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REPORT OF GEOTECHNICAL ENGINEERING SERVICES
PDX FUEL TANK DESIGN
PORTLAND INTERNATIONAL AIRPORT
PORTLAND, OREGON

by
Haley & Aldrich, Inc.
Portland, Oregon

for
Burns & McDonnell
Bloomington, Minnesota

File No. 0204679-001
December 2023

HALEY & ALDRICH, INC.
6420 S. Macadam Avenue
Suite 100
Portland, OR 97239-3517
503.620.7284

18 December 2023
File No. 0204679-001

Burns & McDonnell
8201 Norman Center Drive #500
Bloomington, Minnesota 55437

Attention: Reid Unke

Subject: Report of Geotechnical Engineering Services
PDX Fuel Tank Design
Portland International Airport
Portland, Oregon

Dear Reid Unke:

Enclosed is Haley & Aldrich, Inc.'s (Haley & Aldrich's) geotechnical engineering report for the proposed Portland International Airport (PDX) Fuel Tank Design (Project) in Portland, Oregon. The Project site is located within the property of PDX in Portland, Oregon, along the Columbia River.

We understand the Project includes the design and construction of a new truck offload facility, three new large fuel storage tanks, secondary tank containment walls, operations and fire protection buildings, pipelines and utility racks, and ancillary light poles. The project will also include demolition of existing fuel storage tanks and an existing operations building. The proposed improvements will interface with existing improvements, including a fuel pump and underground fuel piping.

This report contains the results of our research, explorations, and analyses, and provides recommendations for design and construction of the proposed Project. This report should be reviewed in conjunction with our Geotechnical Data Report (Haley & Aldrich, 2023b) and our Enhanced Seismic Design Considerations (Haley & Aldrich, 2023a) for the site.

The most significant geotechnical concerns regarding the proposed site development include the potential for very strong seismic shaking, seismic hazards including liquefaction and liquefaction-induced vertical settlements, and lateral spreading. These effects will cause instability of the nearby Columbia River riverbanks during an earthquake which can adversely affect the Project site. Ground improvement measures and/or deep foundations will be required to protect the proposed structures and other features which have seismic stability requirements.

We appreciate the opportunity to provide our services to you on this Project. If you have any questions, please call.

Sincerely yours,
HALEY & ALDRICH, INC.

Micah D. Hintz, P.E., G.E.
Technical Specialist, Geotechnical Engineer

Allison M. Pyrch, P.E., G.E.
Senior Associate, Geotechnical Engineer

Enclosures

c: JH Kelly, Attn.: Derek Koistinen

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HALEY ALDRICH

HALEY & ALDRICH, INC.
6420 S. MACADAM AVENUE
SUITE 100
PORTLAND, OR 97239-3517
503.620.7284

SIGNATURE PAGE FOR

REPORT ON
PDX FUEL TANK DESIGN
PORTLAND INTERNATIONAL AIRPORT
PORTLAND, OREGON

PREPARED FOR
BURNS & MCDONNELL
BLOOMINGTON, MINNESOTA

PREPARED BY:

Kayla Ahrens, P.E.
Technical Specialist, Geotechnical Engineer
Haley & Aldrich, Inc.

Micah D. Hintz, P.E., G.E.
Project Manager, Geotechnical Engineer
Haley & Aldrich, Inc.

REVIEWED AND APPROVED BY:

REGISTERED PROFESSIONAL ENGINEER
69452PE
Allison Pyrch
Digitally signed by Allison Pyrch
Date: 2023.12.18 16:30:30 -08'00'
OREGON
Nov. 14, 2006
ALLISON MARIE PYRCH
RENEWAL DATE: 12/31/24

Allison M. Pyrch, P.E., G.E.
Senior Associate, Geotechnical Engineer
Haley & Aldrich, Inc.

Daniel J. Trisler, P.E., G.E.
Principal, Geotechnical Engineer
Haley & Aldrich, Inc.

