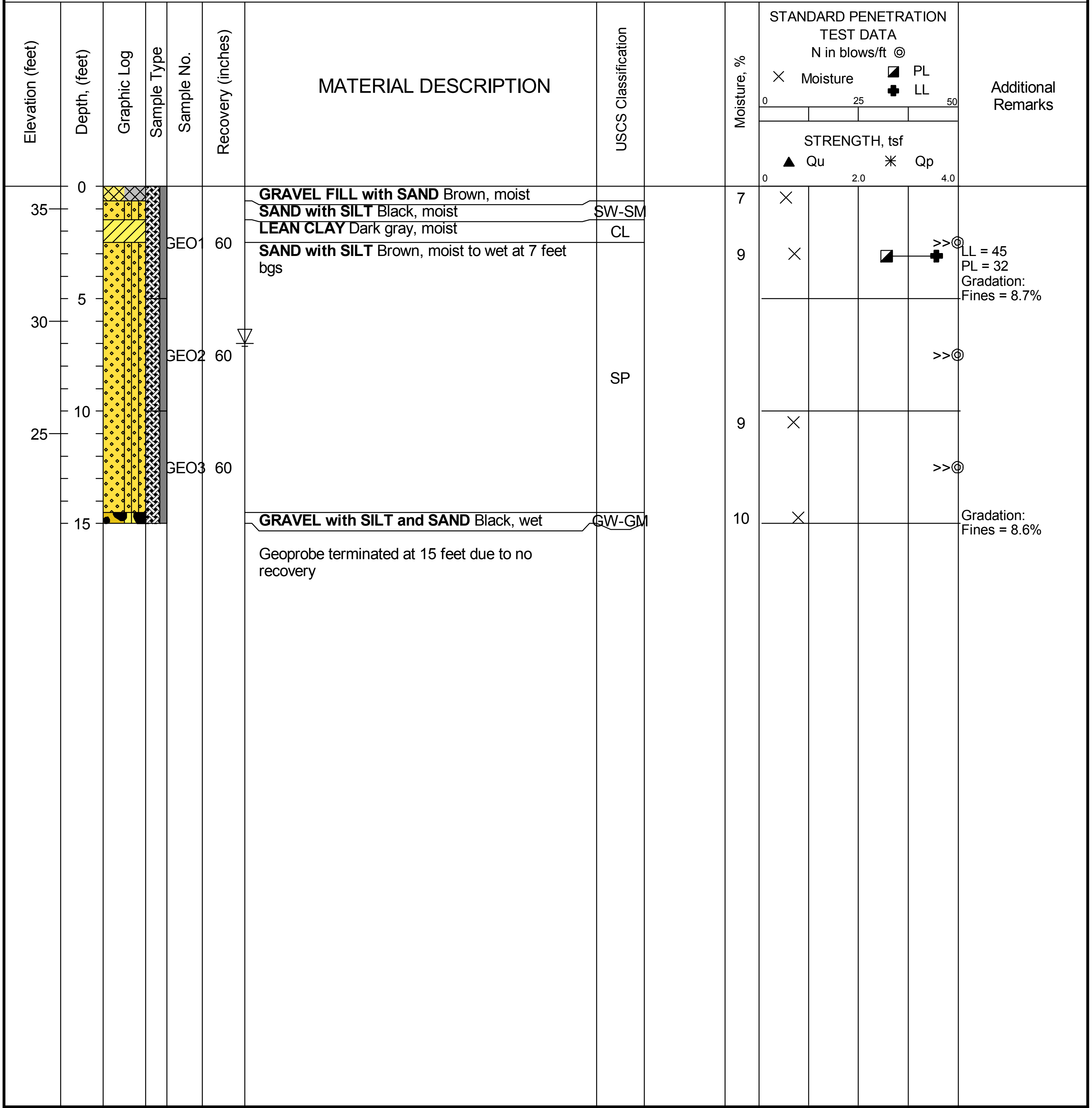
DATE STARTED: 10/24/19
DATE COMPLETED: 10/24/19
COMPLETION DEPTH 15.0 ft
BENCHMARK: N/A
ELEVATION: 36 ft
LATITUDE: 45.562891°
LONGITUDE: -122.735369°
STATION: N/A **OFFSET:** N/A
REMARKS:

DRILL COMPANY: Oregon Geotechnical Explorations
DRILLER: Terry **LOGGED BY:** HR
DRILL RIG: Track Mounted Geoprobe
DRILLING METHOD: Geoprobe
SAMPLING METHOD: Continuous Core
HAMMER TYPE:
EFFICIENCY N/A%
REVIEWED BY: SB

BORING GP1

Water		
▽	While Drilling	7 feet
▼	Upon Completion	N/A
▽	Delay	N/A

BORING LOCATION:



Sample No.	Recovery (inches)	N in blows/ft
GEO1	60	>>
GEO2	60	>>
GEO3	60	>>

MATERIAL DESCRIPTION

MATERIAL DESCRIPTION	USCS Classification
GRAVEL FILL with SAND Brown, moist	
SAND with SILT Black, moist	SW-SM
LEAN CLAY Dark gray, moist	CL
SAND with SILT Brown, moist to wet at 7 feet bgs	SP
GRAVEL with SILT and SAND Black, wet	GW-GM

Geoprobe terminated at 15 feet due to no recovery

Moisture, %	Additional Remarks
7	
9	LL = 45 PL = 32 Gradation: Fines = 8.7%
9	
10	Gradation: Fines = 8.6%

Professional Service Industries, Inc.
6032 N. Cutter Circle, Suite 480
Portland, OR 97219
Telephone: (503) 289-1778

PROJECT NO.: 07041274
PROJECT: McCall Oil and Chemical Corporation
LOCATION: 5480 NW Front Avenue
Portland, Oregon

The stratification lines represent approximate boundaries. The transition may be gradual.

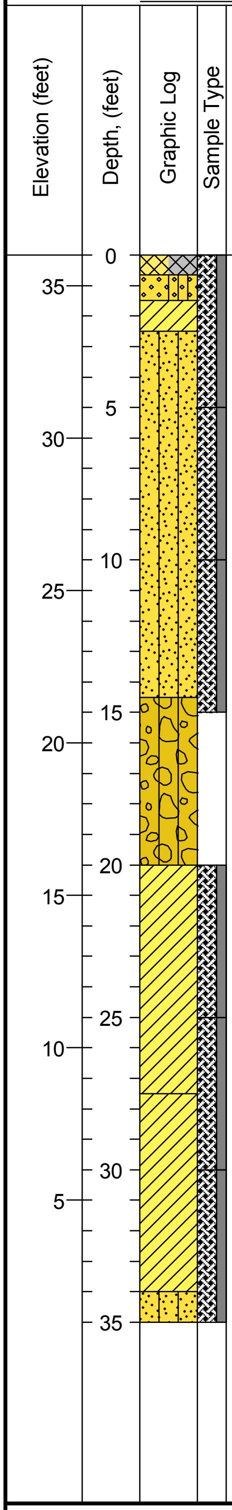
BORING GP2

DATE STARTED: 10/24/19
DATE COMPLETED: 10/24/19
COMPLETION DEPTH 35.0 ft
BENCHMARK: N/A
ELEVATION: 36 ft
LATITUDE: 45.562877°
LONGITUDE: -122.73538°
STATION: N/A **OFFSET:** N/A
REMARKS:

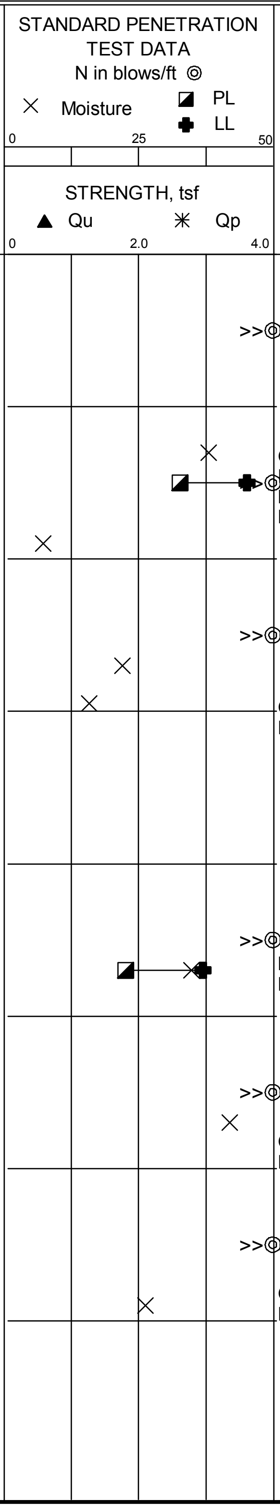
DRILL COMPANY: Oregon Geotechnical Explorations
DRILLER: Terry **LOGGED BY:** HR
DRILL RIG: Track Mounted Geoprobe
DRILLING METHOD: Geoprobe
SAMPLING METHOD: Continuous Core
HAMMER TYPE:
EFFICIENCY N/A%
REVIEWED BY: SB

Water
∇ While Drilling 7 feet
▼ Upon Completion N/A
∇ Delay N/A

BORING LOCATION:



Sample No.	Recovery (inches)	MATERIAL DESCRIPTION	USCS Classification	Moisture, %	Additional Remarks
		GRAVEL FILL with SAND Brown, moist			
		SAND with SILT Black, moist	SW-SM		
		LEAN CLAY Dark gray, moist	CL		
1 2 3	60 60 60	**SILTY SAND** Brown, moist to wet at 7 feet bgs	SM	38 7 22	Gradation: Fines = 47.4% LL = 45 PL = 33
		SILTY GRAVEL with SAND Brown, wet, no recovery from 15 to 20 feet bgs	GM	16	Gradation: Fines = 14.2%
4	60	**LEAN CLAY** Trace sand, dark gray, moist	CL	35	LL = 37 PL = 23
5 6	60 60	**SANDY LEAN CLAY** Dark gray, moist	CL	42	Gradation: Fines = 63%
		SILTY SAND Gray, wet	SM	26	Gradation: Fines = 12.6%
		Geoprobe terminated at 35 feet			



intertek psi

Professional Service Industries, Inc.
6032 N. Cutter Circle, Suite 480
Portland, OR 97219
Telephone: (503) 289-1778

PROJECT NO.: 07041274
PROJECT: McCall Oil and Chemical Corporation
LOCATION: 5480 NW Front Avenue
Portland, Oregon

The stratification lines represent approximate boundaries. The transition may be gradual.

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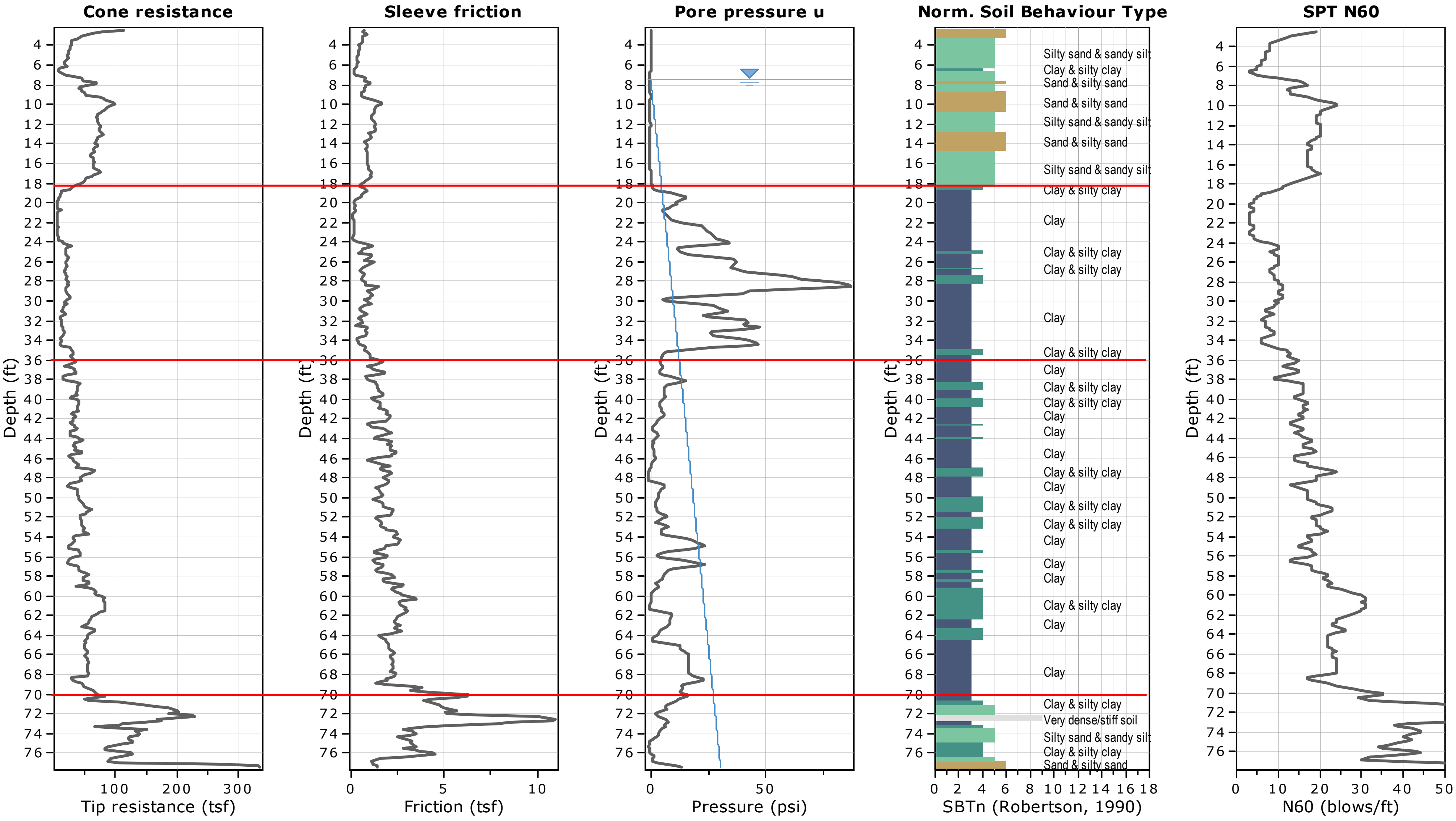
CPT: 19181 CPT-1 Text File
Total depth: 77.43 ft, Date: 11/13/2019
Surface Elevation: 0.00 ft
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator:

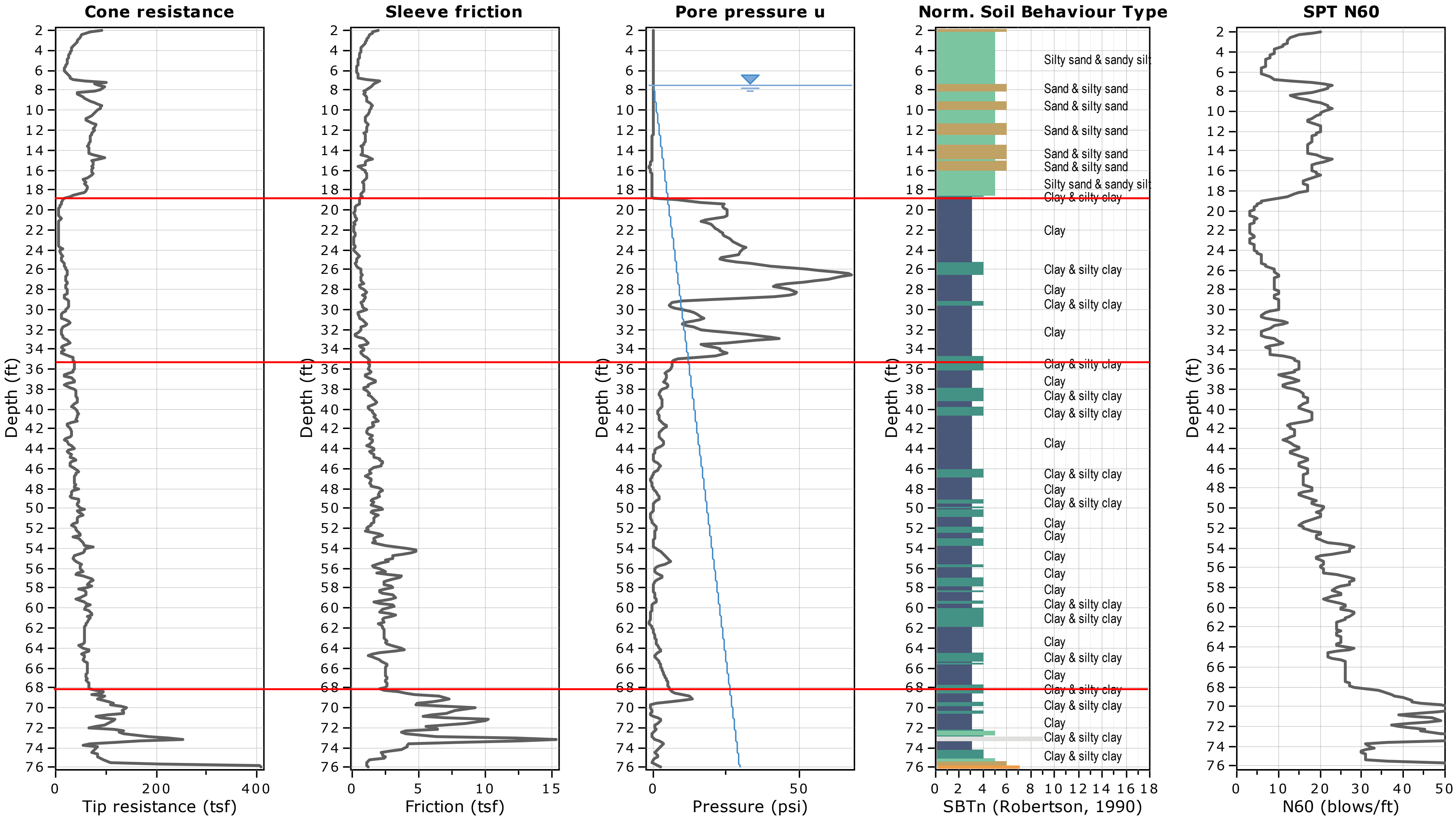
Project: **McCall Oil and Chemical Corporation PMA Project**
Location: **Front Ave, Portland, OR**



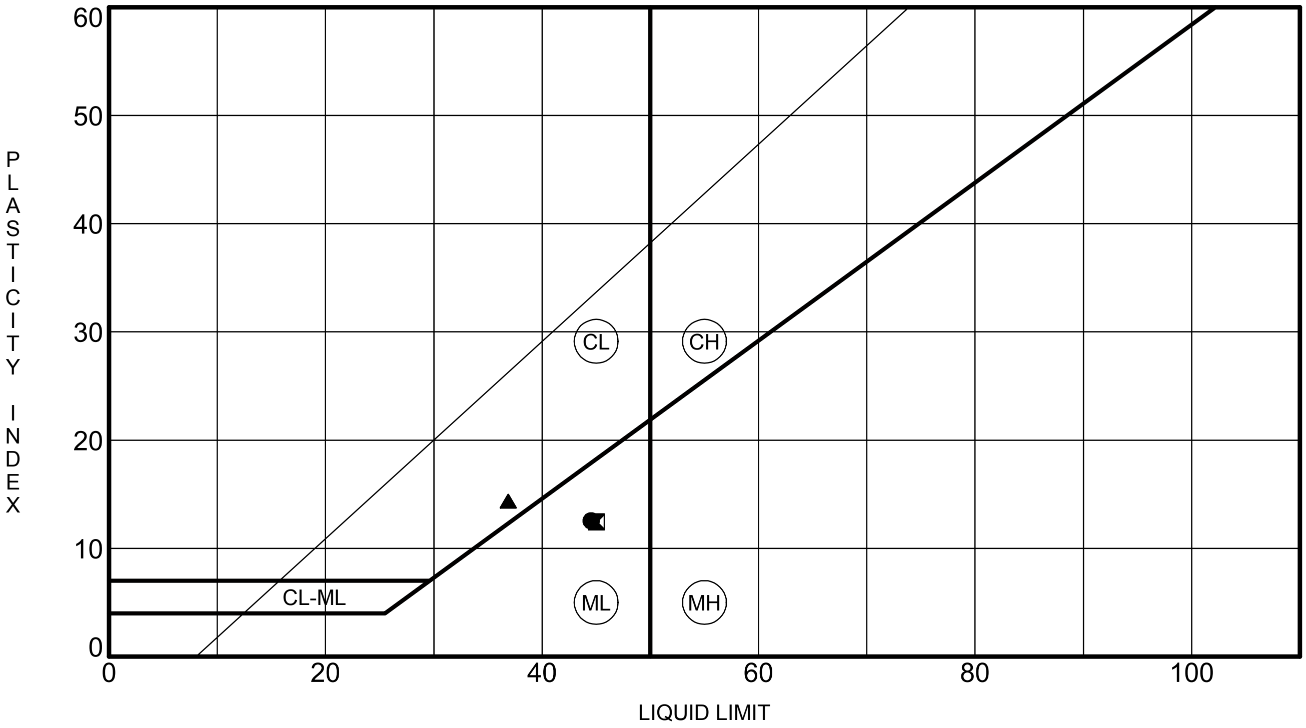
Intertek PSI
6032 North Cutter Circle Suite 480

CPT: 19181 CPT-2 Text File
Total depth: 75.95 ft, Date: 11/13/2019
Surface Elevation: 0.00 ft
Coords: X:0.00, Y:0.00
Cone Type:
Cone Operator:

Project: **McCall Oil and Chemical Corporation PMA Project**
Location: **Front Ave, Portland, OR**



APPENDIX B – LABORATORY TEST RESULTS



	Boring	Depth (ft)	LL	PL	PI	Fines	Classification (*Visual)
●	**GP1**	**1.5**	**45**	**32**	**13**		
⧗	**GP2**	**7.5**	**45**	**33**	**12**		
▲	**GP2**	**23.5**	**37**	**23**	**14**		