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1. Introduction

Haley & Aldrich, Inc. (Haley & Aldrich) is pleased to submit this report to Burns & McDonnell summarizing our geotechnical engineering services for the Portland International Airport (PDX) Fuel Project Tank Design (Project) at 4300 NE Marine Drive, located within the property of PDX in Portland, Oregon. We completed our work in general accordance with the scope of services included in the Incidental Service & Material Order issued by JH Kelly, executed 24 February 2023 (Agreement).

Burns & McDonnell plans on making facility improvements to the existing fueling facility located in the northwestern portion of the PDX property. We understand the proposed improvements project includes demolition of three existing above-ground fuel tanks (designated Tanks #1, #2, and #3) and associated piping and containment area walls as well as demolition of an existing operations building located east of the tanks. Proposed improvements include three new above-ground fuel tanks with dike walls, an operations building, a truck offload and Hazardous Cargo Transportation Security (HCTS) facility, and piping to connect the proposed fuel tanks to existing pipelines and facilities at the site. The proposed 110-foot-diameter fuel tanks (designated Tanks #5, #6, and #7) will be located to the south and east of the existing tanks. The proposed operations building will be roughly rectangular in shape with plan dimensions of about 50 by 70 feet. The proposed truck offload and HCTS facility will be rectangular in shape with a footprint area of about 75 by 85 feet. The project will also include construction of new pavements, above and below ground utilities (water, electricity, storm and sanitary sewer, etc.), and stormwater infiltration facilities.

Existing site structures and equipment pads to remain are understood to be supported on shallow mat foundations with bearing pressures on the order of 500 to 750 pounds per square foot (psf). These improvements include the following:

- Pump pad with footprint area of about 35 by 70 feet;
- Maintenance building with footprint area of about 20 by 25 feet;
- Testing lab with footprint area of about 10 by 30 feet;
- Generator pad with footprint area of about 20 by 30 feet;
- Power distribution center (PDC) with footprint area of about 30 by 45 feet; and
- Product tank with footprint area of about 10 by 35 feet.

Several Kinder Morgan-owned equipment pads and facilities are present at the site but are not within the scope of this project.

We understand the proposed improvements will be designed in accordance with the State of Oregon Department of Environmental Quality Fuel Tank Seismic Stability Rule 340-300, 2019 Oregon State Structural Code (OSSC), and American Society of Civil Engineers (ASCE) 7-16.

This report contains the results of our analyses and provides recommendations for design and construction of the proposed improvements. This report relies on the site data presented in the project Geotechnical Data Report, which includes detailed descriptions of the recent and historical field explorations, laboratory test results, and subsurface conditions (Haley & Aldrich, 2023b). This report also builds on the seismic evaluation presented in our report on Enhanced Seismic Design Considerations prepared for the project by Haley & Aldrich (2023a).

Figures are presented following the text. The location of the site is shown on Figure 1, Vicinity Map, and the site layout with recent and historical exploration locations is presented as Figure 2, Site Plan. A subsurface profile of the site is presented as Figure 3, Subsurface Cross Section A-A'. Pile design capacity plots are presented on Figures 4 through 6. Figures 7 and 8 present plots of estimated vertical and lateral displacements.

2. Scope of Services

The purpose of our services was to evaluate the subsurface conditions at the Project site and to provide geotechnical engineering recommendations for design and construction of the project elements. We completed the following tasks in general accordance with the Agreement:

- Performed a geotechnical exploration program at the site as presented and discussed in our Geotechnical Data Report (Haley & Aldrich, 2023b).
- Conducted engineering analysis to develop geotechnical design recommendations for seismic design criteria, foundations, excavations, and pavement design criteria.
- Prepared this report outlining our findings and recommendations, including information related to the following:
  - Subsurface soil and groundwater conditions;
  - Seismic hazards (e.g., liquefaction, settlement, and lateral spreading);
  - Site preparation and grading;
  - Utility trench construction;
  - Shallow and deep foundation design parameters;
  - Seismic design criteria; and
  - Slab and pavement design.
- Provided project management and support services, including coordinating staff and subcontractors and conducting telephone consultations and email communications with you and the design team, etc.