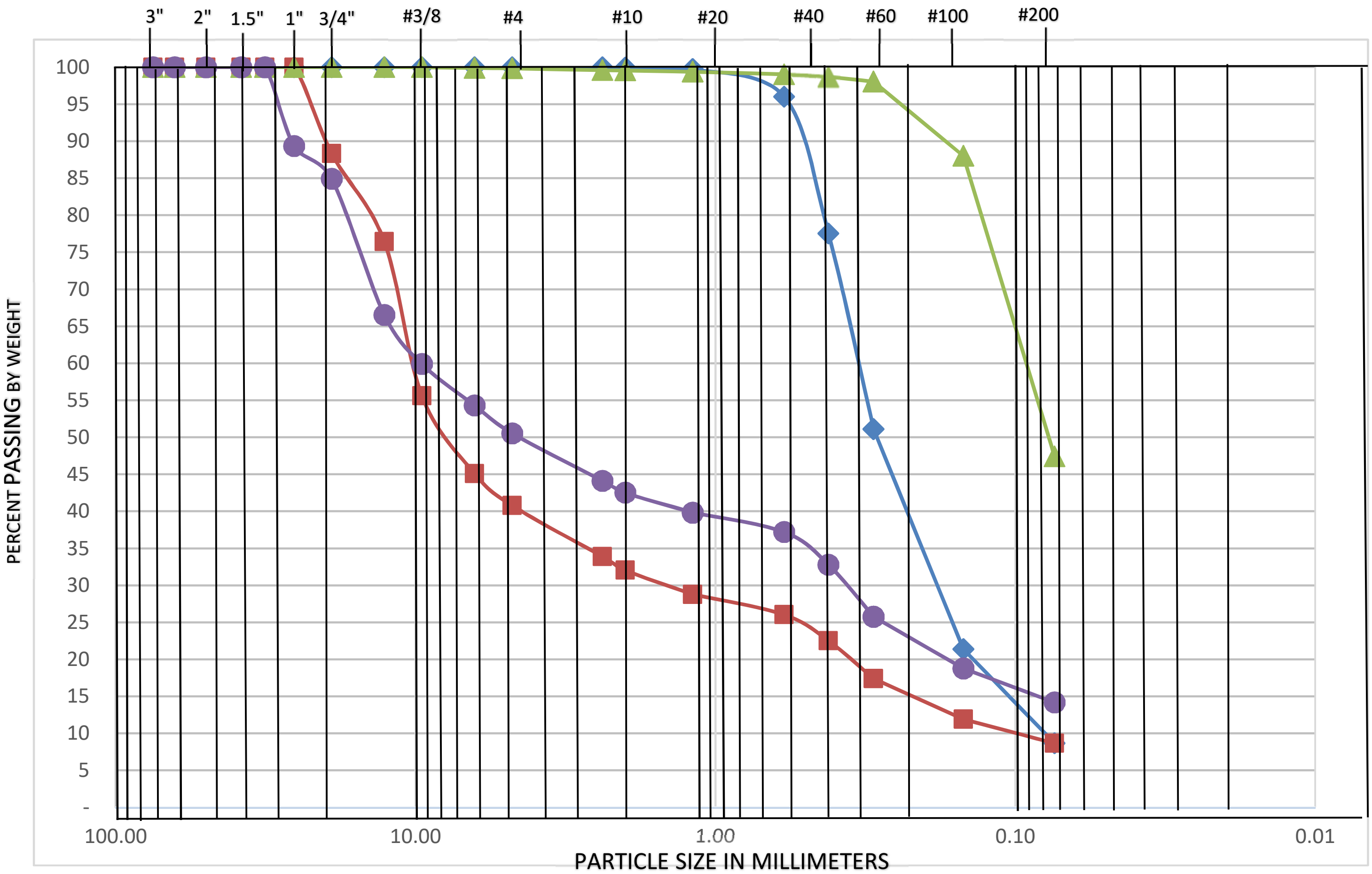
Professional Service Industries, Inc.
6032 N. Cutter Circle, Suite 480
Portland, OR 97219
Telephone: (503) 289-1778
Fax: (503) 289-1918

ATTERBERG LIMIT RESULTS

PSI Job No.: 07041274
Project: McCall Oil and Chemical
Location:

PARTICLE SIZE ANALYSIS - ASTM (D-422)

Project Name McCall Oil and Chemical Corporation
Project Location Portland, OR
Project Number 07041274
Tested By EL
Date of Sampling 10/24/2019
Date of Testing 11/8/2019
Reviewed By SB



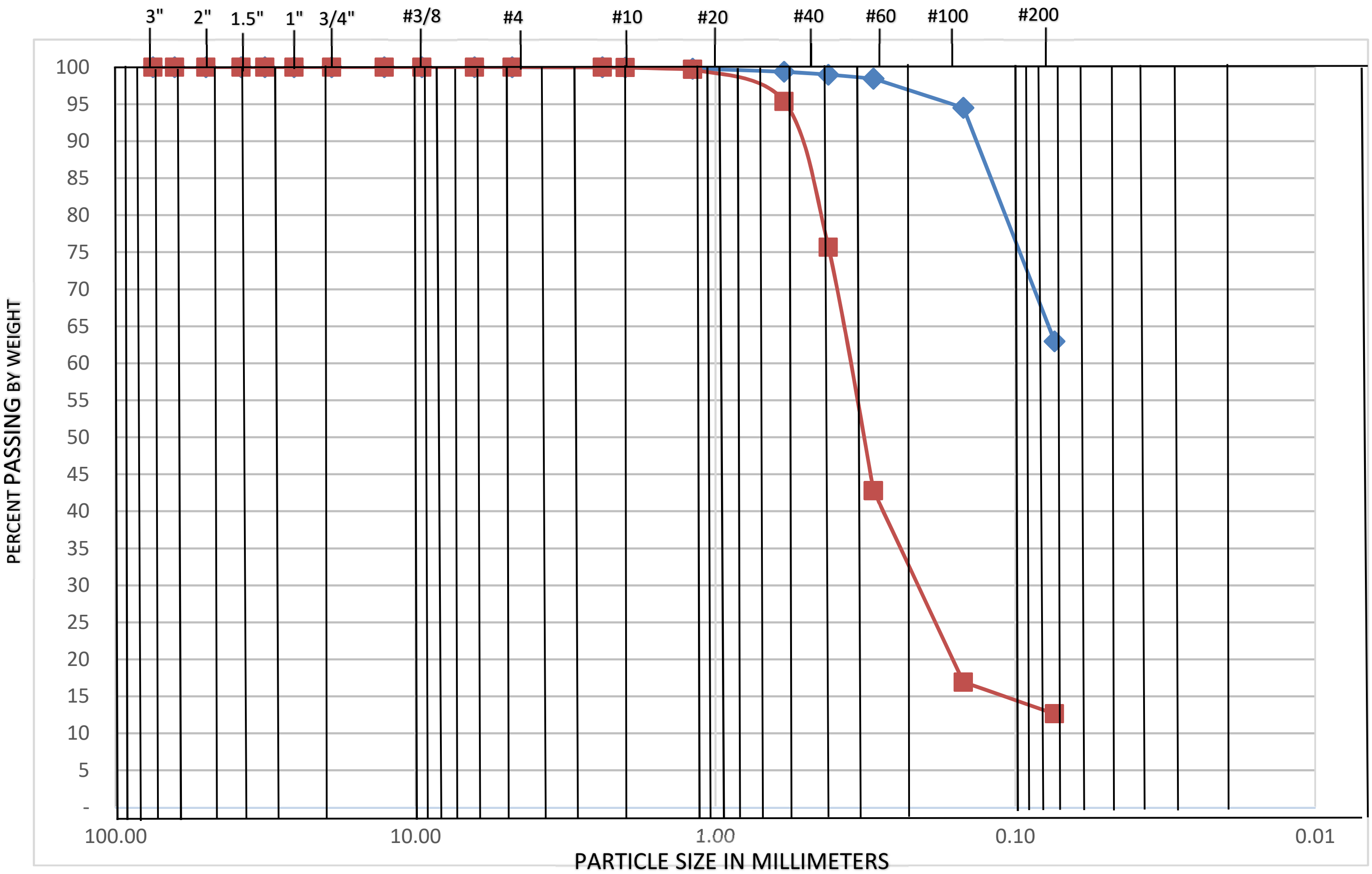
Coarse	Fine	Coarse	Medium	Fine	Silt/Clay
Gravel		Sand			Fine Grained

Boring #	Depth	% Gravel	% Sand	% Fines	PL	LL	PI	Moisture (%)
GP1	3		91.3%	8.7%				8.9%
GP1	14.75	59.2%	32.1%	8.6%				9.9%
GP2	6.5	0.2%	52.4%	47.4%				38.1%
GP2	14.75	49.5%	36.3%	14.2%				15.8%

Boring #	Depth	USCS Symbol	USCS Name	Plot Lines
GP1	3	SW-SM	Well Graded SAND with Silt	◆
GP1	14.75	GW-GM	Well Graded GRAVEL with Silt and Sand	■
GP2	6.5	SM	Silty SAND	▲
GP2	14.75	GM	Silty GRAVEL with Sand	●

PARTICLE SIZE ANALYSIS - ASTM (D-422)

Project Name McCall Oil and Chemical Corporation
Project Location Portland, OR
Project Number 07041274
Tested By EL
Date of Sampling 10/24/2019
Date of Testing 11/8/2019
Reviewed By SB



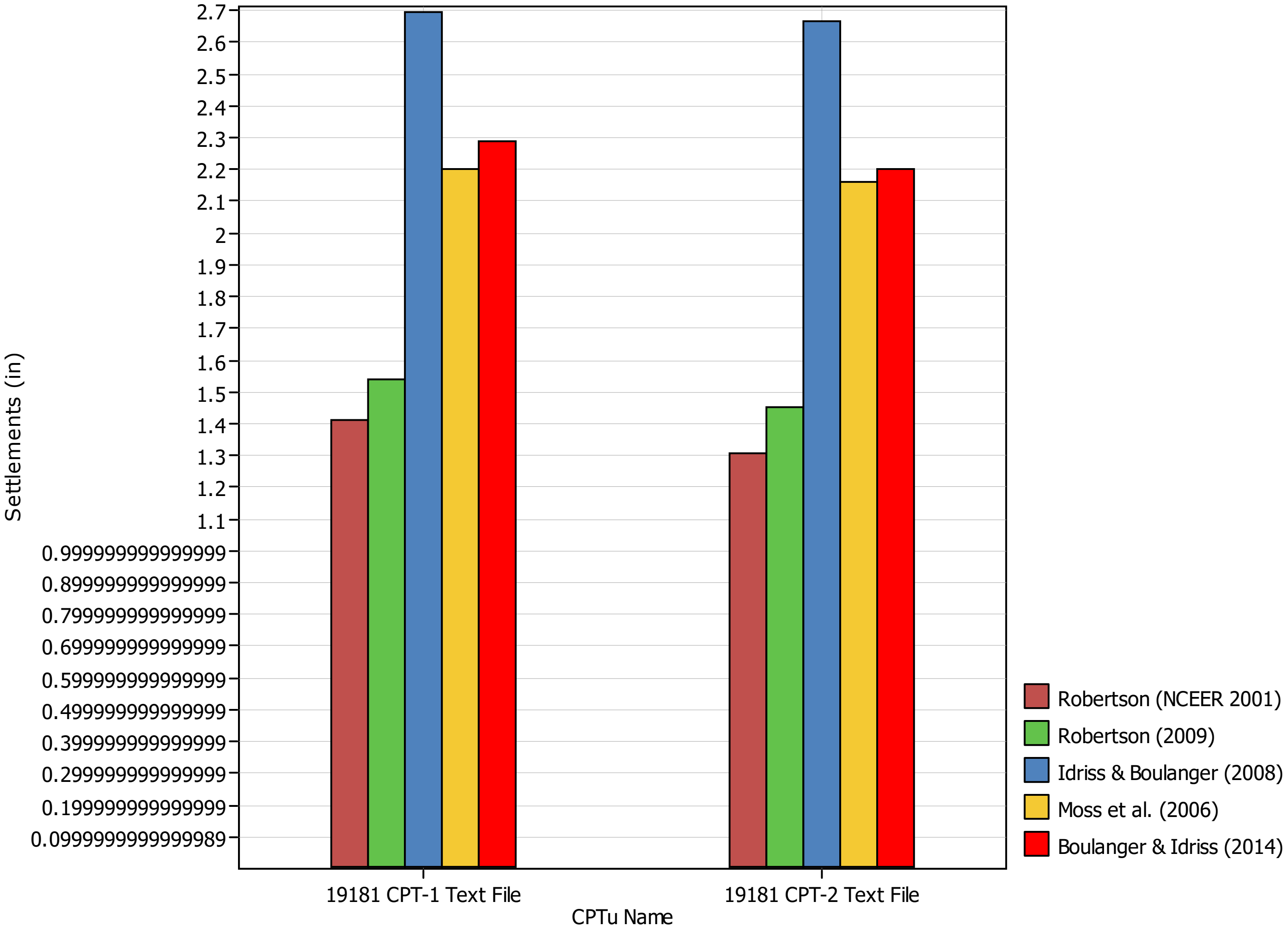
Coarse	Fine	Coarse	Medium	Fine	Silt/Clay
Gravel		Sand			Fine Grained

Boring #	Depth	% Gravel	% Sand	% Fines	PL	LL	PI	Moisture (%)
GP2	29		37.0%	63.0%				42.0%
GP2	34.50		87.4%	12.6%				26.2%

Boring #	Depth	USCS Symbol	USCS Name	Plot Lines
GP2	29	CL	Sandy Lean CLAY	◆
GP2	34.50	SM	Silty SAND	■

APPENDIX C – LIQUEFACTION RESULTS

Overall Parametric Assessment Method



:: CPT main liquefaction parameters details ::

CPT Name	Earthquake Mag.	Earthquake Accel.	GWT in situ (ft)	GWT earthq. (ft)
19181 CPT-1 Text Fil	7.44	0.46	7.50	7.50
19181 CPT-2 Text Fil	7.44	0.46	7.50	7.50

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6032 North Cutter Circle Suite 480

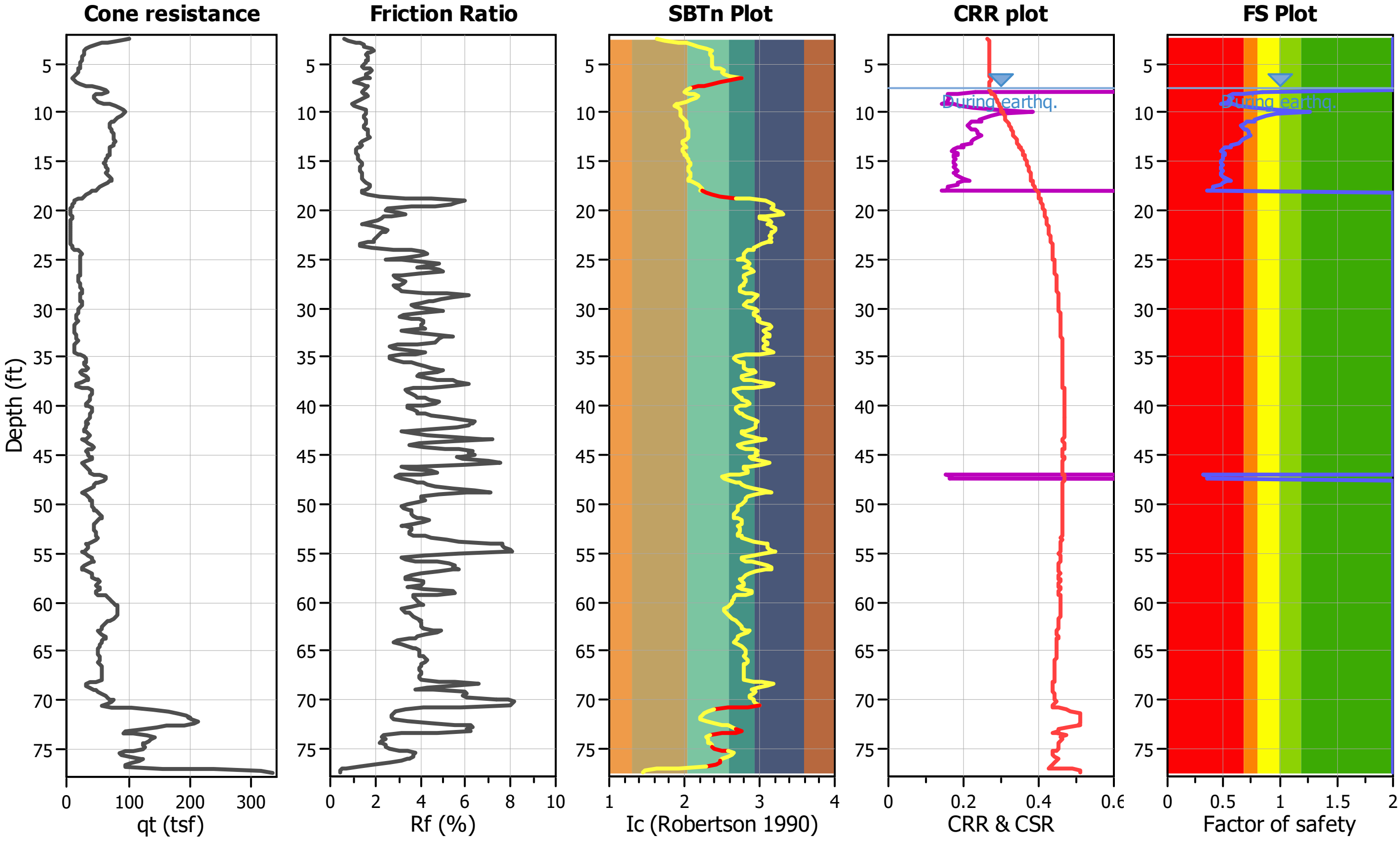
LIQUEFACTION ANALYSIS REPORT

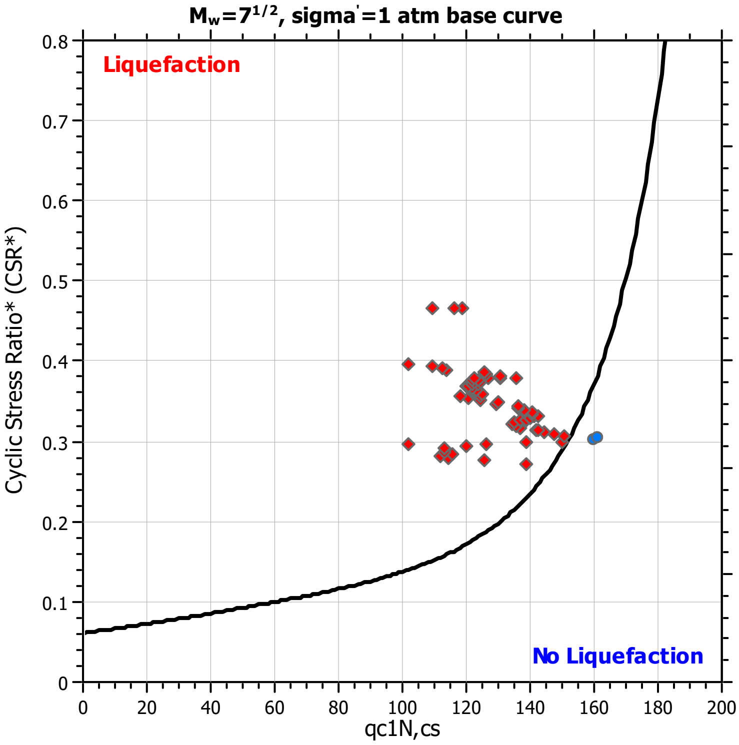
Project title : McCall Oil and Chemical Corporation PMA **Location : Front Ave, Portland, OR**

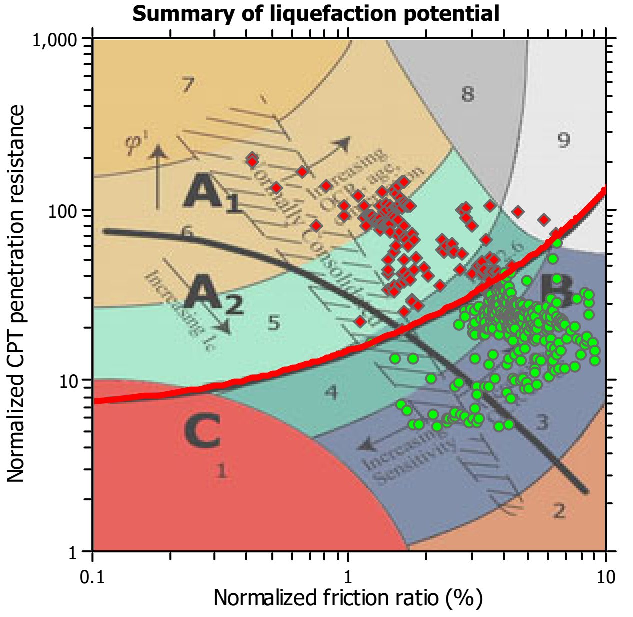
CPT file : 19181 CPT-1 Text File

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	7.50 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	7.50 ft	Fill height:	N/A		
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth applied:	Yes
Earthquake magnitude M_w:	7.44	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	Limit depth:	60.00 ft
Peak ground acceleration:	0.46	Unit weight calculation:	Based on SBT	K_σ applied:	Yes	MSF method:	Method

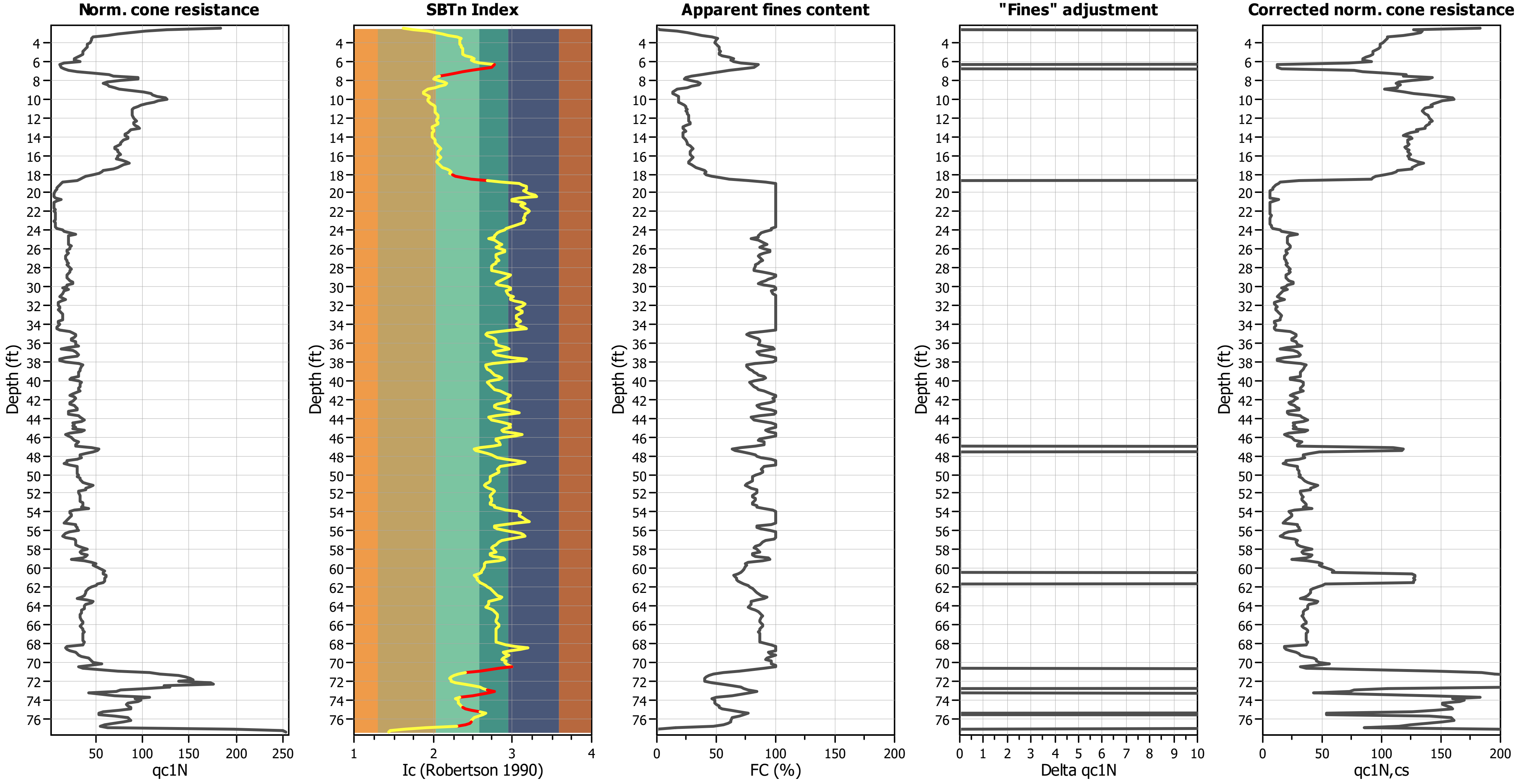






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

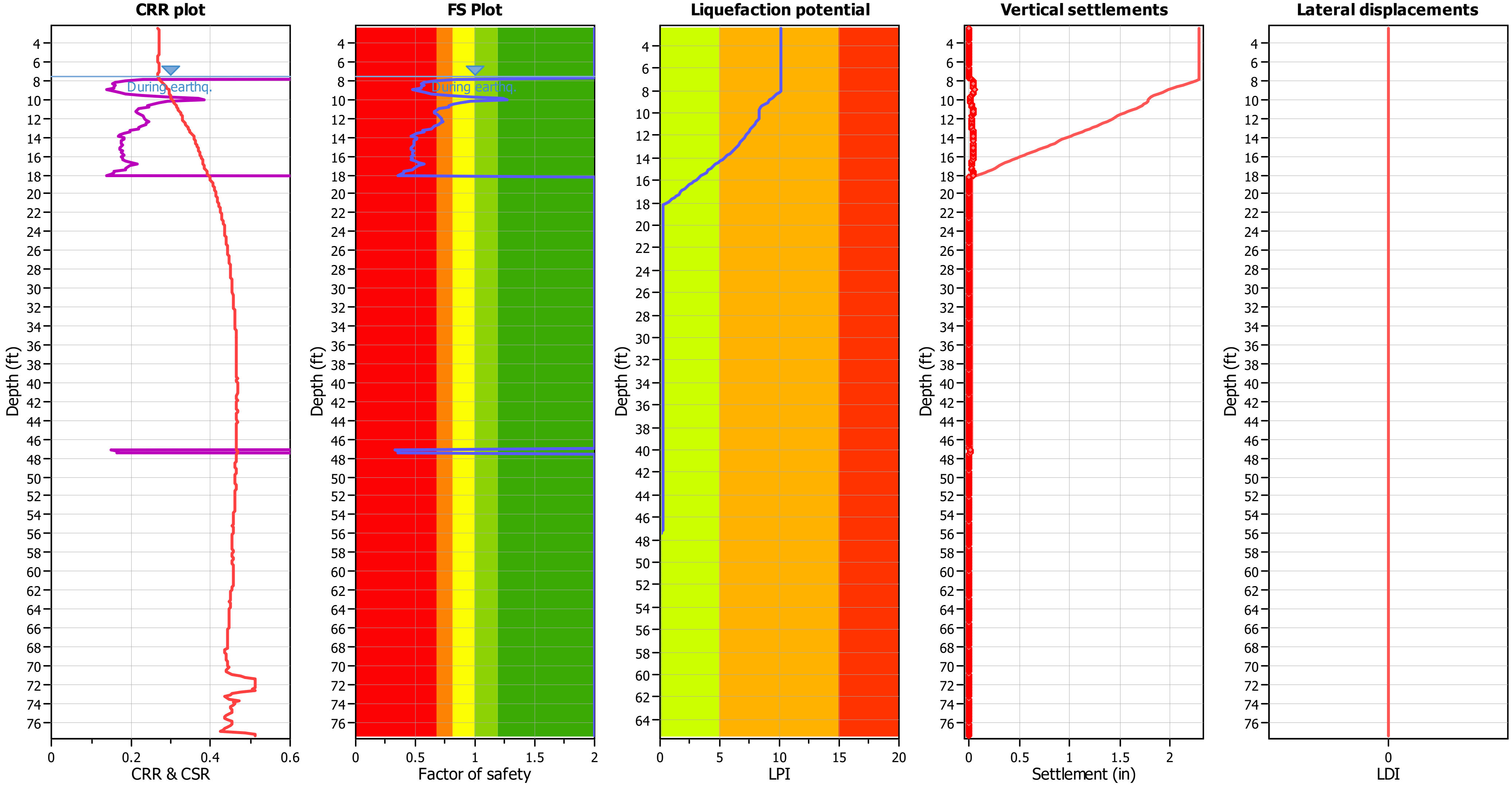
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	7.50 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	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:	7.44	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.46	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	7.50 ft	Fill height:	N/A	Limit depth:	60.00 ft

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	7.50 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	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:	7.44	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.46	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	7.50 ft	Fill height:	N/A	Limit depth:	60.00 ft

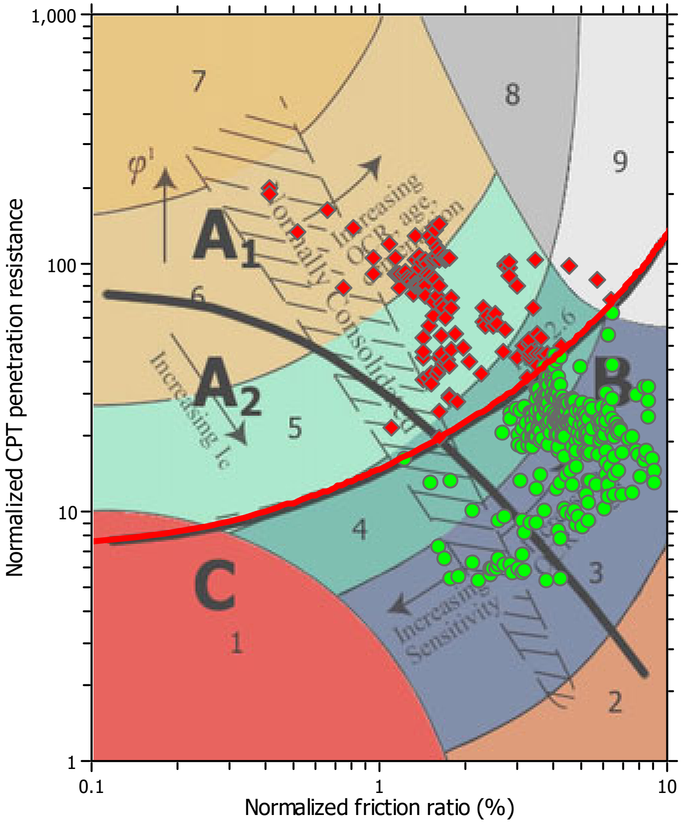
F.S. color scheme

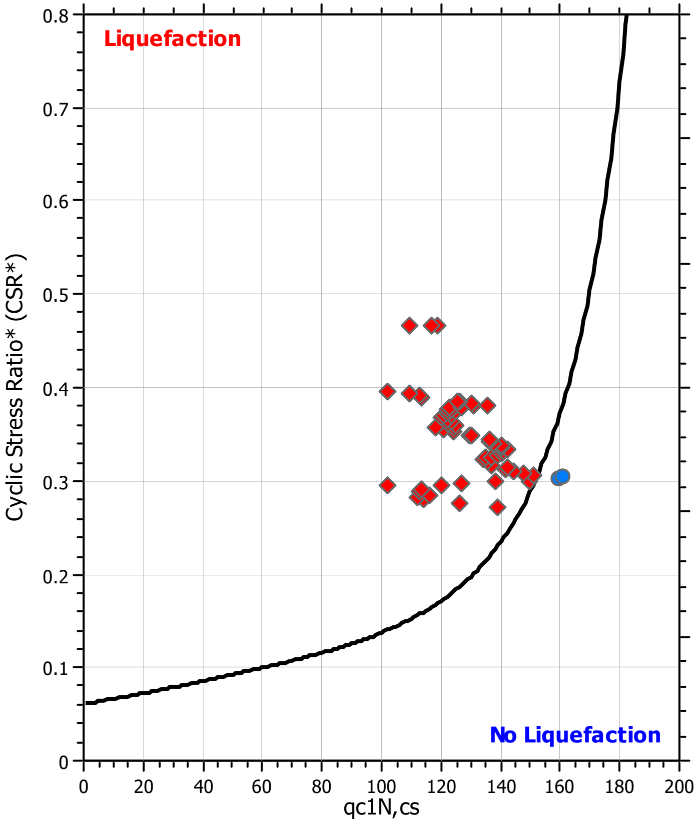
- 🟥 Almost certain it will liquefy
- 🟧 Very likely to liquefy
- 🟨 Liquefaction and no liq. are equally likely
- 🟩 Unlike to liquefy
- 🟩 Almost certain it will not liquefy

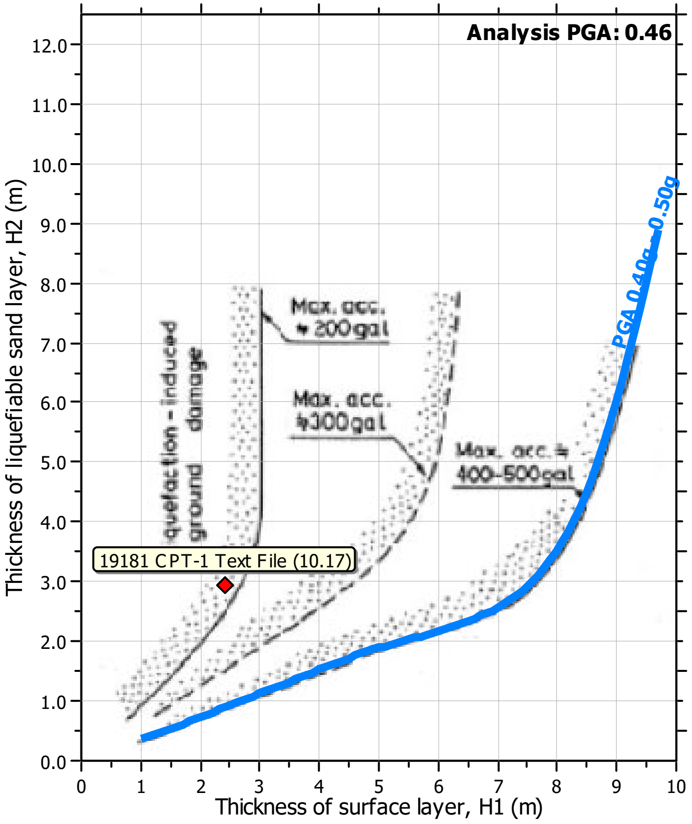
LPI color scheme

- 🟥 Very high risk
- 🟧 High risk
- 🟨 Low risk

Liquefaction analysis summary plots



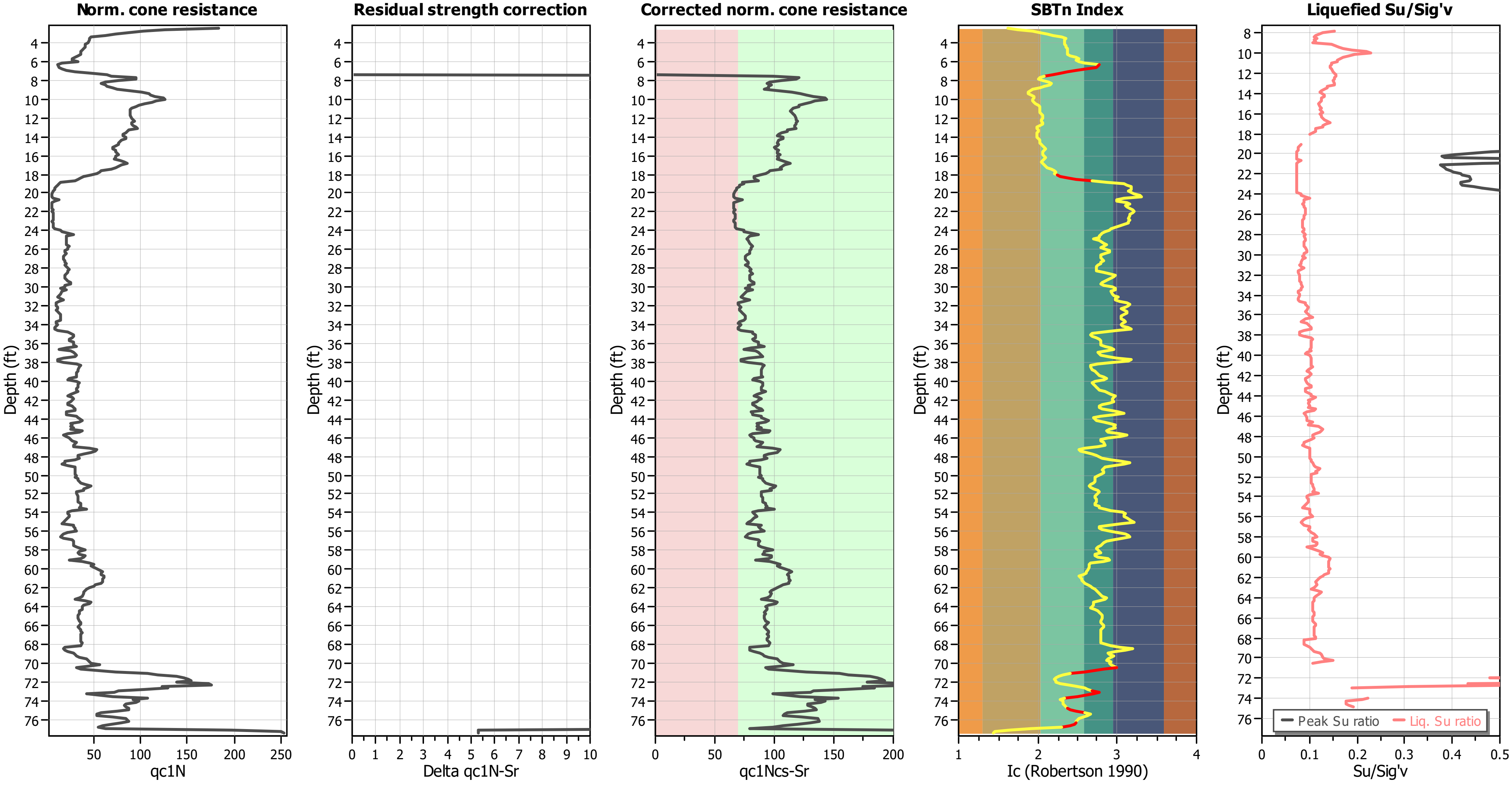




Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	7.50 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	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:	7.44	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.46	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	7.50 ft	Fill height:	N/A	Limit depth:	60.00 ft

Check for strength loss plots (Idriss & Boulanger (2008))



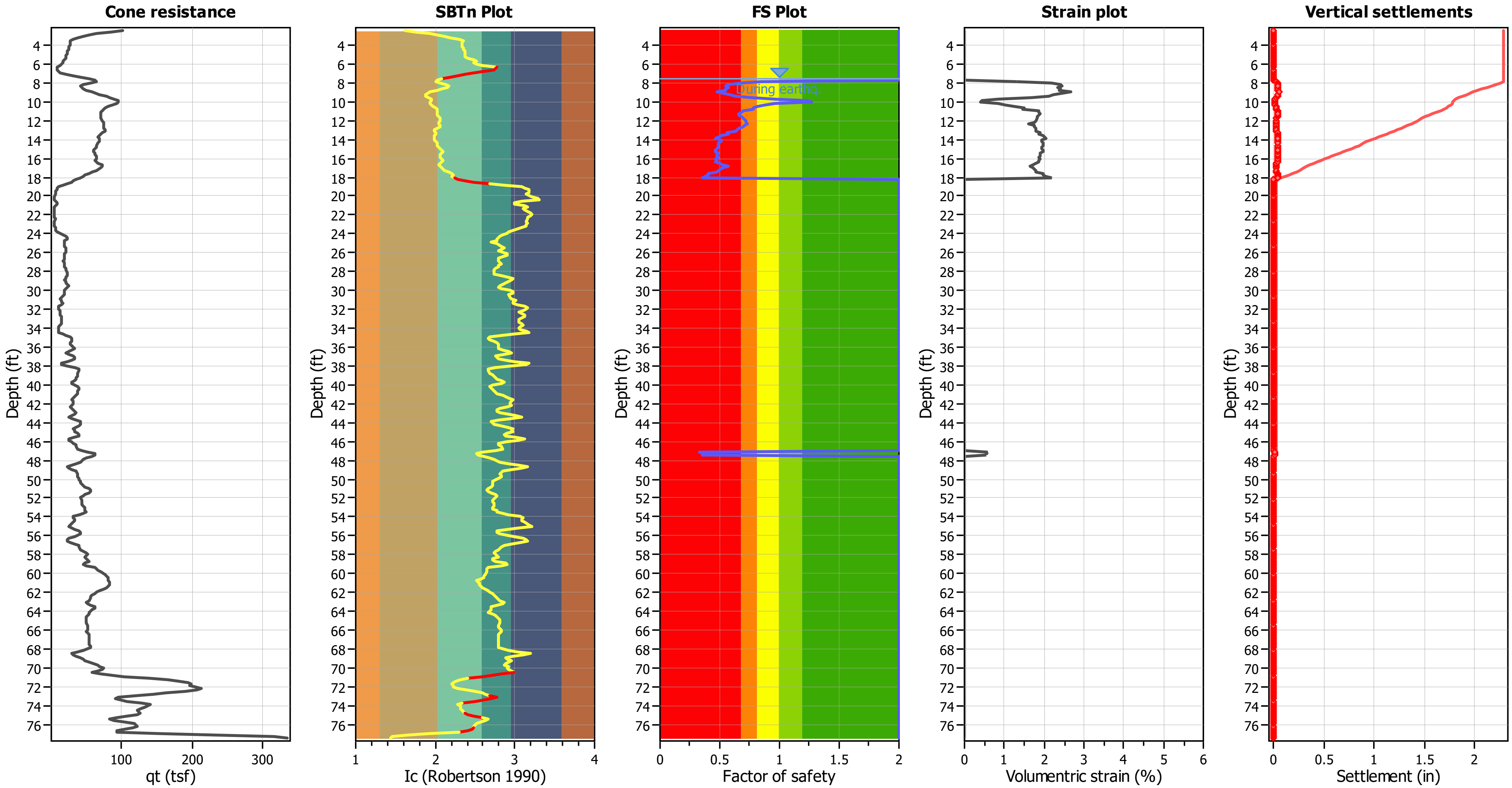
Input parameters and analysis data

Analysis method: B&I (2014)
Fines correction method: B&I (2014)
Points to test: Based on Ic value
Earthquake magnitude M_w: 7.44
Peak ground acceleration: 0.46
Depth to water table (insitu): 7.50 ft

Depth to GWT (erthq.): 7.50 ft
Average results interval: 3
Ic cut-off value: 2.60
Unit weight calculation: Based on SBT
Use fill: No
Fill height: N/A

Fill weight: N/A
Transition detect. applied: Yes
K_σ applied: Yes
Clay like behavior applied: Sands only
Limit depth applied: Yes
Limit depth: 60.00 ft

Estimation of post-earthquake settlements



Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

Intertek PSI
6032 North Cutter Circle Suite 480

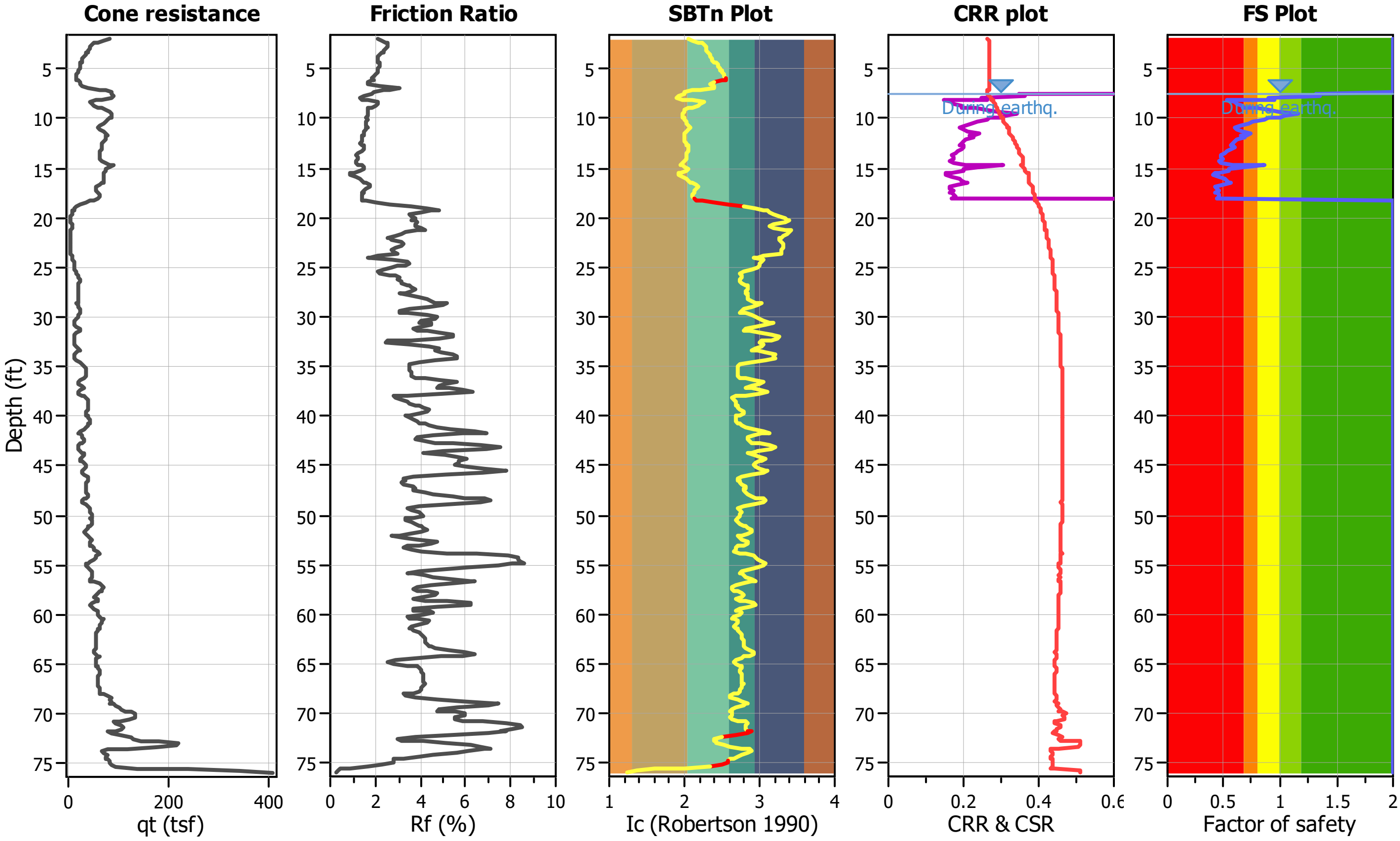
LIQUEFACTION ANALYSIS REPORT

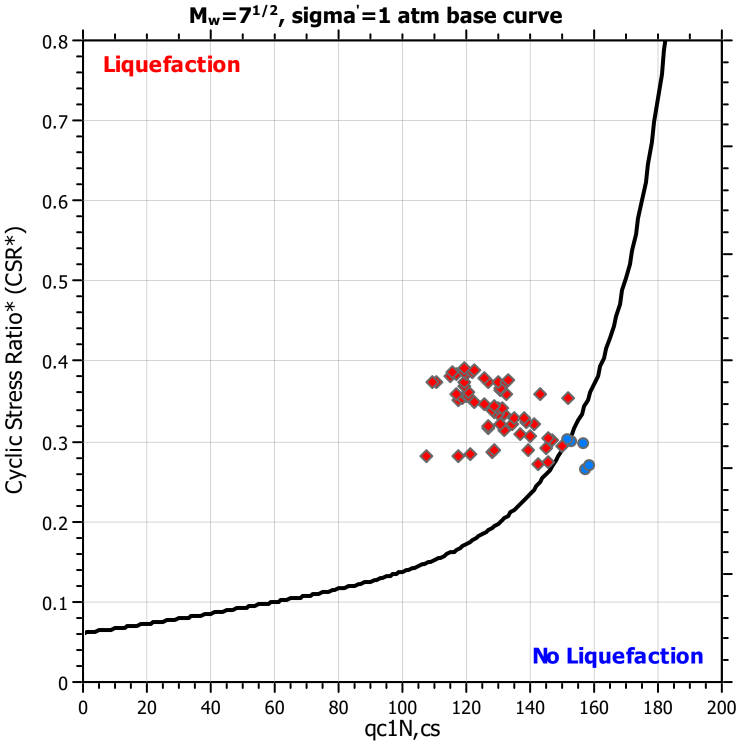
Project title : McCall Oil and Chemical Corporation PMA **Location : Front Ave, Portland, OR**

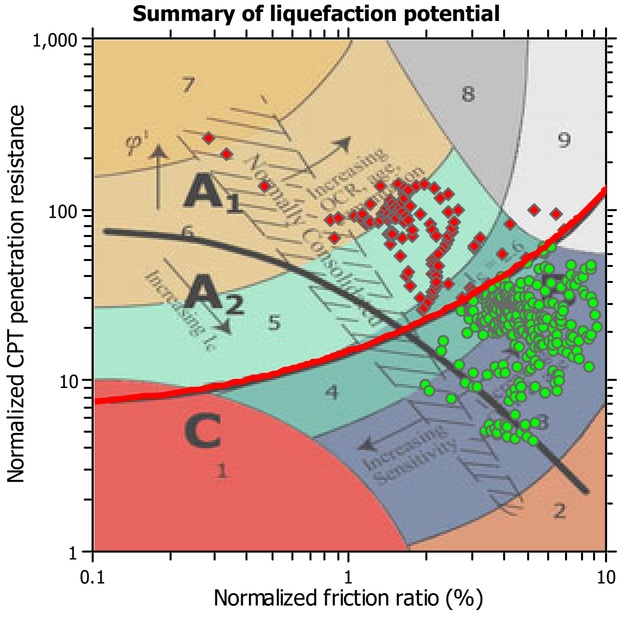
CPT file : 19181 CPT-2 Text File

Input parameters and analysis data

Analysis method:	B&I (2014)	G.W.T. (in-situ):	7.50 ft	Use fill:	No	Clay like behavior applied:	Sands only
Fines correction method:	B&I (2014)	G.W.T. (earthq.):	7.50 ft	Fill height:	N/A	Limit depth applied:	Yes
Points to test:	Based on Ic value	Average results interval:	3	Fill weight:	N/A	Limit depth:	60.00 ft
Earthquake magnitude M_w:	7.44	Ic cut-off value:	2.60	Trans. detect. applied:	Yes	MSF method:	Method
Peak ground acceleration:	0.46	Unit weight calculation:	Based on SBT	K_σ applied:	Yes		

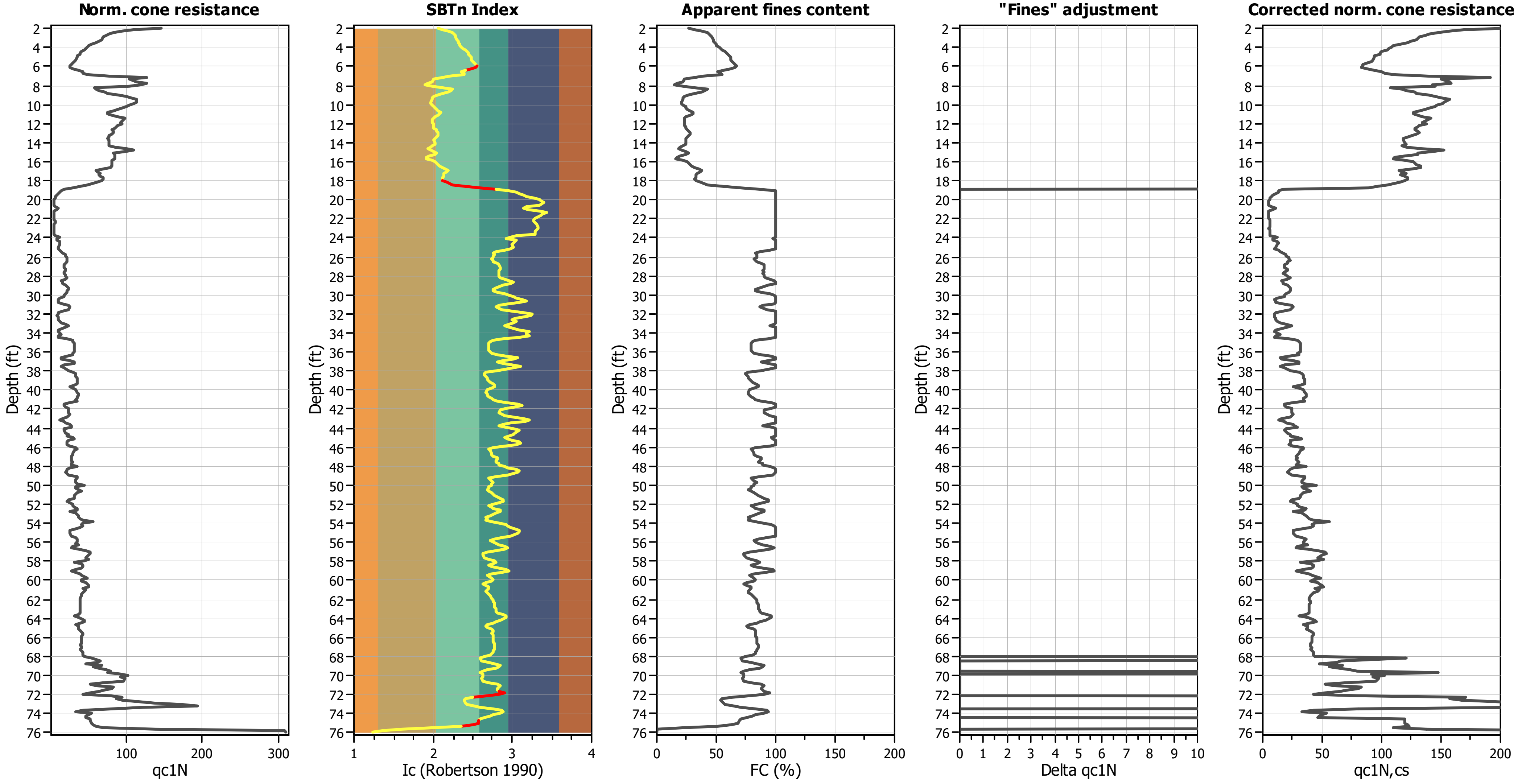






Zone A_1: Cyclic liquefaction likely depending on size and duration of cyclic loading
Zone A_2: Cyclic liquefaction and strength loss likely depending on loading and ground geometry
Zone B: Liquefaction and post-earthquake strength loss unlikely, check cyclic softening
Zone C: Cyclic liquefaction and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

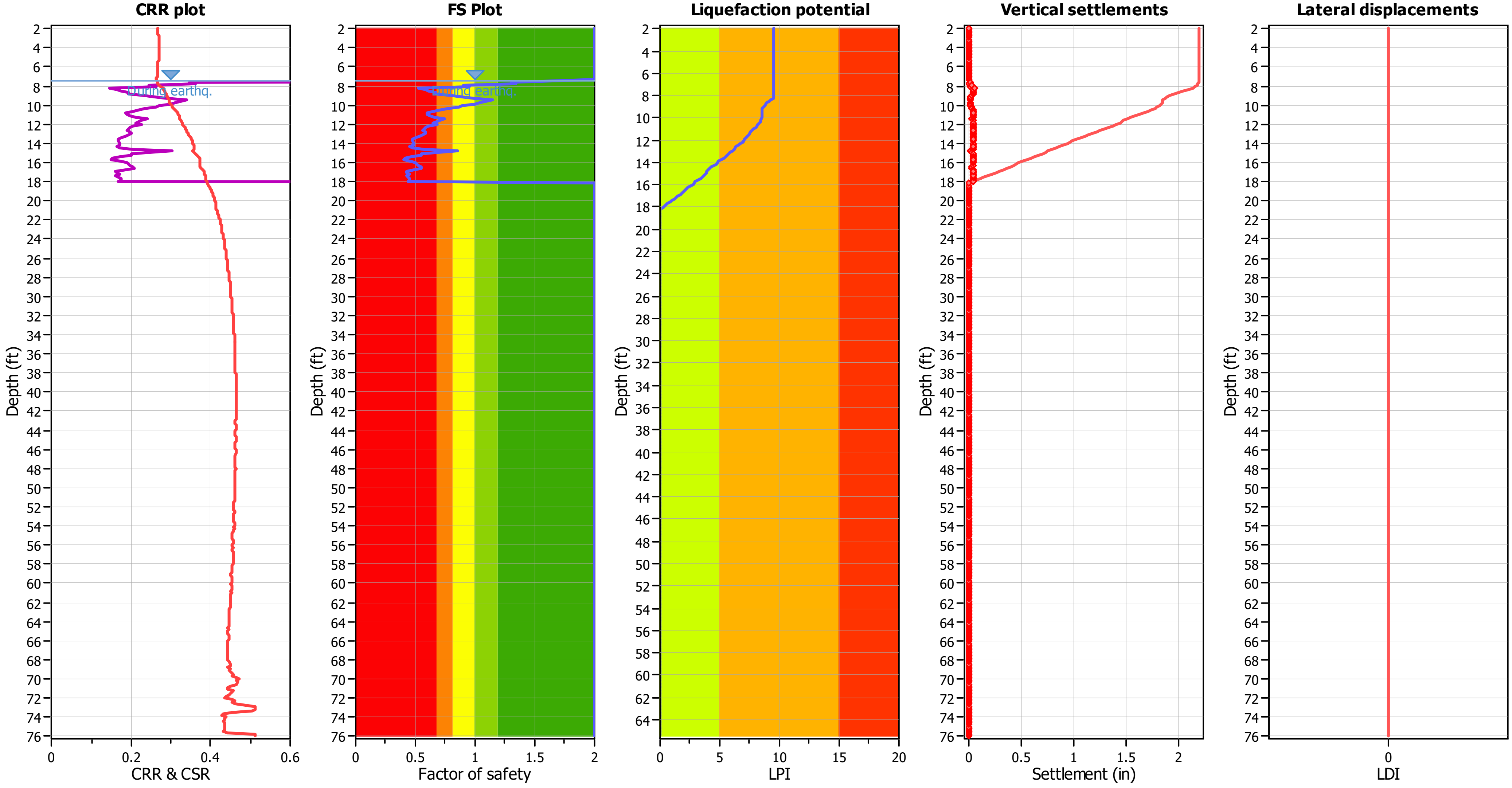
Liquefaction analysis overall plots (intermediate results)



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	7.50 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	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:	7.44	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.46	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	7.50 ft	Fill height:	N/A	Limit depth:	60.00 ft

Liquefaction analysis overall plots



Input parameters and analysis data

Analysis method: B&I (2014)
Fines correction method: B&I (2014)
Points to test: Based on Ic value
Earthquake magnitude M_w: 7.44
Peak ground acceleration: 0.46
Depth to water table (insitu): 7.50 ft

Depth to GWT (erthq.): 7.50 ft
Average results interval: 3
Ic cut-off value: 2.60
Unit weight calculation: Based on SBT
Use fill: No
Fill height: N/A

Fill weight: N/A
Transition detect. applied: Yes
K_σ applied: Yes
Clay like behavior applied: Sands only
Limit depth applied: Yes
Limit depth: 60.00 ft

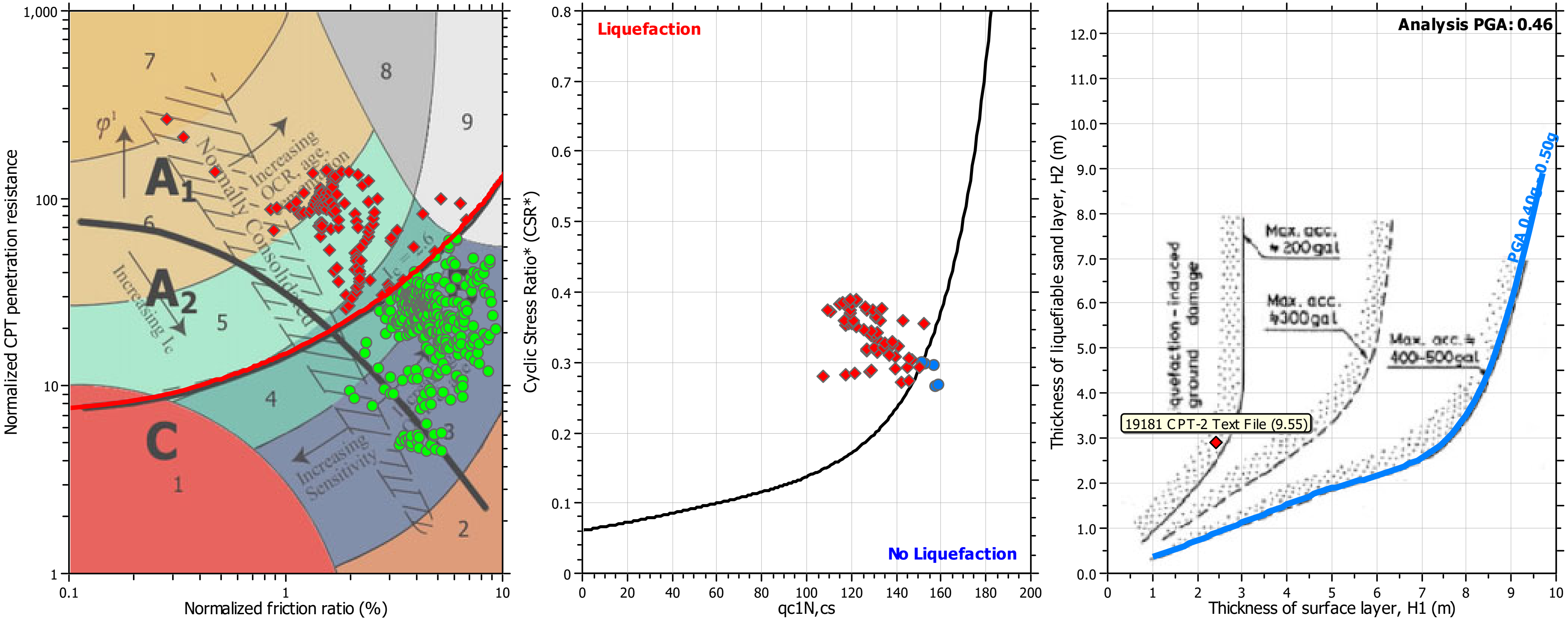
F.S. color scheme

🟥 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

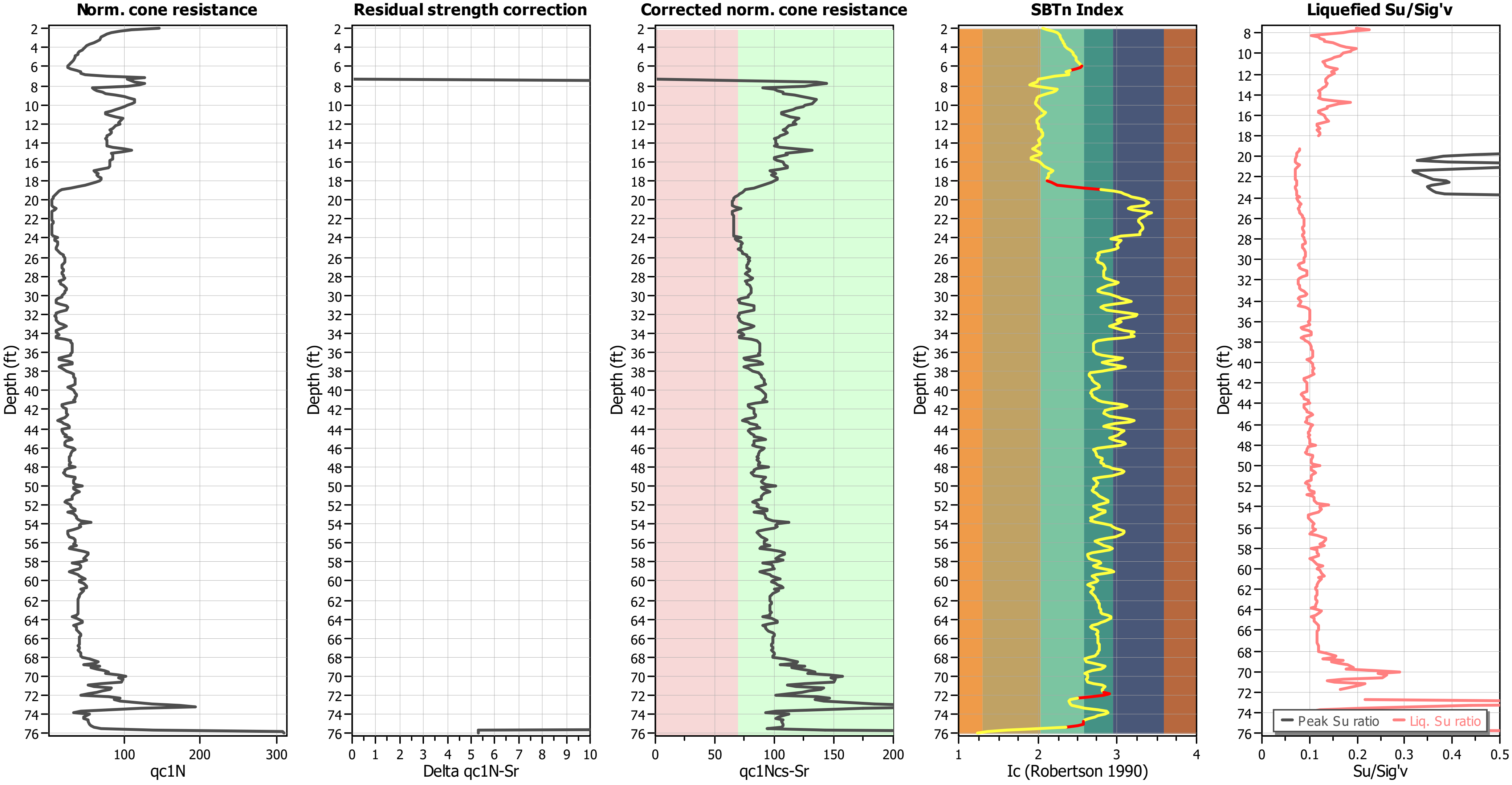
Liquefaction analysis summary plots



Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	7.50 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	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:	7.44	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.46	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	7.50 ft	Fill height:	N/A	Limit depth:	60.00 ft

Check for strength loss plots (Idriss & Boulanger (2008))


Norm. cone resistance

Residual strength correction

Corrected norm. cone resistance

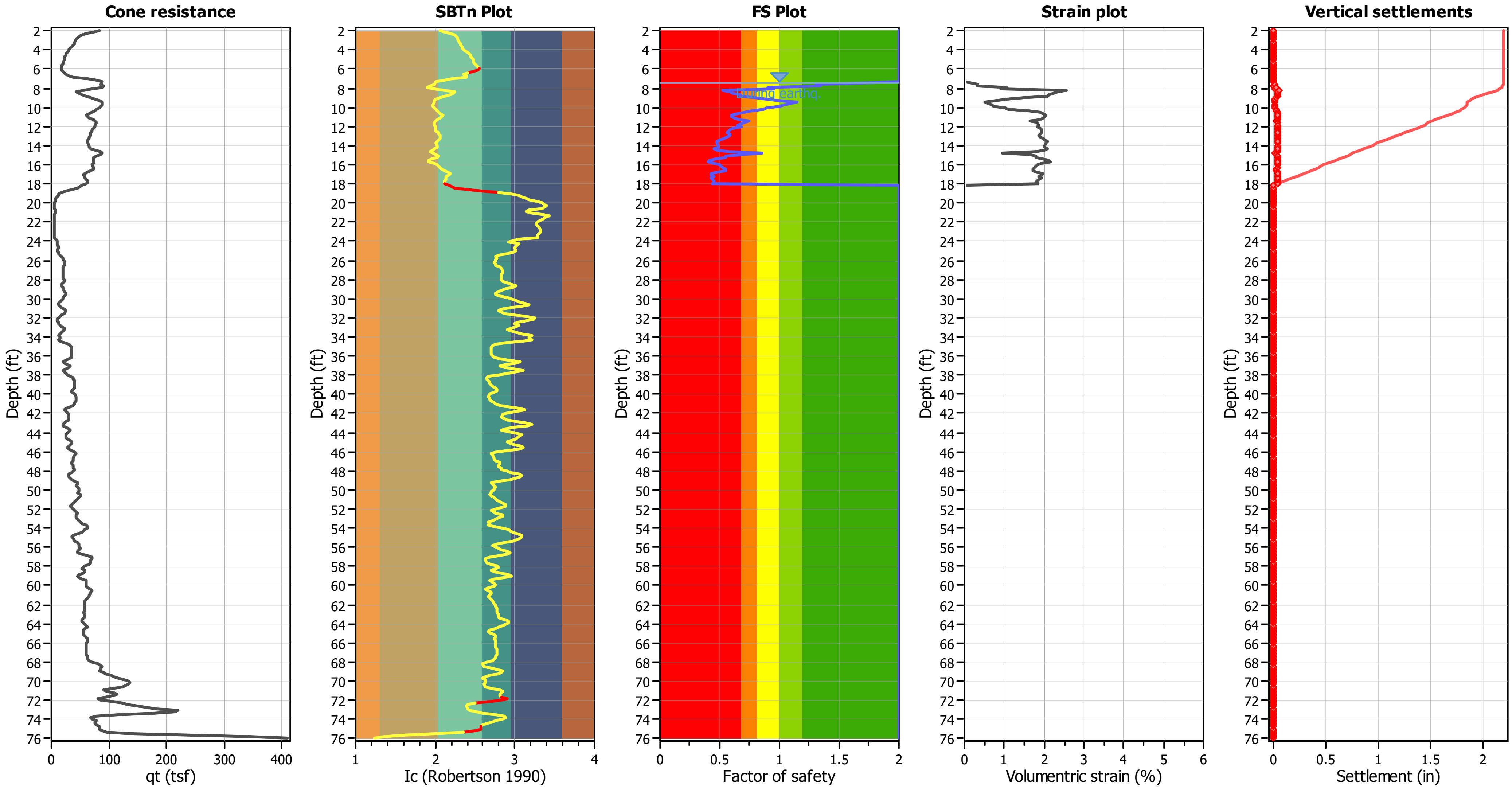
SBTn Index

Liquefied Su/Sig'v

Input parameters and analysis data

Analysis method:	B&I (2014)	Depth to GWT (erthq.):	7.50 ft	Fill weight:	N/A
Fines correction method:	B&I (2014)	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:	7.44	Unit weight calculation:	Based on SBT	Clay like behavior applied:	Sands only
Peak ground acceleration:	0.46	Use fill:	No	Limit depth applied:	Yes
Depth to water table (insitu):	7.50 ft	Fill height:	N/A	Limit depth:	60.00 ft

Estimation of post-earthquake settlements

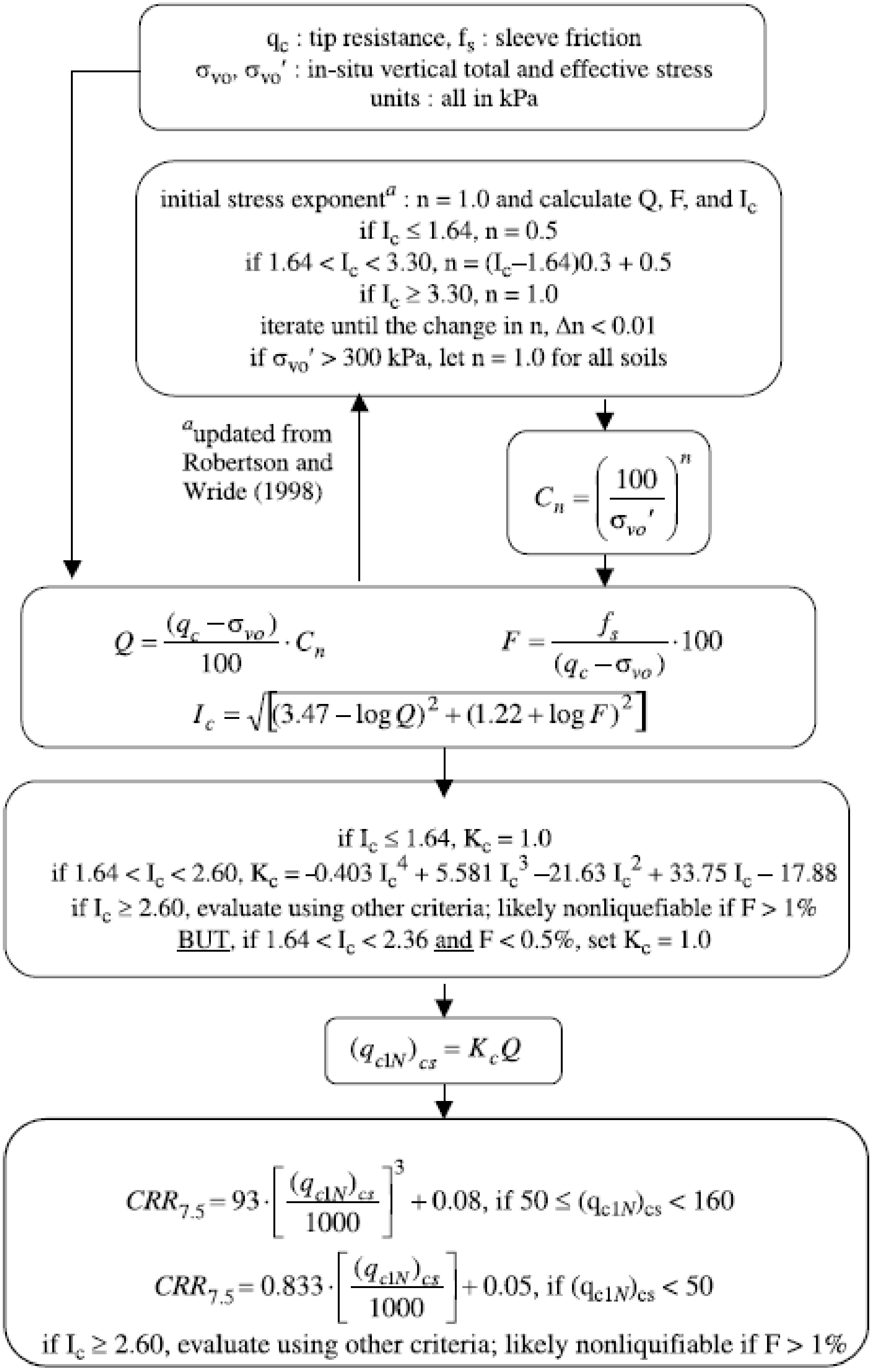


Abbreviations

q_t:	Total cone resistance (cone resistance q_c corrected for pore water effects)
I_c:	Soil Behaviour Type Index
FS:	Calculated Factor of Safety against liquefaction
Volumentric strain:	Post-liquefaction volumentric strain

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]:



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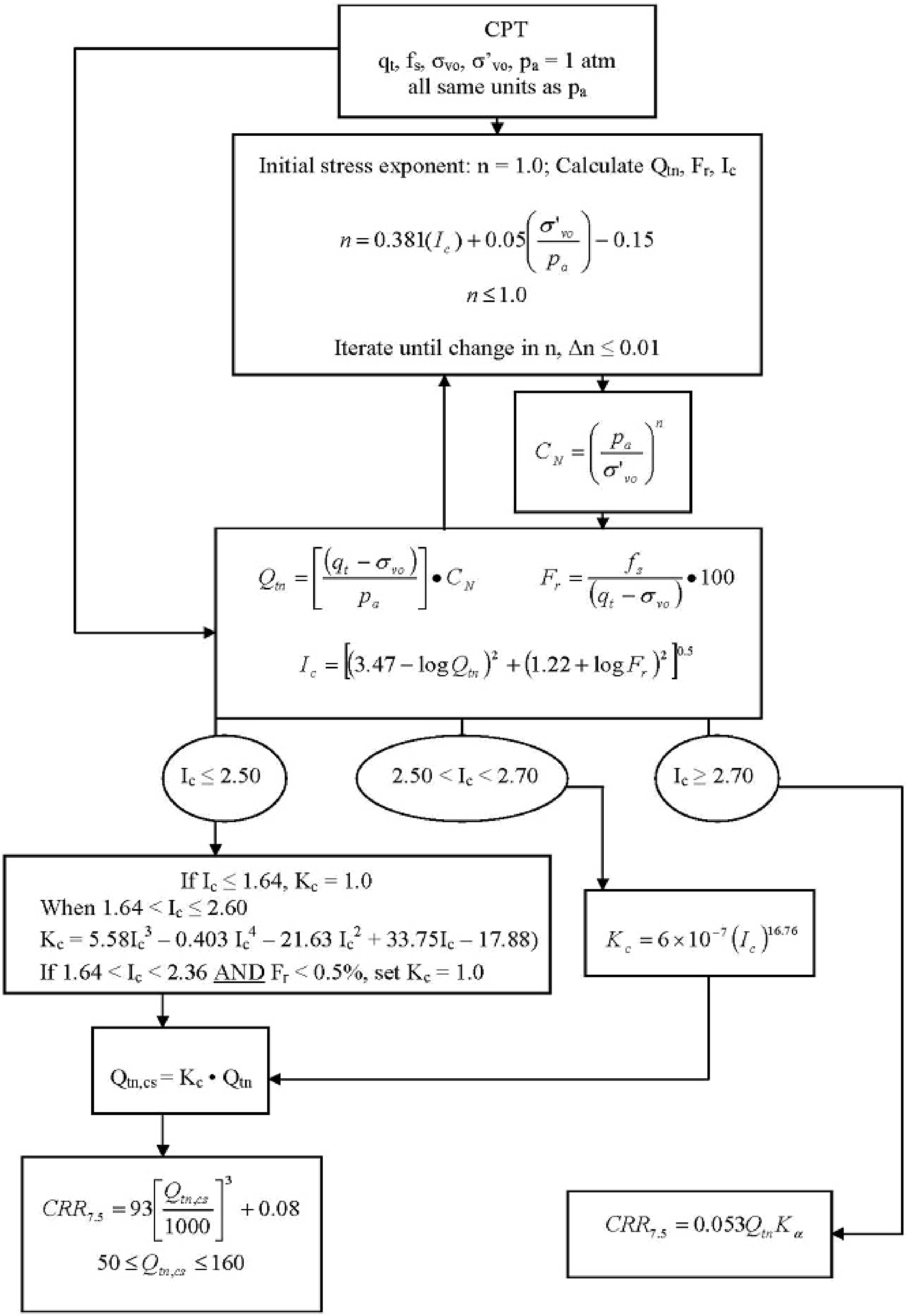
[1] "Estimating liquefaction-induced ground settlements from CPT for level ground", G. Zhang, P.K. Robertson, and R.W.I. Brachman

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]:



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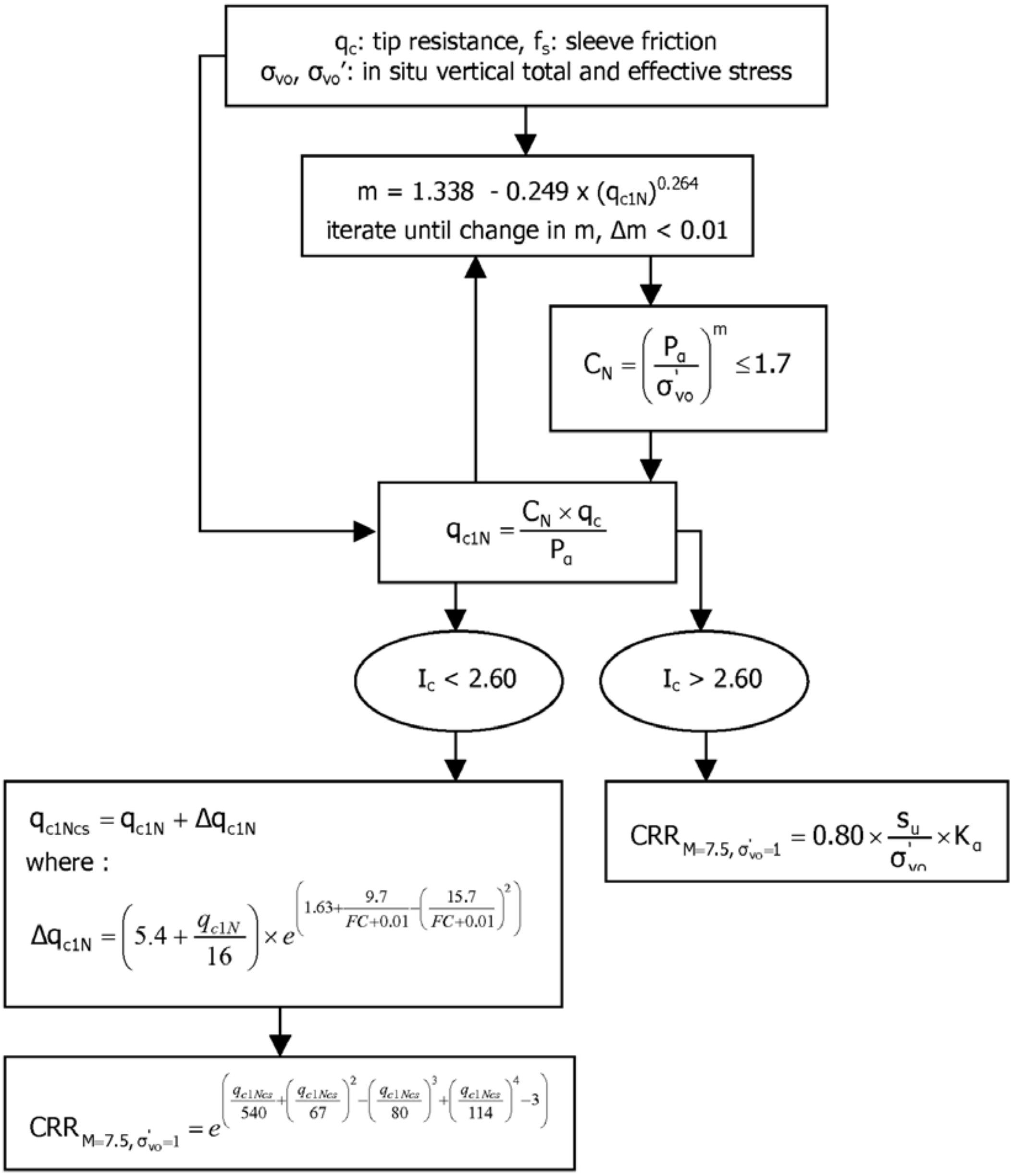
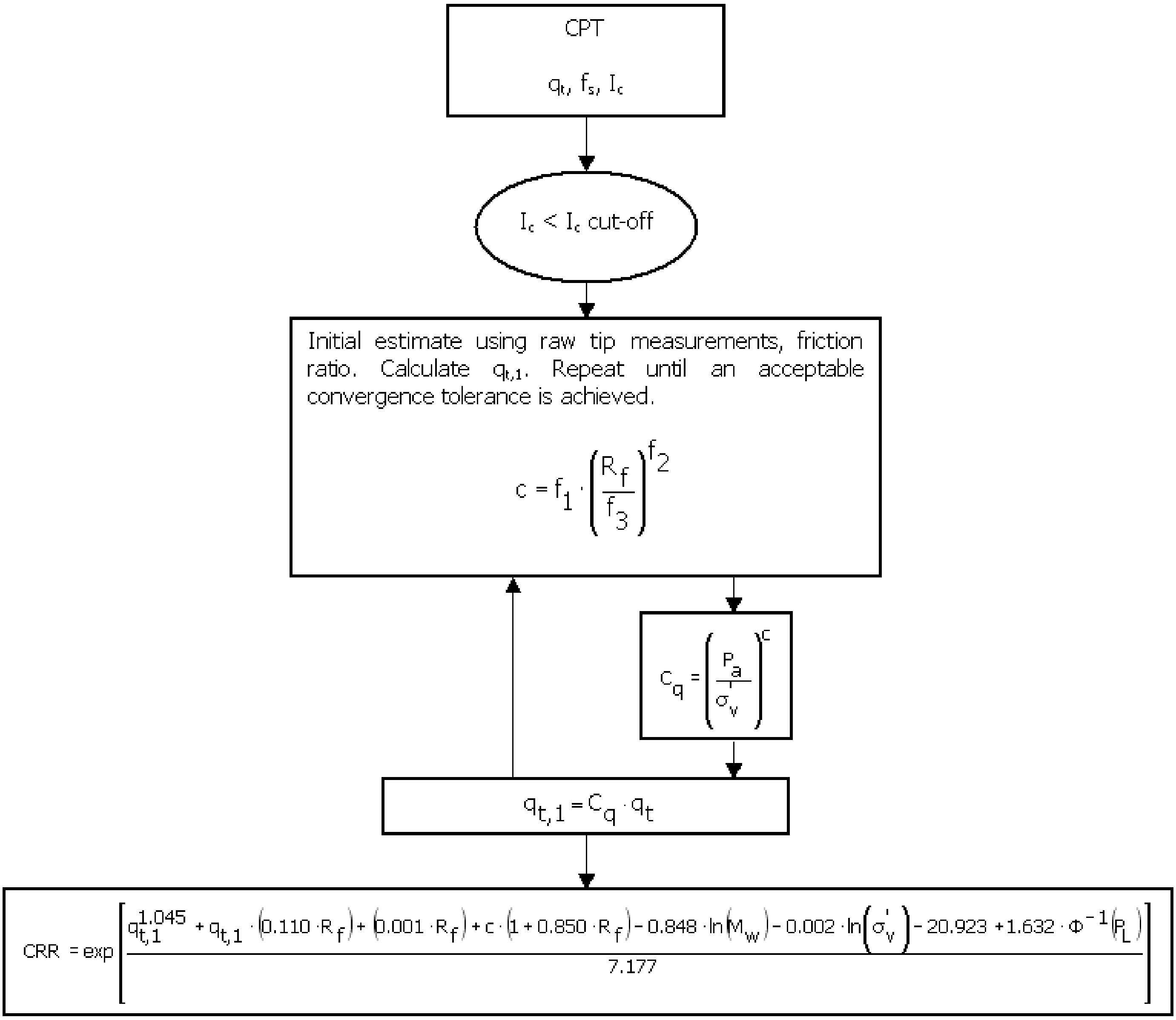
[1] P.K. Robertson, 2009. "Performance based earthquake design using the CPT", Keynote Lecture, International Conference on Performance-based Design in Earthquake Geotechnical Engineering – from case history to practice, IS-Tokyo, June 2009

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)



Initial estimate using raw tip measurements, friction ratio. Calculate $q_{t,1}$. Repeat until an acceptable convergence tolerance is achieved.

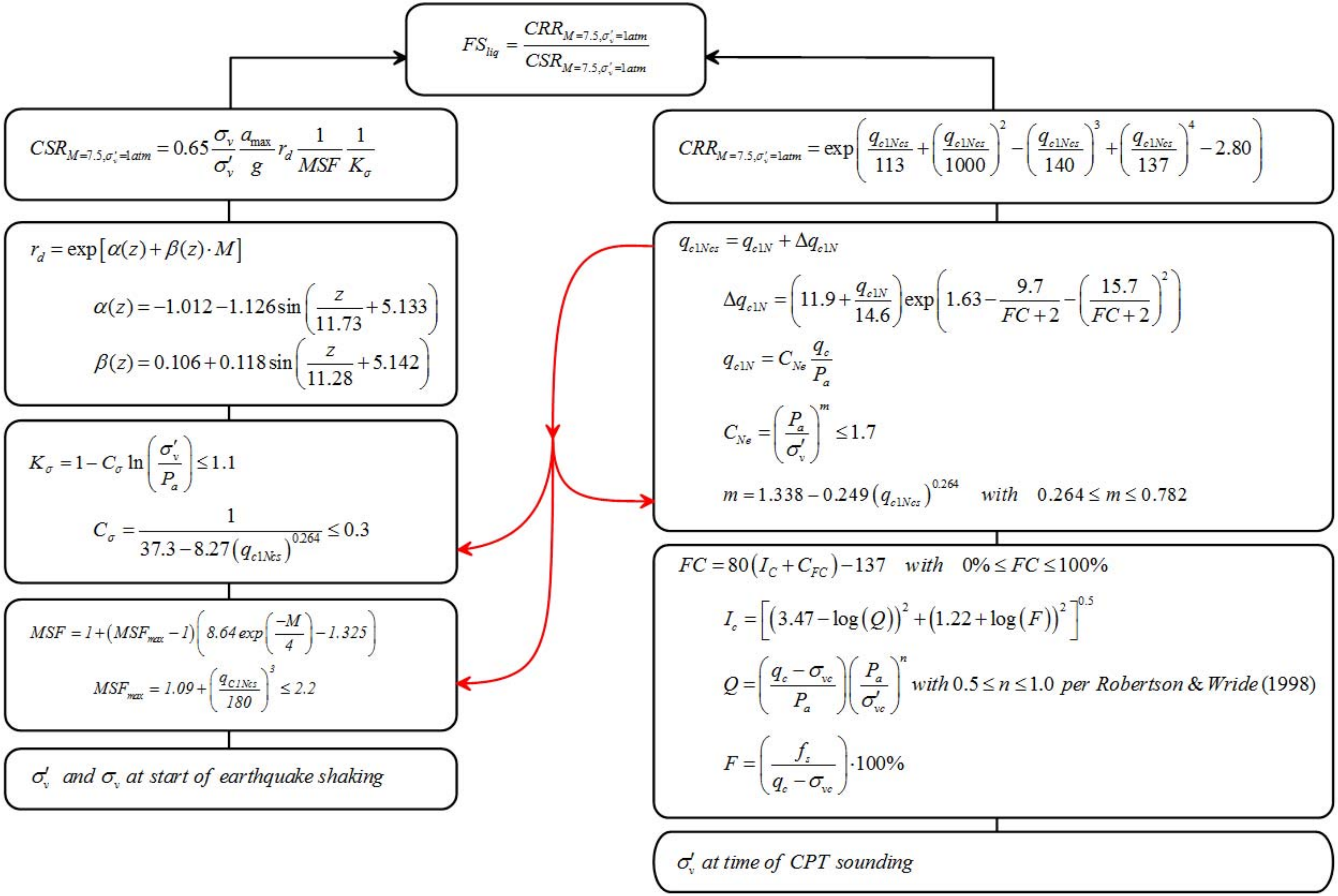
$$c = f_1 \cdot \left(\frac{R_f}{f_3}\right)^{f_2}$$

$$C_q = \left(\frac{P_a}{\sigma'_v}\right)^{c}$$

$$q_{t,1} = C_q \cdot q_t$$

$$CRR = \exp\left[\frac{q_{t,1}^{1.045} + q_{t,1} \cdot (0.110 \cdot R_f) + (0.001 \cdot R_f) + c \cdot (1 + 0.850 \cdot R_f) - 0.848 \cdot \ln(M_w) - 0.002 \cdot \ln\left(\sigma'_v\right) - 20.923 + 1.632 \cdot \Phi^{-1}(P_L)}{7.177}\right]$$

Procedure for the evaluation of soil liquefaction resistance, Boulanger & Idriss(2014)



$$FS_{liq} = \frac{CRR_{M=7.5,\sigma'_v=1atm}}{CSR_{M=7.5,\sigma'_v=1atm}}$$

$$CSR_{M=7.5,\sigma'_v=1atm} = 0.65\frac{\sigma_v}{\sigma'_v}\frac{a_{\max}}{g} r_d \frac{1}{MSF}\frac{1}{K_\sigma}$$

$$r_d = \exp\left[\alpha(z)+\beta(z)\cdot M\right]$$

$$\alpha(z) = -1.012-1.126\sin\left(\frac{z}{11.73}+5.133\right)$$

$$\beta(z) = 0.106+0.118\sin\left(\frac{z}{11.28}+5.142\right)$$

$$K_\sigma = 1-C_\sigma \ln\left(\frac{\sigma'_v}{P_a}\right)\leq 1.1$$

$$C_\sigma = \frac{1}{37.3-8.27\left(q_{c1Ncs}\right)^{0.264}}\leq 0.3$$

$$MSF = 1+\left(MSF_{max}-1\right)\left(8.64\exp\left(\frac{-M}{4}\right)-1.325\right)$$

$$MSF_{max} = 1.09+\left(\frac{q_{c1Ncs}}{180}\right)^3\leq 2.2$$

σ'_v and σ_v at start of earthquake shaking

$$CRR_{M=7.5,\sigma'_v=1atm} = \exp\left(\frac{q_{c1Ncs}}{113}+\left(\frac{q_{c1Ncs}}{1000}\right)^2-\left(\frac{q_{c1Ncs}}{140}\right)^3+\left(\frac{q_{c1Ncs}}{137}\right)^4-2.80\right)$$

$$q_{c1Ncs} = q_{c1N}+\Delta q_{c1N}$$

$$\Delta q_{c1N} = \left(11.9+\frac{q_{c1N}}{14.6}\right)\exp\left(1.63-\frac{9.7}{FC+2}-\left(\frac{15.7}{FC+2}\right)^2\right)$$

$$q_{c1N} = C_{N\sigma}\frac{q_c}{P_a}$$

$$C_{N\sigma} = \left(\frac{P_a}{\sigma'_v}\right)^m\leq 1.7$$

$$m = 1.338-0.249\left(q_{c1Ncs}\right)^{0.264} \quad with \quad 0.264\leq m\leq 0.782$$

$$FC = 80\left(I_C+C_{FC}\right)-137 \quad with \quad 0\%\leq FC\leq 100\%$$

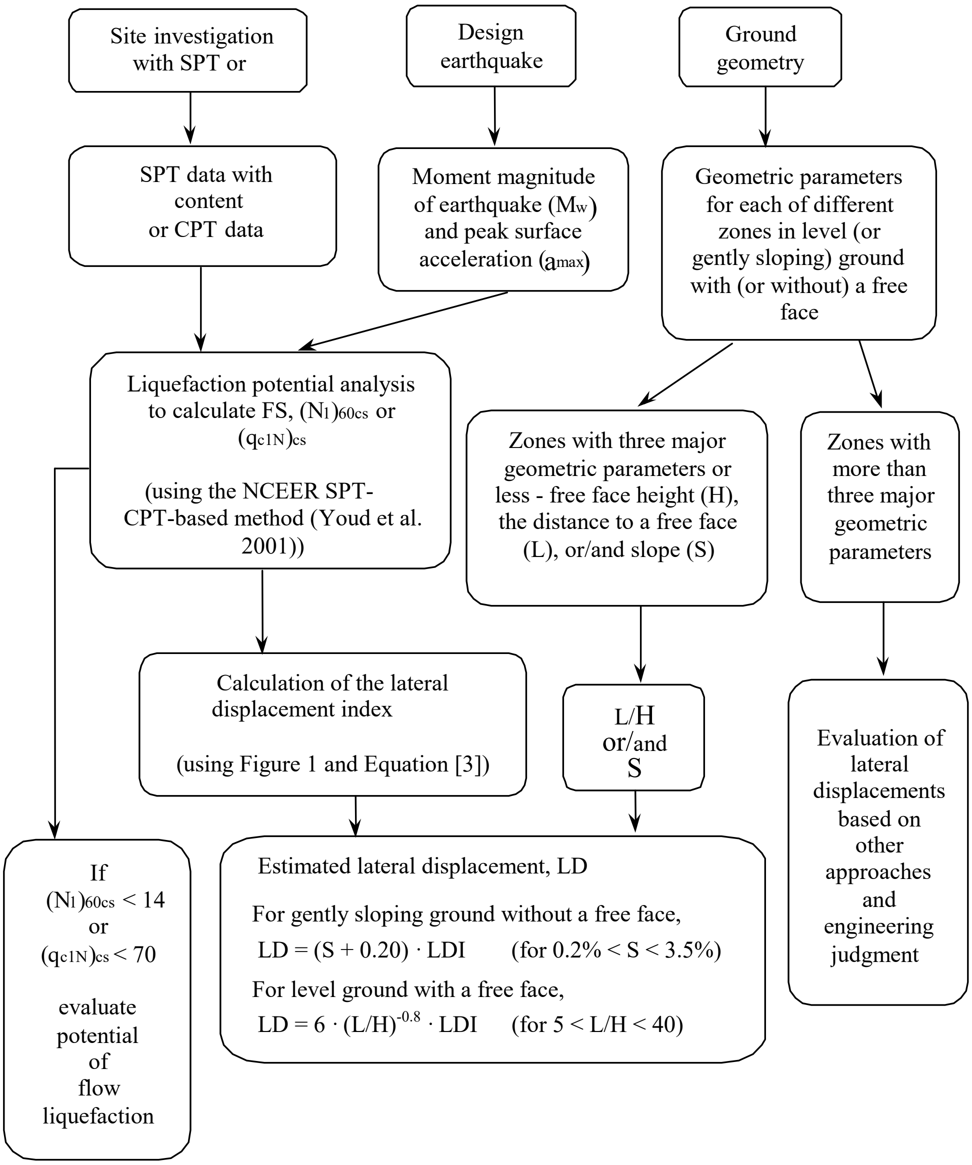
$$I_c = \left[\left(3.47-\log\left(Q\right)\right)^2+\left(1.22+\log\left(F\right)\right)^2\right]^{0.5}$$

$$Q = \left(\frac{q_c-\sigma_{vc}}{P_a}\right)\left(\frac{P_a}{\sigma'_{vc}}\right)^n \quad with\, 0.5\leq n\leq 1.0 \; per\; Robertson\;\&\; Wride\,(1998)$$

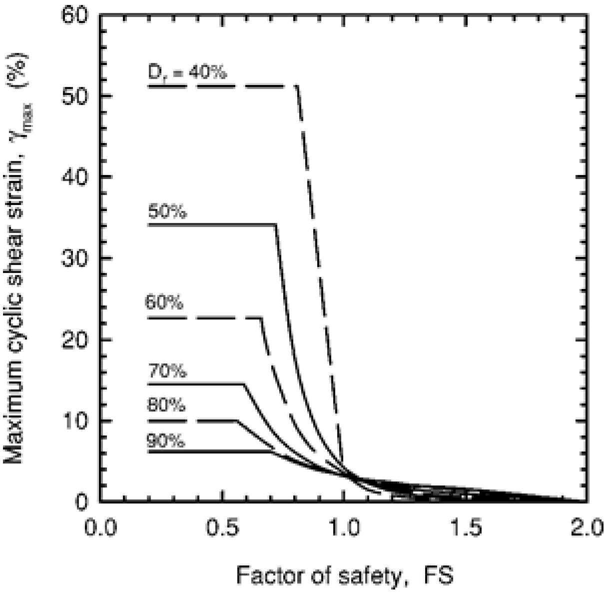
$$F = \left(\frac{f_s}{q_c-\sigma_{vc}}\right)\cdot 100\%$$

σ'_v at time of CPT sounding

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]**

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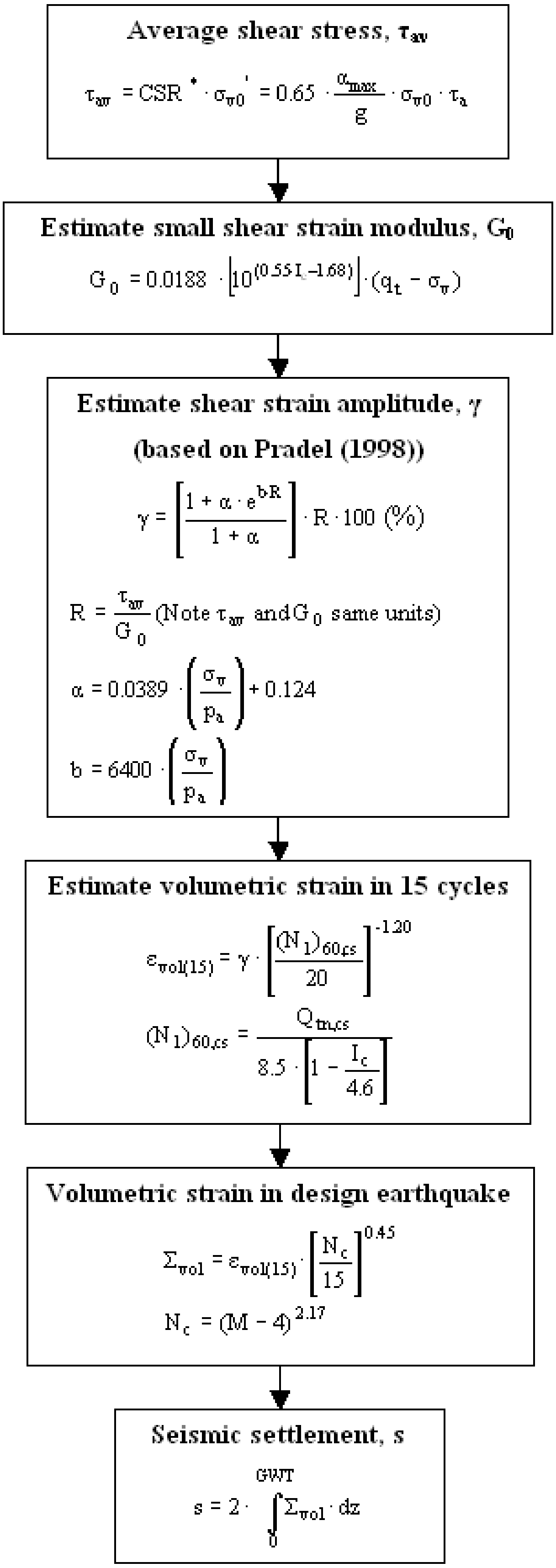
[1] "Estimating liquefaction-induced ground settlements from CPT for level ground", G. Zhang, P.K. Robertson, and R.W.I. Brachman

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.

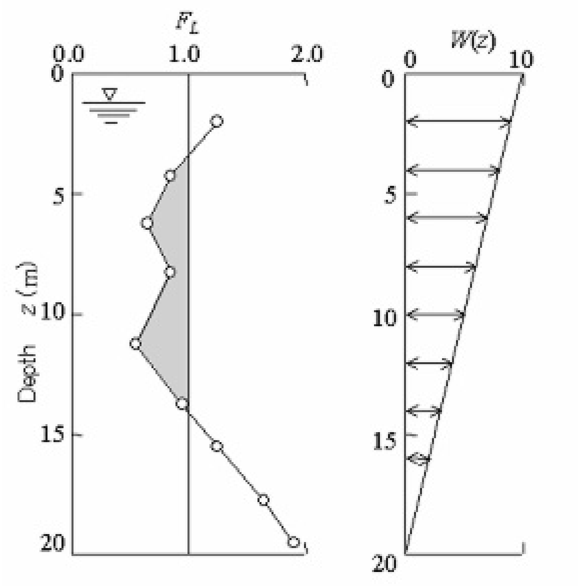
To estimate the severity of liquefaction extent at a given site, LPI is calculated based on the following equation:

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

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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- 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

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Appendix B

Tanks

Appendix 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.*

The tank farm is shown in Figure 2 of the main body of the report. The plan does not include cross-sections or dimensions of the berm 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 petroleum product distribution facility includes 31 aboveground storage tanks. At his time, a catalog of the seismic relevant tank properties has been developed based on the information available. This facility has been in operation for approximately 70 years. Two tanks (28 and 29) are permanently out of service. Most tanks at this facility were built in the 20th century with tanks that were built post 2000 having a smaller volume capacity on average. 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.

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 short timeframe between publication of the assessment forms and the rules' deadline (less than 3 months between publication in March, 2024 and the June 1, 2024 deadline), the API 653 inspection reports have not been able to been completely 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 McCall Oil and Chemical Corporation Spill Prevention and Control and Countermeasure (SPCC) Plan dated April 2023, 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

Appendix C. Form 3: Pipes and Pipeline Systems (PIP)

C.1 PIP1

1. *Provide current Piping and Instrumentation Diagrams for all pipelines on the facility. Provide the jurisdictional limits of the facility pipelines.*

Piping network information has not been able to be extensively cataloged at this time. Currently available information and observations are provided in the narrative report.

Given the age and apparent detailing of much of the site's piping, it is unlikely that much of the piping and its supports were designed to accommodate the ground's anticipated movement due to soil liquefaction in an earthquake. In addition, relatively few existing lateral braces have been observed on site that brace piping. This indicates that portions of the piping system may deform more than desired in an earthquake and could be damaged. However, it should be noted that a majority of the length of the site's piping exists within the boundaries of the large secondary containment areas surrounding the Tank Farm and the Asphalt Plant.

C.2 PIP2

2. *Provide for each pipeline: product, age, inspection history (internal and external).*

   Please see the response to PIP1.

C.3 PIP3

3. *Identify all pipelines that are buried with locations.*

   Please see the response to PIP1.

C.4 PIP4

4. *Identify any pipelines that are on raised racks.*

   Please see the response to PIP1.

C.5 PIP5

5. *Identify any and all pressure tests on pipelines (e.g. static liquid pressure tests, SLPT)*

   Please see the response to PIP1.

C.6 PIP6

6. *Are there any pipe flanges over water.*

   Please see the response to PIP1.

C.7 PIP7

7. *Provide all pipeline stress analyses and dates.*

   Please see the response to PIP1.

C.8 PIP8

8. *Any historical seismic anchor movement.*

   Please see the response to PIP1.

C.9 PIP9

9. *Any interaction of the pipelines with adjacent elements, especially existing tanks, or berms.*

   Please see the response to PIP1.

C.10 PIP10

10. *Corrosion*
    a. *Extensive corrosion*
    b. *Malfunction of cathodic protection systems for an extended period (buried pipes) (inspection reports for CP systems will be included in the submittals)*

    Please see the response to PIP1.

C.11 PIP11

11. *Are there any non-ductile materials (e.g. cast iron, fiberglass, etc.).*

    Please see the response to PIP1.

C.12 PIP12

12. *Any failures of pipeline support*

Please see the response to PIP1.

C.13 PIP13

13. Any evidence of settlement of supports/pipelines

Please see the response to PIP1.

C.14 PIP14

14. For buried pipelines:

  - *a. Possible liquefaction and lateral spreading*
  - *b. Seismic displacement*
  - *c. Surface faulting*
  - *d. Landslides*

Please see the response to PIP1.

C.15 PIP15

15. Long unsupported pipeline segments

Please see the response to PIP1.

C.16 PIP16

16. Brittle elements

Please see the response to PIP1.

C.17 PIP17

17. Threaded connections, flange joints and special fittings

Please see the response to PIP1.

C.18 PIP18

18. Inadequate supports, where a portion of the pipeline may lose its primary support

Please see the response to PIP1.

C.19 PIP19

19. Connections to components with high seismic displacements

Please see the response to PIP1.

C.20 PIP20

20. Inadequate anchorage

Please see the response to PIP1.

C.21 PIP21

21. Short/rigid spans that cannot accommodate relative displacements

Please see the response to PIP1.

C.22 PIP22

22. Damaged supports, including corrosion

Please see the response to PIP1.

C.23 PIP23

23. Long vertical runs with possible drift

Please see the response to PIP1.

C.24 PIP24

24. Large unsupported masses (e.g. valves) attached to pipeline

Please see the response to PIP1.

C.25 PIP25

25. Flanged/threaded connections in high stress locations

Please see the response to PIP1.

C.26 PIP26

26. Leakage (flanges, valves, welds, etc.)

Please see the response to PIP1.

C.27 PIP27

27. Significant external corrosion – or under insulation (Corrosion under Insulation, CUI)

Please see the response to PIP1.

C.28 PIP28

28. Inadequate vertical supports or insufficient lateral restraints

Please see the response to PIP1.

C.29 PIP29

29. Welded attachments to thin-walled pipes

Please see the response to PIP1.

C.30 PIP30

30. Excessive seismic displacement of expansion joints

Please see the response to PIP1.

C.31 PIP31

31. Sensitive equipment possible impact (e.g. control valves)

Please see the response to PIP1.

C.32 PIP32

32. In addition to oil service pipelines, the review should document ethanol, waste oil, fire water, utility, and auxiliary pipelines.

Please see the response to PIP1.

C.33 PIP33

33. Are pipeline materials, seals, gaskets, and other elastomers compatible with products and product additives?

Please see the response to PIP1.

C.34 PIP34

34. Is non-metallic piping used for any pipeline service? If yes, describe briefly.

Please see the response to PIP1.

C.35 PIP35

35. Has an incident or accident involving pipelines occurred? If so, provide a brief description.

Please see the response to PIP1.

C.36 PIP36

36. List all in-kind replacements and component modifications.

Please see the response to PIP1.

C.37 PIP37

37. List all new replacements and component modifications not considered in-kind.

Please see the response to PIP1.

C.38 PIP38

38. Have any new pipelines or components been placed in service? If so, please provide brief details.

Please see the response to PIP1.

C.39 PIP39

39. Have any existing Out of Service (O-O-S) pipelines been placed back in-service? If so, please provide details.

Please see the response to PIP1.

C.40 PIP40

40. Are all piping/pipelines, including components, documented on a current P & ID? If so, please provide as report attachment.

Please see the response to PIP1.

C.41 PIP41

41. Verify that P&ID depicts out-of-service pipelines. Also, are removed pipelines either designated as such or no longer shown on P&ID?

Please see the response to PIP1.

C.42 PIP42

42. Identify vulnerable areas where pipelines are not protected from vehicle or vessel impact. [API 2610]

Please see the response to PIP1.

C.43 PIP43

43. Is any pipeline or valve susceptible to vandalism? [API 2610]. If so, describe briefly.

Please see the response to PIP1.

C.44 PIP44

44. Have the removed portions of replaced pipelines been studied for internal corrosion or other pipe wall anomalies? What are the results of this study?

Please see the response to PIP1.

C.45 PIP45

45. Does the facility have piping flow diagrams indicating all major valves and flow directions for normal conditions as well as upset conditions? Provide as report attachment. [API 2610]

Please see the response to PIP1.

C.46 PIP46

46. Does either configuration or routing of piping or pipelines obstruct access to or removal of other components? If so, describe briefly.

Please see the response to PIP1.

C.47 PIP47

47. Is plastic piping used for hydrocarbon services? If so, have manufacturer specifications been verified that it is rated for oil service?

Please see the response to PIP1.

C.48 PIP48

48. Does a flange connection exist within 20 pipe diameters from the end of any replaced section? If so, identify and document the location.

Please see the response to PIP1.

C.49 PIP49

49. Are there dead legs in the pipelines? If so, identify location(s). [API 2610]

Please see the response to PIP1.

C.50 PIP50

50. Identify all pipelines that do not have a valid SLPT (Static Liquid Pressure Tests) certificate.

Please see the response to PIP1.

C.51 PIP51

51. *Have any piping or pipelines not been used for transferring oil in the last three years? If so, are these designated and/or marked "Out of Service," gas-freed, and physically isolated from oil sources?*

Please see the response to PIP1.

C.52 PIP52

52. *For each identified O-O-S pipeline, specify whether it's above ground, over water, submerged, or buried.*

Please see the response to PIP1.

C.53 PIP53

53. *Have buried or submerged O-O-S pipelines been filled with inert gas or corrosion inhibitors? If so, describe briefly.*

Please see the response to PIP1.

C.54 PIP54

54. *Is there any plan(s) to physically remove any O-O-S pipeline? Indicate which pipelines and the associated schedule for removal.*

Please see the response to PIP1.

C.55 PIP55

55. *Do above ground pipelines have enough flexibility for movement (seismic and thermal) in all directions? [API 2610]*

Please see the response to PIP1.

C.56 PIP56

56. *Has a pipeline stress analysis for oil and fire water service pipelines been performed for:*
    - *a) New piping and pipelines.*
    - *b) Significant routing/relocation of piping.*
    - *c) Any replacement of "not-in-kind" piping.*
    - *d) Any significant rearrangement or replacement of "not-in-kind" anchors and/or supports.*

Significant seismic displacements calculated from the structural assessment.

Please see the response to PIP1.

C.57 PIP56

56. Does the completed PSA represent and reflect current conditions and configurations?

Please see the response to PIP1.

C.58 PIP57

57. What are the maximum transverse and longitudinal seismic displacements used in the PSA?

Please see the response to PIP1.

C.59 PIP58

58. Have all PSAs been performed in accordance with ANSI/ASME B31E or B31.4, as appropriate?

Please see the response to PIP1.

C.59 PIP59

59. Has a pipeline flexibility analysis been performed in accordance with ASME B31.4?

Please see the response to PIP1.

C.60 PIP60

60. Has the largest temperature differential considered all thermal load cases (startup, shutdown, normal and abnormal) have been used in the flexibility analysis?

Please see the response to PIP1.

C.61 PIP61

61. Are there large unsupported masses (e.g. valves) included in the analysis?

Please see the response to PIP1.

C.62 PIP62

62. Are buried pipelines evaluated to withstand the dynamic forces exerted by anticipated traffic loads? [49CFR195]

Please see the response to PIP1.

C.63 PIP63

63. Has the piping system been evaluated for seismic interaction with other elements (equipment, falling objects, other pipelines, etc.)?

Please see the response to PIP1.

C.64 PIP64

64. During a seismic event, is there a possibility of the pipeline(s) impacting safety-sensitive equipment?

Please see the response to PIP1.

C.65 PIP65

65. Are flanged and threaded connections present in high-stress locations? If yes, provide recommendations.

Please see the response to PIP1.

C.66 PIP66

66. Are flanged or threaded connection locations susceptible to high moment loads? If so, are they checked for leakage?

Please see the response to PIP1.

C.67 PIP67

67. Are there adequate expansion loops or joints in the pipeline? If not, provide recommendations.

Please see the response to PIP1.

C.68 PIP68

68. Are check valves relied on for positive shut off in the reverse direction? [API 2610]

Please see the response to PIP1.

C.69 PIP69

69. Are non-ductile materials, iron, cast iron or low melting temperature materials used in hydrocarbon service valves? If so, describe. [API 2610]

Please see the response to PIP1.

C.70 PIP70

70. Are any cast iron or brass fittings used in hydrocarbon service? [API 2610]

Please see the response to PIP1.

C.71 PIP71

71. Is there a documented testing program for all pressure relief valves and are these valves tested on a regular schedule? Provide date(s) of last test(s). [API 2610]

Please see the response to PIP1.

C.72 PIP72

72. Is all piping with blocked sections containing stagnant oil, provided with a relief valve to mitigate pressure build-up due to temperature increase? [API 2610]

Please see the response to PIP1.

C.73 PIP73

73. Identify any information labels on valves that are: Illegible, painted over, damaged, or missing. [API 2610]

Please see the response to PIP1.

C.74 PIP74

74. Are valves susceptible to damage and tampering, protected? [49CFR195]

Please see the response to PIP1.

C.75 PIP75

75. Is access to valves and important appurtenances inhibited during emergencies? [API 2610]

Please see the response to PIP1.

C.76 PIP76

76. Are valve stems oriented in a way that doesn't pose a hazard in operation or maintenance?

Please see the response to PIP1.

C.77 PIP77

77. Are swing check valves installed in vertical down-flow piping? If so, describe briefly

Please see the response to PIP1.

C.78 PIP78

78. Are pressure safety valves set to equal or higher than the maximum allowable working pressure of the protected tank, pipeline, or system?

Please see the response to PIP1.

C.79 PIP79

79. Is discharge from PSVs directed into lower-pressure piping for recycling and proper disposal? If not, what are the discharge areas?

Please see the response to PIP1.

C.80 PIP80

80. Are double-block and bleed valves used for manifold valves?

Please see the response to PIP1.

C.81 PIP81

81. Are all the oil transfer system valves included in a periodic maintenance program? Describe briefly.

Please see the response to PIP1.

C.82 PIP82

82. Are all fire water system valves maintained, inspected, and tested per NFPA-25?

Please see the response to PIP1.

C.83 PIP83

83. Do all SIV and ESD valves conform to MOTEMS requirements? [MOTEMS 3108F.3.2.1 and 3108F.3.2.2]

Please see the response to PIP1.

C.84 PIP84

84. Do valve actuators have a readily accessible manually operated overriding device to enable operation during a power loss?

Please see the response to PIP1.

C.85 PIP85

85. Are torque switches set to stop the motor opening operation at a specified limit switch setting?

Please see the response to PIP1.

C.86 PIP86

86. Is thermal insulation for critical valves inspected and maintained at periodic intervals? How frequent? Are the records kept for at least six years? Are they available?

Please see the response to PIP1.

C.87 PIP87

87. At what interval has the electrical insulation for critical valves been measured for resistance following installation? Are the past records for the six years available?

Please see the response to PIP1.

C.88 PIP88

88. Are utility and auxiliary pipelines included on P&IDs? Are P&IDs current? Attach the report.

Please see the response to PIP1.

C.89 PIP89

89. What is the design standard(s) for stripping and sampling lines, compressed air, nitrogen, or natural gas pipelines? If so, describe the design standard for each.

Please see the response to PIP1.

C.90 PIP90

90. Are there any buried utility/auxiliary pipelines? If so, briefly describe service, location, and corrosion protection.

Please see the response to PIP1.

C.91 PIP91

91. Does vapor collection piping provide proper slope toward condensation collection points?

Please see the response to PIP1.

C.92 PIP92

92. *Are firewater and AFFF pipelines included on P&IDs? Are P&IDs current? Please attach the report.*

    Please see the response to PIP1.

C.93 PIP93

93. *Is carbon steel used for all fire main piping? If not, describe construction material and location.*

    Please see the response to PIP1.

C.94 PIP94

94. *Are any portion(s) of fire water pipelines buried? Are they cathodically protected? What was the date of last inspection?*

    Please see the response to PIP1.

C.95 PIP95

95. *Are all fire water and foam pipelines color-coded per local jurisdiction requirements or ASME A13.1?*

    Please see the response to PIP1.

Appendix D

Piers and Wharves

Appendix D. Form 4: Inspection and Assessment of Piers and Wharves (MOT)

D.1 Basic Information

1. *Provide terminal name, location, company, berthing system, and date*

For the currently available information regarding the piers and wharves located at the McCall Fuel Terminal, see Section 6.0 of the narrative report. Based on the date of construction, the pier is not expected to meet the performance objective.

D.2 Pier Trestle Information

2. *Provide trestle length (ft), width (ft), roadway width (ft), pipe way width (ft), minimum pile length from mudline to trestle (ft), maximum pile length from mudline to trestle (ft), maximum allowable uniform vertical load (psf), as-built design drawings, and structural calculations.*

   Please see the response in D.1.

D.3 Pier Trestle Construction Information

3. *Provide material and corrosion protection information for the following items: piles, pilecaps, deck beams, bracing, bulkhead/retaining wall, and deck*

   Please see the response in D.1.

D.4 Main Loading Platform Information

4. *Provide length (ft), width (ft), minimum pile length from mudline to platform (ft), maximum pile length from mudline to platform (ft), maximum allowable uniform vertical load (psf), maximum design impact load (kips), any tanks, concentrated loads, or areas of live load greater than a minimum, as-built design drawings, and structural calculations (including fender/dolphin capacities).*

   Please see the response in D.1.

D.5 Main Loading Platform Construction Information

5. *Provide material and corrosion protection information for the following items: piles/batter piles, pilecaps, deck beams, bracing, bulkhead/retaining wall, and deck*

   Please see the response in D.1.

D.6 MOT01

6. *Has an overall above water inspection of the terminal been performed, looking for gross damage or deterioration of structural items, or potentially dangerous situations?*

   Please see the response in D.1.

D.7 MOT02

7. *Has an inspection been made of all above water steel components?*

   Please see the response in D.1.

D.8 MOT03

8. *Has an underwater inspection been made of all underwater steel components? If not, what is the date of the last underwater inspection?*

   Please see the response in D.1.

D.9 MOT04

9. *Did the underwater inspection include corrosion measurements using NDT methods?*

   Please see the response in D.1.

D.10 MOT05

10. *Does the above water portion of steel structures have a protective coating (paint or other)?*

    Please see the response in D.1.

D.11 MOT06

11. *If H-beams are present, have corrosion measurements of the web and flanges been taken at critical locations?*

    Please see the response in D.1.

D.12 MOT07

12. Is there a cathodic protection system installed at this facility?

Please see the response in D.1.

D.13 MOT08

13. If there is cathodic protection, has the system been inspected or the effectiveness of the system tested?

Please see the response in D.1.

D.14 MOT09

14. If there is a sheet piling retaining wall, has it been inspected for corrosion, scour, and loss of fill? If there are tie-backs, have they been inspected, and if not, why not?

Please see the response in D.1.

D.15 MOT10

15. Has an inspection been made of all above water concrete components?

Please see the response in D.1.

D.16 MOT11

16. If there is a concrete deck, has the underside of the deck been inspected?

Please see the response in D.1.

D.17 MOT12

17. Has an underwater inspection been made of the piles?

Please see the response in D.1.

D.18 MOT13

18. If not, what is the date of the last underwater inspection?

Please see the response in D.1.

D.19 MOT14

19. Is there evidence of damage to the concrete structure from erosion or overstressing?

Please see the response in D.1.

D.20 MOT15

20. Is there evidence of chemical damage to the concrete?

Please see the response in D.1.

D.21 MOT16

21. Is there evidence of corrosion of the reinforcing steel?

Please see the response in D.1.

D.22 MOT17

22. Is the concrete protected using surface coatings or linings, if so, what is the condition?

Please see the response in D.1.

D.23 MOT18

23. Has an inspection been made of all above-water timber components?

Please see the response in D.1.

D.24 MOT19

24. Is there any cracking or other surface damage in the above-water timber structural members?

Please see the response in D.1.

D.25 MOT20

25. Has an underwater inspection been made of the piles?

Please see the response in D.1.

D.26 MOT21

26. If not, what is the date of the last underwater inspection?

Please see the response in D.1.

D.27 MOT22

27. Is there any evidence of marine borer damage?

Please see the response in D.1.

D.28 MOT23

28. Are the piles protected with plastic or other type of coating?

Please see the response in D.1.

D.29 MOT24

29. If so, does the protective layer appear to be effective?

Please see the response in D.1.

D.30 MOT25

30. If there are bracing members, have the bracing connections been inspected?

Please see the response in D.1.

D.31 MOT26

31. Has a laydown pattern with equipment loads been provided of the wharf/pier deck?

Please see the response in D.1.

D.32 MOT27

32. What assumptions have been made for the pipeline trestle and on the wharf/pier deck?

Please see the response in D.1.

D.33 MOT28

33. *Has the anchorage, flexibility and seismically-induced interaction of these components been considered?*

    Please see the response in D.1.

Appendix E

Liquefied Natural Gas Tanks and Pipelines

Appendix E

LIQUEFIED NATURAL GAS TANKS AND PIPELINES

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

Appendix F

Berms and Dikes

Appendix F

BERMS AND DIKES

This appendix presents the preliminary seismic vulnerability assessments performed to date for the earthen berm and concrete wall at the site. As shown in Figure 2 in the main body of this report, an approximate 8 to 12-foot-high earthen berm capped in asphalt surrounds the Tank Farm ASTs, and an approximate 3.2-foot-high concrete wall surrounds the Asphalt Plant ASTs.

F.1 Earthen Berm

The earthen berm has a capacity of approximately 340,000 barrels (bbls), which is approximately 120 percent of the largest AST volume. The bermed area is graded to drain to four catch basins. The catch basins discharge to an oil/water separator. The berm outlet is controlled by a positive seal gate valve, which is normally locked and closed.

Based on the preliminary analysis performed to date as presented in Appendix A, Section A.5, approximately three-fourths of the earthen berm is located within the potential flow failure zone as shown in Figure 7 in the main body of this report. The earthen berm would be subjected to the same deformations as the adjacent ground if lateral spreading were to occur.

F.2 Concrete Wall

The walled area has a containment capacity of approximately 10,554 bbls, which is approximately 109 percent of the largest AST volume. Runoff from the Asphalt Plant drains to a catch basin and into a drainage sump. The manually controlled sump pump discharges onto the ground in drainage area, where it would then flow into a catch basin before being routed to an oil/water separator.

Based on the preliminary analysis performed to date as presented in Appendix A, Section A.5, the concrete wall is located beyond the potential flow failure zone identified in Figure 7 in the main body of this report, but still within an area subject to potentially large lateral displacements during a design earthquake.

F.3 Berms and Dikes Surrounding Tank Farms Checklist

The following is a checklist to satisfy the Oregon DEQ requirements for berms and dikes surrounding tank farms:

1. *Does the geotechnical investigation or report show any variation between each length (all sides) of the berm/dike? Is the berm susceptible to differential settlement or liquefaction? Can the protective layer remain intact under differential settlement or liquefaction condition? [BER1].*

   Response: Both the earthen berm surrounding the Tank Farm ASTs and the concrete wall surrounding the Asphalt Plant ASTs are located in the areas that are susceptible to liquefaction. Please refer to Sections F.1 and F.2 for our preliminary evaluations on the earthen berm and the concrete wall.

   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 GeoEngineers using high-level empirical formulations. Additional engineering work must be done to more precisely understand the magnitude of total and differential settlement caused by seismic soil liquefaction. Once the precise magnitude of the site's total and differential seismic settlement is understood, the concrete secondary containment walls can be evaluated for their ability to accommodate the settlement

2. *Are there any 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 for seismic loads (demand and capacity) calculations including possible overturning, stability, potential differential settlement. Provide the age of the dike and all structural properties. [BER2].*

   Response: The cumulative length of the site's cast-in-place concrete secondary containment walls exceeds 800 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.

3. *Are there any penetrations for piping or drainage – explain and sketch. [BER3]*

   Response: The cumulative length of the site's cast-in-place concrete secondary containment walls exceeds 800 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.

4. *Is there any evidence of water ponding at the base. If the tank far is on a slope, is the downslope dike length and volume sufficient to facilitate the maximum postulated tank farm leakage? [BER4].*

   Response: The tank farm areas are generally flat with only slight slopes. Based on the McCall Oil and Chemical Corporation Spill Prevention and Control and Countermeasure (SPCC) Plan dated April 2023, 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.

5. *Is the design sufficient (demand and capacity) to be fit-for-purpose post DE event? The potential leakage should be contained post-earthquake or fire. [BER5]*

   Response: 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 described in Section 3.1 in the main body of this report and Appendix A, Section A.4. Additional assessment is required by GeoEngineers to more precisely calculate the site's expected settlement due to soil liquefaction. In addition, additional assessment is required to understand the potential for leaking of the secondary containment walls.

6. *What is the plan to evacuate the spillage post event? [BER6]*

   Response: The fuel terminal has a detailed SPCC plan. The most recent version of this document is dated April 2023. 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.

7. *Are there any penetrations, pipelines, or other possible openings in the dike. Are there rigid pipeline penetrations that might rupture during seismic displacement? [BER7]*

   Response: Please see the response to BER3.

8. *Any evidence of other damage to the existing dike, and does it satisfy the DE requirements of OAR 340-300-0003. [BER8]*

   Response: Please see the response to BER2.

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 ally hydrodynamic forces per ASCE 7-22 Section 15.6.5. [BER9]*

   Response: 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.

10. *Assess possible deterioration from rodents, erosion, liquefaction, cracks, vegetation, or other visible signs of distress. [BER10]*

    Response: We will assess the possible deterioration in the next phase.

11. *Evaluate the soil for permeability and fitness-for-purpose, based on a local geotechnical review, for all areas and lengths. What soil types are used for the berm and is there multiple different soil types for various layers. Provide a geotechnical fitness-for-purpose of earth berm and underlaying foundation material. [BER11]*

    Response: There is no available geotechnical information for soil layers and types in the earthen berm at this time. Additional explorations and evaluation will be needed in the next phase.

12. *Determine if the height is sufficient to accommodate the postulated maximum spill volumes. Has the height decreased over time. Compare actual height to the design dimensions. [BER12]*

    Response: Based on the McCall Terminal SPCC plan, the earthen berm and soil floor is sufficiently impervious to contain spilled oil for at least 72 hours. The berm has a containment capacity of about 340,000 bbls. Approximately 21,500 bbls are required to contain a 24-hour, 25-year storm (about 4 inches of rain), so the maximum available containment volume is approximately 318,500 bbls. The McCall terminal stores no more than 270,000 bbls in any single AST within the Tank Farm. Additional evaluation will be needed in the next phase to measure the actual height of the earthen berm.

13. *For minimum top horizontal widths (Ref: Table 5-1, "Earth Dams and Reservoirs", TR-60, U.S. Department of Agriculture, Conservation Engineering Division, July 2005. [BER13]*

Height (ft)	*Minimum width (ft) at top*
15-18	*8*
20-24	*10*

Response: There is no information available at this time for the top horizontal widths along the earthen berm. Additional assessment will be needed in the next phase.

14. *Is there a protective layer over the soil material, and if so, specify the type, durability, estimated remaining life. [BER14]*

Response: Based on the McCall Terminal SPCC plan, the earthen berm is capped in asphalt. There is no information available at this time for durability and estimated remaining life. Additional assessment will be needed in the next phase.

15. *Is there evidence of earlier failures, patched sections, or other possible historical damage. [BER15]*

Response: There is no information available at this time for the evidence of earlier failures, patched sections, or other historical damage. Additional assessment will be needed in the next phase.

16. *Is there water ponding at the interior base of the berm. If the tank farm is not on level soil, can the downslope berm facilitate the required spill volume? [BER16]*

Response: The tank farm areas are generally flat. Based on the McCall Terminal SPCC plan, the bermed area is graded to drain to four catch basins. The catch basins discharge to an oil/water separator.

17. *Are there sheet piles in addition to the soil? [BER17]*

Response: Given known information at this time, there are no sheet piles in addition to the soil.

18. *Does the geotechnical report show any differential characteristics from one end of each segment of the berm to the other? How will this be accommodated during an earthquake with possible differential settlement/motion. Is there any evidence of subsidence? [BER18]*

Response: Based on the existing subsurface information, the subsurface conditions were considered relatively consistent across the site, as presented in Section 2.2 in the main body of this report, while the thickness of each soil layer varies. We propose additional subsurface explorations be performed in the next phase that include a two-dimensional (2D) geophysical survey to develop 2D Vs profiles across the site and capture site variability. Based on the results from the 2D geophysical survey, the subsurface conditions can be refined, and earthquake-induced settlements can be evaluated based on the continuous 2D Vs profile to better capture the differential settlement along the earthen berm.

19. *Possible seepage beneath the base? What about a piping failure through the berm? How is this being assessed? [BER19]*

Response: We will assess the possible seepage and piping failure in the next phase.

20. *Verify that the earth berm will satisfy the seismic demand as provided in OAR 340-300-0004. [BER20]*

Response: As presented in Section F.1, based on the results of simplified analyses, the earthen berm is located within the potential flow failure zone as shown in Figure 7 in the main body of this report. The earthen berm would be subjected to the same deformations as the adjacent ground if lateral spreading were to occur during a Mw 9.0 earthquake event with MCE_R ground shaking intensity.

21. *Possible slumping of the berm during an earthquake? [BER21]*

    Response: Please see the response to BER20.

Appendix G

Building and Building Structures

Appendix G. Form 7: Buildings and Building-Like Structures (BLG)

G.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 does not have any buildings that serve a product storage or product handling function. This checklist is not applicable for the facility to meet the OAR 340-300 performance objective.

G.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.*

Please see response to BLG1.

G.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.*

Please see response to BLG1.

G.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 BLG1.

G.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.*

Please see response to BLG1.

Appendix H

Fire Detection and Suppression

Appendix H. Form 8: Fire Detection and Suppression (FDS)

H.1 FDS1

1. *A site-specific Fire Protection Assessment shall be prepared by a registered engineer or a competent fire protection professional. The assessment shall consider all of the hazards and risks associated with the facility, and shall include but not be limited to, the elements of pre-fire planning, goals, resources, organization, strategy and tactics, including the following:*
   a. *The characteristics of the entire facility (e.g. tanks, marine terminals, pipeline systems, etc.)*
   b. *Product types, and any other flammables, corrosive or toxic chemicals at the facility and fire scenarios*
   c. *Possible collateral fire damage to adjacent facilities*
   d. *Fire-fighting capabilities, including the availability of water (flow rates and pressure), foam type and associated shelf life, proportioning equipment, and vehicular access.*
   e. *Justify the selection of appropriate extinguishing agents.*
   f. *Calculation of water and foam capacities, as applicable, consistent with area coverage requirements.*
   g. *Coordination of emergency efforts (company and external fire departments)*
   h. *Emergency escape routes for both personnel safety and required external fire department vehicles.*
   i. *Requirements for fire drills, training of all personnel, and the use of non-fixed equipment.*
   j. *Life safety, safe egress, and denoted safety zone areas available to all personnel.*
   k. *Rescue for personnel (if an oil terminal includes vessel personnel).*
   l. *Sufficient cooling water for pipelines and valves exposed to the heat (internal to the facility or outside).*
   m. *Contingency planning when supplemental fire support is not available. What are the mutual aid agreements and are they sufficient.*
   n. *Consideration of adverse conditions, such as electrical power failure, steam failure, fire pump failure, an earthquake or other damage to the fire water system.*
   o. *Provide the date of the assessment and schedule to review/update. This assessment must be updated in accordance with OAE 340-300-0003.*

The facility has a variety of fire protection systems and fire prevention processes. At this time, little information has been able to be gathered regarding the cataloging of fire detection and fire protection systems present at the McCall Fuel Terminal. There is no existing database of all the existing equipment. Additional data gathering is necessary to provide answers to this checklist.

H.2 FDS2

2. *Is there a common or separate fire system for each berthing system? Are there any firewalls?*

   Please see the response in FDS1.

H.3 FDS3

3. *Is all existing fire protection equipment shown on an equipment layout drawing? Identify by drawing number(s).*

   Please see the response in FDS1.

H.4 FDS4

4. *Are fire water pipelines shown on P&IDs or "asbuilt" drawings? Identify diagram or drawing number(s)*

   Please see the response in FDS1.

H.5 FDS5

5. *Have the following items been field verified (location and condition) to ensure operability:*
   - *a) Water supply?*
   - *b) Fire pumps?*
   - *c) Fire water jockey pumps?*
   - *d) Hydrant locations?*
   - *e) Foam supply?*
   - *f) Wheeled extinguishers?*
   - *g) Portable extinguishers?*
   - *h) Hose connections?*
   - *i) Hose storage stations?*
   - *j) Fire alarm pull stations?*
   - *k) Fire Detector(s)*
   - *l) Fire monitors?*
   - *m) Fire boat connections?*
   - *n) International Shore Connection?*

   Note leakage, physical damage, or corrosion. Summarize any deficiencies or recommendations.

   Please see the response in FDS1.

H.6 FDS6

6. *Are all fire water pumps inspected, maintained, and tested per NFPA-25?*

   Please see the response in FDS1.

H.7 FDS7

7. *Describe how the terminal protected from static electricity, lightning, and stray currents (API 2003)*

   Please see the response in FDS1.

H.8 FDS8

8. *Verify that cargo manifolds and loading arms conform to electrical isolation requirements. Provide brief details*

   Please see the response in FDS1.

H.9 FDS9

9. *If the wharf structure is steel, is there an insulating flange that electrically isolates the pipeline on wharf from the first pipeline support on-shore? Are pipeline(s) electrically bonded to the wharf? [API 2003, Section 6.3]*

   Please see the response in FDS1.

H.10 FDS10

10. *If the wharf is concrete or timber, is the pipeline grounded either to the water or on shore? [API 2003, Section 6.3]*

    Please see the response in FDS1.

H.11 FDS11

11. *If a multi-berth terminal, what is the distance between adjacent manifolds?*

    Please see the response in FDS1.

H.12 FDS12

12. Fill out the table below for ESD valves

Please see the response in FDS1.

H.13 FDS13

13. What is the ESD effective time to stop the flow of oil after initiating closure action?

Please see the response in FDS1.

H.14 FDS14

14. For ESD systems, are actuation stations located such that ESD can be initiated within 30 seconds of a shutdown order received on the wharf?

Please see the response in FDS1.

H.15 FDS15

15. Are communications or control circuits synchronized for the simultaneous closure of the SIVs and the shutdown of the loading pumps?

Please see the response in FDS1.

H.16 FDS16

16. Is there an alarm to indicate failure of the primary power source? Describe location.

Please see the response in FDS1.

H.17 FDS17

17. Is there a secondary power source should the primary power source fail?

Please see the response in FDS1.

H.18 FDS18

18. Is the automated ESD system tested periodically? Date of last test?

Please see the response in FDS1.

H.19 FDS19

19. Are electrical, instrument, and control systems (i.e. ESD system), located within hazardous classified areas, protected from fire damage, if such equipment is used to activate equipment needed to control a fire or mitigate its consequences. Have API Pub 2218 guidelines been followed and the Oregon state electrical code?

Please see the response in FDS1.

H.20 FDS20

20. Are all ESD valves located near the dock manifold connection or loading arm? Describe location(s)?

Please see the response in FDS1.

H.21 FDS21

21. Fill out the table below for SIVs.

Please see the response in FDS1.

F.22 FDS22

22. Are all SIVs for each cargo line located on shore and clustered together?

Please see the response in FDS1.

H.23 FDS23

23. Are SIVs clearly marked with the identification of each associated pipeline?

Please see the response in FDS1.

H.24 FDS24

24. Is there adequate lighting to identify and manually operate the SIVs?

Please see the response in FDS1.

H.25 FDS25

25. Is there a manual reset to restore the SIV system after shutdown?

Please see the response in FDS1.

H.26 FDS26

26. Are thermal expansion relief valves installed to relieve pressure from a blocked-in offshore segment of the pipeline when the SIV is in the closed position?

Please see the response in FDS1.

H.27 FDS27

27. Does the MOT have a permanently installed automated fire detection or sensing system?

Please see the response in FDS1.

H.28 FDS28

28. Are fire (flame, heat, or smoke) sensors installed in all enclosed spaces within classified areas?

Please see the response in FDS1.

H.29 FDS29

29. Is each fire detection system of the manual reset type?

Please see the response in FDS1.

H.30 FDS30

30. Is each fire-detection system capable of continuous monitoring?

Please see the response in FDS1.

H.31 FDS31

31. Do detection devices automatically initiate ESD?

Please see the response in FDS1.

H.32 FDS32

32. Is there periodic testing of the detection system? When last tested?

Please see the response in FDS1.

H.33 FDS33

33. Are fire detection system specifications available and have these been verified by the audit team?

Please see the response in FDS1.

H.34 FDS34

34. Are there automatic and manual fire alarm initiating devices at strategic locations?

Please see the response in FDS1.

H.35 FDS35

35. Are triggered alarms visible and audible by all MOT and vessel personnel involved in transfer operations?

Please see the response in FDS1.

H.36 FDS36

36. During a triggered MOT fire alarm, is the alarm also visually and audibly displayed at the facility's control center?

Please see the response in FDS1.

H.37 FDS37

37. Is the fire alarm system integrated with the ESD system?

Please see the response in FDS1.

H.38 FDS38

38. Is the alarm system tested per NFPA-72? When last tested?

Please see the response in FDS1.

H.39 FDS39

39. *Are fire alarm system manufacturer maintenance and testing requirements available and have these been complied with and verified by the audit team?*

    Please see the response in FDS1.

H.40 FDS40

40. *Is the firewater flow rate consistent with the requirements of Table 19.1 of ISGOTT (International Safety Guide for Oil Tankers and Terminals)? (Table repeated in PIANC WG 253B, "Recommendations for the Design and Assessment of Marine Oil, Gas and Petrochemical Terminals", 2022)*

    Please see the response in FDS1.

H.41 FDS41

41. *Field verify fire pump capacity and pressure ratings and compare to the latest pump flow test results. Do pump ratings and test results match? Any recommendations? Provide the latest flow test results in audit report and reference here.*

    Please see the response in FDS1.

H.42 FDS42

42. *Verify that water-based fire protection systems have been maintained by the MOT operator per NFPA-25.*

    Please see the response in FDS1.

H.43 FDS43

43. *For diesel-powered pumps, field verify the following:*
    - *a) Fuel tank at least 2/3 full.*
    - *b) Battery electrolyte level is within acceptable range.*
    - *c) Crankcase oil is within acceptable range.*

    Coolant level is within acceptable range. Note observation results. [NFPA-25]

    Please see the response in FDS1.

H.44 FDS44

44. For seawater drafting pumps, field verify that pump suction is free from marine growth and other obstructions. Note observation results.

Please see the response in FDS1.

H.45 FDS45

45. Is a standby fire pump available? If so, describe.

Please see the response in FDS1.

H.46 FDS46

46. Does the fire suppression include coverage for:

a) Marine structures (pier, wharf, or approach trestle)?
b) Terminal cargo manifold?
c) Vessel manifold?
d) Cargo transfer systems?
e) Sumps?
f) Pipelines?
g) Control stations?

Summarize any deficiencies or recommendations.

Please see the response in FDS1.

H.47 FDS47

47. What is the maximum separation distance between hydrants?

Please see the response in FDS1.

H.48 FDS48

48. Is the facility currently accessible to fire trucks and mutual aid equipment? Are firewater connections accessible to fire trucks or mutual aid equipment? Describe access locations.

Please see the response in FDS1.

H.49 FDS49

49. Do hoses and monitors have the capability of applying two independent water streams to cover the cargo manifold, transfer system, vessel manifold and sumps?

Please see the response in FDS1.

H.50 FDS50

50. If there is a wet system, is it pressurized?

Please see the response in FDS1.

H.51 FDS51

51. Does the terminal have a pump-in point for firefighting vessels and trucks to augment the fire water supply to the shore fire main grid?

Please see the response in FDS1.

H.52 FDS52

52. Are pump-in-points located at a safe distance from high-risk areas, such as sumps, manifolds, loading arms, etc.?

Please see the response in FDS1.

H.53 FDS53

53. Have calculations as to aqueous film forming foam (AFFF) type, flow rates, and application duration been verified by the audit team?

Please see the response in FDS1.

H.54 FDS54

54. Record AFFF type, quantity, and location.

Please see the response in FDS1.

H.55 FDS55

55. Is AFFF proportioning equipment located at least 100 feet from sumps, manifolds and loading arms?

Please see the response in FDS1.

H.56 FDS56

56. Is a facility program/procedure in place to ensure that AFFF is replaced consistent with the manufacturer's recommendations? Date of last AFFF replacement?

Please see the response in FDS1.

H.57 FDS57

57. Can all monitors be oscillated and moved throughout their full range? [NFPA-25]

Please see the response in FDS1.

H.58 FDS58

58. Is AFFF educator tubing and its connection to monitors, free from obstructions and in good serviceable condition?

Please see the response in FDS1.

H.59 FDS59

59. Are monitors located to provide an unobstructed path between the monitor and the target area?

Please see the response in FDS1.

H.60 FDS60

60. What is the maximum vessel manifold height (ballast draft, high tide) above the MOT deck?

Please see the response in FDS1.

H.61 FDS61

61. If the maximum vessel manifold height is greater than 30 feet above the wharf deck, are the monitors raised?

Please see the response in FDS1.

H.62 FDS62

62. Are there sprinklers and/or remotely controlled water/foam monitors to protect personnel, escape routes, shelter locations and the fire water system?

Please see the response in FDS1.

H.63 FDS63

63. Are there isolation valves in the firewater and foam lines, and are the isolation valves at least 150 feet from the manifold and loading arm/hose area?

Please see the response in FDS1.

H.64 FDS64

64. Is supplemental fire suppression necessary to meet minimum suppression requirements?

Please see the response in FDS1.

H.65 FDS65

65. If yes, does it provide less than 25% of the fire water/foam requirements of the Fire Protection Assessment?

Please see the response in FDS1.

H.66 FDS66

66. How much time from the activation of the fire alarm does it take for supplementary resources to arrive?

Please see the response in FDS1.

H.67 FDS67

67. *Is there a contingency wherein the supplemental fire/foam resource is not available? Is this considered in the Fire Protection Assessment?*

Please see the response in FDS1.

H.68 FDS68

68. *The extent of such protection shall be determined by an evaluation based on fire protection engineering principles, analysis of local conditions, hazards within the facility, and exposure to or from other property. The evaluation shall determine the following (Section 12.2.1):*
    - *a. The type, quantity, and location of equipment necessary for the detection and control of fires, leaks, and spills of LNG, flammable refrigerants, or flammable gases,*
    - *b. The type, quantity, and location of equipment necessary for the detection and control of potential nonprocess and electrical fires,*
    - *c. The methods necessary for the protection of the equipment and structures from the effects of fire exposure,*
    - *d. Requirements for fire protection water systems,*
    - *e. Requirements for fire-extinguishing and other fire control Equipment,*
    - *f. The equipment and processes to be incorporated within the ESD system, including analysis of subsystems, if any, and the need for depressurizing specific vessels or equipment a. during a fire emergency,*
    - *g. The type and location of sensors necessary to initiate the automatic operation of the ESD system or its subsystems,*
    - *h. The availability and duties of individual plant personnel and the availability of external response personnel during an emergency,*
    - *i. The protective equipment, special training, and qualification needed by individual plant personnel as specified by NFPA 600, Standard on Industrial Fire Brigades, for their respective emergency duties,*
    - *j. Requirements for other fire protection equipment and systems,*

Please see the response in FDS1.

H.69 FDS69

69. *Fire and Leak Detection (Section 12.4) Areas, including enclosed buildings, that can have the presence of flammable gas, LNG or flammable refrigerant spills, and fire shall be monitored as required by the evaluation in Section 12.2.1.*

Please see the response in FDS1.

H.70 FDS70

70. *Gas Detection (Section 12.4.2.1,2) Continuously monitored low-temperature sensors or flammable gas detection systems shall sound an alarm at the plant site and at a constantly attended location if the plant site is not attended continuously. Flammable gas detection systems shall activate an audible and a visual alarm at not more than 25 percent of the lower flammable limit of the gas or vapor being monitored.*

    Please see the response in FDS1.

H.71 FDS71

71. *Fire Detection (Section 12.4.3.1,2,4) Fire detectors shall activate an alarm at the plant site and at a constantly attended location if the plant site is not attended continuously. If so, determined by an evaluation in accordance with 12.2.1, fire detectors shall be permitted to activate portions of the ESD system. The detection systems shall be designed, installed, and maintained in accordance with NFPA 72, National Fire Alarm and Signaling Code.*

    Please see the response in FDS1.

H.72 FDS72

72. *Fire Protection Water Systems. (Section 12.5) A water supply and a system for distributing and applying water shall be provided for protection of exposures; for cooling containers, equipment, and piping; and for controlling unignited leaks and spills, unless an evaluation in accordance with 12.2.1 determines that the use of water is unnecessary or impractical. The fire water supply and distribution systems, if provided, shall simultaneously supply water to fixed fire protection systems, including 9 monitor nozzles, at their design flow and pressure, involved in the maximum single incident expected in the plant plus an allowance of 1000 gpm (63 L/sec) for hand hose streams for at least 2 hours.*

    Please see the response in FDS1.

H.73 FDS73

73. *Fire Extinguishing and Other Fire Control Equipment. (Section 12.6) Portable or wheeled fire extinguishers shall be recommended for gas fires by their manufacturer. Portable or wheeled fire extinguishers shall be available at strategic locations, as determined in accordance with 12.2.1, within an LNG facility and on tank vehicles. Portable and wheeled fire extinguishers shall conform to the requirements of NFPA 10, Standard for Portable Fire Extinguishers. Handheld portable dry chemical extinguishers shall contain minimum nominal agent capacities of 20 lb. (9 kg) or greater and shall have a minimum 1 lb./sec (0.45 kg/sec) agent discharge rate. For facility hazard areas*

where minimal class "A" fire hazards are present, the selection of potassium bicarbonate–based dry chemical extinguishers is recommended. Wheeled portable dry chemical extinguishers shall contain minimum nominal agent capacities of 125 lb. (56.7 kg) or greater and shall have a minimum 2 lb./sec (0.90 kg/sec) agent discharge rate. If provided, automotive and trailer-mounted fire apparatus shall not be used for any other purpose. Fire trucks shall conform to NFPA 1901, Standard for Automotive Fire Apparatus. Automotive vehicles assigned to the plant shall be provided with a minimum of one portable dry chemical extinguisher having a capacity of not less than 18 lb. (8.2 kg).

Please see the response in FDS1.

H.74 FDS74

74. *Maintenance of Fire Protection Equipment. (Section 12.7) Facility operators shall prepare and implement a maintenance program for all plant fire protection equipment.*

Please see the response in FDS1.

H.75 FDS75

75. *Personnel Safety. (Section 12.8) Protective clothing that will provide protection against the effects of exposure to LNG shall be available and readily accessible at the facility. Employees who are involved in emergency response activities shall be equipped with protective clothing and equipment and trained in accordance with NFPA 600, Standard on Industrial Fire Brigades. Written practices and procedures shall be developed to protect employees from the hazards of entry into confined or hazardous spaces. At least three portable flammable gas indicators shall be readily available.*

Please see the response in FDS1.

H.76 FDS76

76. *Fire Protection (49 CFR 193.2611)*
    a. *Maintenance activities on fire control equipment must be scheduled so that a minimum of equipment is taken out of service at any one time and is returned to service in a reasonable period of*
    b. *Access routes for movement of fire control equipment within each LNG plant must be maintained to reasonably provide for use in all weather conditions.*

Please see the response in FDS1.

H.77 FDS77

77. *Protective enclosures (49 CFR 193.2905), The following facilities must be surrounded by a protective enclosure:*
    a. *Storage tanks*
    b. *Impounding systems*
    c. *Vapor barriers*
    d. *Cargo transfer systems*
    e. *Control rooms and stations*
    f. *Control systems*
    g. *Fire control equipment*
    h. *Security communications systems*
    i. *Alternative power sources*

    The protective enclosure may be one or more separate enclosures surrounding a single facility or multiple facilities. Ground elevations outside a protective enclosure must be graded in a manner that does not impair the effectiveness of the enclosure. Protective enclosures may not be located near features outside of the facility, such as trees, poles, or buildings, which could be used to breach the security. At least two accesses must be provided in each protective enclosure and be located to minimize the escape distance in the event of emergency. Each access must be locked unless it is continuously guarded. During normal operations, an access may be unlocked only by persons designated in writing by the operator. During an emergency, a means must be readily available to all facility personnel within the protective enclosure to open each access.

    Please see the response in FDS1.

H.78 FDS78

78. *The ESD System (NFPA 59A, Section 12.3)*
    a. *Each LNG facility shall have an ESD system(s) to isolate or shut off a source of LNG, flammable liquids, flammable refrigerant, or flammable gases, and shut down equipment whose continued operation could add to or sustain an emergency.*
    b. *Valves, control systems, and equipment required by the ESD system shall not be required to duplicate valves, control systems, and equipment installed to meet other requirements of the standard where multiple functions are incorporated in the valves, control systems, and equipment. The valves, control systems, and equipment shall meet the requirements for ESD systems.*
    c. *If equipment shutdown will introduce a hazard or result in mechanical damage to equipment, the shutdown of any equipment or its auxiliaries shall be omitted from the ESD system if the effects of the continued release of flammable or combustible fluids are controlled.*
    d. *The ESD system(s) shall be of a fail-safe design or shall be otherwise installed, located, or protected to minimize the possibility that it will become inoperative in the event of an emergency or a failure at the normal control system.*

e. *ESD systems that are not of a fail-safe design shall have all components that are located within 50 ft (15 m) of the equipment controlled in either of the following ways:*
    i. *Installed or located where they cannot be exposed to a fire*
    ii. *Protected against failure due to a fire exposure/heat for at least 10 minutes duration*
    iii. *Operating instructions identifying the location and operation of emergency controls shall be posted in the facility area.*
    iv. *Manual actuators shall be located in an area accessible in an emergency, shall be at least 50 ft (15 m) from the equipment they serve and shall be marked with their designated function.*

Please see the response in FDS1.

H.79 FDS79

79. *The ESD System shall be automatically activated when any of the following occur:*
    a. *The detection of an abnormal operating condition by pressure sensors in the inlet and outlet systems or in the process systems. The detection of fire on the terminal*
    b. *The detection of flammable gas concentration at 60% of the lower explosive limit*

Please see the response in FDS1.

H.80 FDS80

80. *ESD system components that are exposed to fire or cryogenic effects shall be evaluated to confirm that the actuators will not be impaired by the potential exposures thereby preventing the components to fail to a safe position.*

Please see the response in FDS1.

H.81 FDS81

81. *Verify that the following types of fires are addressed in the fire assessment plan:*
    a. *Rim seal fires: Rim seal fires frequently occur in tanks with a floating roof and can be quickly extinguished using stationary systems, provided they are promptly detected. If a fire persists longer, the seal may be damaged and cause an oil spill, posing the risk of an extensive fire. This damage or excess use of water may sink the floating roof and create a full-surface fire.*
    b. *Fires caused by vapors: Vapors may leak during the storage of petrochemical liquids and are at risk of catching fire (e.g., lightning).*
    c. *Embankment: Tank farms are usually encompassed by a sealed embankment, dike or bund area or stand inside a pond to contain leaking fluids. Leaks from valves and associated equipment can catch fire within the bund area. Likewise, liquids may catch fire if they unexpectedly leak from the tank.*

d. *Explosion: Since explosions can damage stationary extinguishing systems, mobile backup solutions should be incorporated in the fire protection plan.*
e. *Boilover: A boilover is a result of prolonged crude oil tank fires where trapped water quickly evaporates, resulting in a fireball.*
f. *Full-surface tank fire: In extreme cases, the floating roof can sink and catch fire, causing a full-surface tank fire to quickly develop, which requires fixed foam monitors and mobile solutions to extinguish.*
g. *The fire plan and specifics must comply with the Oregon State Fire Marshall regulations and NFPA 15 "Standard for Spray Fixed Systems for Fire Protection", 2017.*

Please see the response in FDS1.

H.82 FDS82

82. For all tanks, including but not limited to petroleum products, LNG, and firewater, check each connecting pipeline, stairways or other attachments for DE level seismic displacement. If seismic displacement as a result of required analyses exceeds the capacity of the connection/pipeline, then this condition must be addressed.

Please see the response in FDS1.

H.83 FDS83

83. For building or building-like structures, the fire protection system must satisfy local building codes (ASCE7, Section 1.3.7). All piping/tubing sections for fire suppression/sprinkler systems must satisfy the seismic relative displacements (ASCE7 Section 13.6.8). Support of the systems must conform to NFPA 13. If the structure is within the hazardous area and not pressurized, components must comply with intrinsically safe specifications (e.g. microwaves, heaters, etc.)

Please see the response in FDS1.

Appendix I

Control Systems

Appendix I. Form 9: Control Equipment OAR 340-300-0003 (CON)

I.1 CON1

1. *Verify that the anchorage for all control equipment meets the requirements of ASCE7, Section 13.4.*

The terminal has no centralized facility control system. All control equipment is operated manually, in its location, and only at the time it is needed. Most of the time control equipment sits idle and not in use. At this time, little information has been able to be gathered regarding the cataloging of all the control equipment present at the McCall Fuel Terminal. There is no existing database of the existing equipment. Additional data gathering is necessary to provide answers to this checklist.

I.2 CON2

2. *All cables connected to the control systems must facilitate all possible seismic displacement (ASCE7 Section 13.6.4).*

   Please see the response in CON1.

I.3 CON3

3. *Control panels and systems relays and other trip-sensitive equipment should be qualified to function during or after an earthquake.*

   Please see the response in CON1.

I.4 CON4

4. *Analyze control systems for susceptibility to impact or excessive displacements.*

   Please see the response in CON1.

I.5 CON5

5. *If control equipment is required to function during or after the SSE (DE), it should be inspected and any vulnerabilities assessed. Verify that there would be no pounding between adjacent control boxes and systems.*

   Please see the response in CON1.

I.6 CON6

6. *Within internal control systems, verify that the individual components are firmly anchored and will not displace during the earthquake.*

   Please see the response in CON1.

I.7 CON7

7. *Battery racks that support control equipment should be structurally sound and be able to resist transverse and lateral loads.*

   Please see the response in CON1.

I.8 CON8

8. *Control panels often contain components on rollers or slides. Verify that these have stops or restraints to remain in place during and after an earthquake.*

   Please see the response in CON1.

I.9 CON9

9. *Design control equipment should be designed for anchorage for resistance to the DE (SSE).*

   Please see the response in CON1.

I.10 CON10

10. *Batteries should be restrained from falling off racks and should have a spacer so that there is no sliding.*

    Please see the response in CON1.

I.11 CON11

11. Overhead equipment should be clear of batteries and control equipment to minimize the potential for falling damage.

Please see the response in CON1.

I.12 CON12

12. Control panels should be inspected for trip-sensitive devices, such as relays. Their functionality and requirements to operate during and after the SSE(DE) should be verified.

Please see the response in CON1.

I.13 CON13

13. Emergency Shutdown Systems shall comply with API RP 14C and Section 12.3 to shut down the flow of LNG to or from the tank and shut down equipment whose continued operation could add or prolong an emergency event. The system must be failsafe, and protected or located to prevent the possibility that it becomes inoperable in an emergency. If exposed to fire, control systems must be evaluated to remain operational.

Please see the response in CON1.

I.14 CON14

14. Critical supports/equipment within the cryogenically exposed areas shall be provided with cryogenic insulation and passive fire protection, sufficient for the incident duration (NFPA 59A, Section 10.6).

Please see the response in CON1.

I.15 CON15

15. If the shutdown system may create an additional hazard or mechanical problem, that portion of the system may be omitted from the automated ESD, but this should not affect the controlled shutdown of LNG or flammable fluids (CSA Z276-22, Section 12.2.2).

Please see the response in CON1.

I.16 CON16

16. *Buildings housing process or control equipment must have a protective enclosure enclosing all control equipment (NFPA 59A 12.9.3).*

    Please see the response in CON1.

I.17 CON17

17. *Operating instructions identifying the location of emergency controls shall be posted conspicuously (NFPA 59A 13.2.4).*

    Please see the response in CON1.

I.18 CON18

18. *From NFPA 59A, Section 9.4.2 – Valve controls under icing conditions must be able to maintain operability, storage, and vaporization facilities shall be designed so that, in the event that power or instrument air failure occurs, the system will proceed to a fail-safe condition that is maintained until the operators can act either to reactivate or to secure the system.*

    Please see the response in CON1.

I.19 CON19

19. *Each LNG facility shall have an ESD system(s) to isolate or shut off a source of LNG, flammable liquids, flammable refrigerant, or flammable gases, and to shut down equipment whose continued operation could add to or sustain an emergency.*

    Please see the response in CON1.

I.20 CON20

20. *Valves, control systems, and equipment required by the ESD system shall not be required to duplicate valves, control systems, and equipment installed to meet other requirements of the standard where multiple functions are incorporated in the valves, control systems, and equipment. The valves, control systems, and equipment shall meet the requirements for ESD systems.*

    Please see the response in CON1.

I.21 CON21

21. *If equipment shutdown will introduce a hazard or result in mechanical damage to equipment, the shutdown of any equipment or its auxiliaries shall be omitted from the ESD system if the effects of the continued release of flammable or combustible fluids are controlled.*

Please see the response in CON1.

I.22 CON22

22. *The ESD system(s) shall be of a fail-safe design or shall be otherwise installed, located, or protected to minimize the possibility that it will become inoperative in the event of an emergency or a failure at the normal control system.*

Please see the response in CON1.

I.23 CON23

23. *ESD systems that are not of a fail-safe design shall have all components that are located within 50 ft (15 m) of the equipment controlled in either of the following ways:*
    - *i. Installed or located where they cannot be exposed to a fire*
    - *ii. Protected against failure due to a fire exposure of at least 10 minutes*

Please see the response in CON1.

I.24 CON24

24. *Manual actuators shall be located in an area accessible in an emergency, shall be at least 50 ft (15 m) from the equipment they serve, and shall be marked with their designated function.*

Please see the response in CON1.

I.25 CON25

25. *At LNG facilities, there shall be a protective enclosure including a peripheral fence, building wall, or natural barrier enclosing major facility components, such as the following:*
    - *i. LNG storage containers*
    - *ii. Flammable refrigerant storage tanks*
    - *iii. Flammable liquid storage tanks*
    - *iv. Other hazardous materials storage areas*
    - *v. Outdoor process equipment areas*
    - *vi. Buildings housing process or control equipment*

vii. *Onshore loading and unloading facilities*

Please see the response in CON1.

I.26 CON26

26. *From Section 13.15 Container Instrumentation - Instrumentation for LNG facilities shall be designed so 4 that, in the event of power or instrument air failure, the system will go into a fail-safe condition that can be maintained until the operators can take action to reactivate or secure the system.*

Please see the response in CON1.

I.27 CON27

27. *From Section 13.18.4.5 Each facility operator shall ensure that a control system that is out of service for 30 days or more is tested prior to its return to service, to ensure that it is in proper working order.*
    i. *Each facility operator shall ensure that the inspections and tests in this section are carried out at the intervals specified.*
    ii. *Control systems that are used seasonally shall be inspected and tested before use each season. iii. Control systems that are used as part of the fire protection system at the facility shall be inspected and tested in accordance with the applicable fire codes and standards.*

Please see the response in CON1.

I.28 CON28

28. *From Section 14.8.10.4 - Control systems that are used as part of the fire protection system at the LNG plant shall be inspected and tested in accordance with the applicable local fire code and conform to four items listed with various NFPA references.*

Please see the response in CON1.

I.29 CON29

29. *From 49 CFR 193.2441 - the facility must have a control center from which operations and warning devices are monitored as required by this part. A control center must have the following capabilities and characteristics:*
    i. *It must be located apart or protected from other LNG facilities so that it is operational during a controllable emergency.*
    ii. *ii. Each remotely actuated control system and each automatic shutdown control system required by this part must be operable from the control center.*

iii. Each control center must have personnel in continuous attendance while any of the components under its control are in operation unless the control is being performed from another control center that has personnel in continuous attendance.
iv. If more than one control center is located at an LNG Plant, each control center must have more than one means of communication with each other center.
v. Each control center must have a means of communicating a warning of hazardous conditions to other locations within the plant frequented by personnel.

Please see the response in CON1.

I.30 CON30

30. From CFR 49 193.2445 Sources of power
i. Electrical control systems, means of communication, emergency lighting, and firefighting systems must have at least two sources of power which function so that failure of one source does not affect the capability of the other source.
ii. ii. Where auxiliary generators are used as a second source of electrical power, they must be located apart or protected from components so that they are not unusable during a controllable emergency and fuel supply must be protected from hazards.

Please see the response in CON1.

Appendix J

Report Limitations and Guidelines for Use

Appendix J

REPORT LIMITATIONS AND GUIDELINES FOR USE[2]

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 McCall Oil & Chemical Corporation. This report may be made available to prospective contractors for their bidding or estimating purposes, but our report, conclusions and interpretations should not be construed as a warranty of the subsurface conditions. This report is not intended for use by others, and the information contained herein is not applicable to other sites.

GeoEngineers, Inc. (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 McCall Terminal 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:

- The function of the proposed structure,

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[2] Developed based on material provided by GBA, Geoprofessional Business Association; www.geoprofessional.org.

- Elevation, configuration, location, orientation, or weight of the proposed structure,
- Composition of the design team, or
- Project ownership.

If important changes are made after the date of this report, GeoEngineers should be given the opportunity to review our interpretations and recommendations and 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.

Biological Pollutants

GeoEngineers’ Scope of Work specifically excludes the investigation, detection, prevention or assessment of the presence of Biological Pollutants. Accordingly, this report does not include any interpretations, recommendations, findings, or conclusions regarding the detecting, assessing, preventing or abating of Biological Pollutants and no conclusions or inferences should be drawn regarding Biological Pollutants, as they may relate to this project. The term “Biological Pollutants” includes, but is not limited to, molds, fungi, spores, bacteria, and viruses, and/or any of their byproducts.

If Client desires these specialized services, they should be obtained from a consultant who offers services in this specialized field.