3. Subsurface Conditions

3.1 GENERAL

Subsurface soil and groundwater conditions interpreted from historical explorations and explorations advanced at the site as part of our current study, in conjunction with soil properties inferred from field and laboratory tests, formed the basis for the conclusions and recommendations in this report. Details of the explorations and laboratory testing completed at the project site are discussed in the site Geotechnical Data Report (Haley & Aldrich, 2023b). Our interpretations of the available subsurface data are provided in the following sections.

3.2 SOIL CONDITIONS

Generally, explorations encountered up to 7 to 16 feet of dredge sand fill overlying overbank deposits of Columbia River Sand Alluvium up to 50 feet below ground surface (bgs), which then overlies sand of the Columbia River Sand Aquifer to the base of the explorations. We divided the encountered soils into three engineering soil units (ESUs), which are grouped by similar geologic origin and/or engineering properties. Descriptions of these ESUs are provided below:

- ESU 1: Loose to Medium Dense Sand (Topsoil / Fill)
- ESU 2: Very Soft Silt (Overbank Deposits)
- ESU 3: Medium Dense to Dense Sand (Columbia River Sand)

These ESUs are discussed in detail in the following sections. A representative cross section is shown on Figure 3, Subsurface Cross Section A-A'.

3.2.1 ESU 1 – Loose to Medium Dense Sand (Topsoil / Fill)

This ESU consists of silty, poorly graded sand (SM to SP-SM) sand to a depth of approximately 7 to 16 feet bgs. The soils appeared to be brown, fine to medium grained, and poorly graded sand with a variable amount of silt. Based on observations taken during test pit excavation, fill sand generally has a loose to medium dense relative density. Groundwater table fluctuations are rarely expected to rise above the base of this ESU; however, this layer frequently appeared saturated in our explorations at depths greater than approximately 4 to 5 feet bgs, which we attribute to locally perched water conditions.

3.2.2 ESU 2 – Very Soft Silt (Overbank Deposits)

This ESU underlies the ESU 1 layer. This ESU consists of interbedded low plasticity clay (CL), silt (ML), and sandy silt to silty sand (SM) extending to depths varying between approximately 40 and 50 feet bgs. We performed soil index testing on undisturbed soil samples from this ESU taken between depths of 7.5 and 42 feet bgs. The plasticity index of samples within this ESU ranges from 0 to 52 with an average value of 17 and a standard deviation of 13. The water content (w_c) ranges from 33 to 83 percent with an average value of 51 percent and standard deviation of 12 percent. The liquid limit (LL) of the soil samples ranges from 0 to 103 (average value of 47) resulting in a w_c/LL value ranging from 0.8 to 1.2 (average value 1.0). According to the Bray and Sancio (2006) criteria, 64 percent of the tested soil samples are classified as susceptible to moderately susceptible to strength loss during cyclic loading and the other 36 percent of the tested soil samples are classified as non-susceptible. A minor amount of organic material was observed in these deposits, with organic content measured by loss on ignition ranging from 2 to 6 percent of the soil unit by mass.

No standard penetration test (SPT) blow counts (N-values) were measured in this ESU because the SPT sampler pushed into the soil due to the weight of hammer (equivalent to 0 blows per foot [bpf]). Field-collected pocket penetrometer readings ranged from 0 to 0.75 tons per square foot. This ESU is considered to be relatively weak and susceptible to liquefaction where saturated below the groundwater table.

3.2.3 ESU 3 – Medium Dense to Dense Sand (Columbia River Sand)

Underlying ESU 2, this ESU consists of fully saturated, poorly graded, micaceous, clean sand with traces of silt (SP to SP-SM) with fines contents ranging from 5 to 23 percent. SPT blow counts (N-values) in this ESU varied from 10 to 48 bpf indicating loose to dense, though typically medium dense to dense, relative density. Based on the normalized penetration resistance value (q_{c1N}) the clean-sand deposit is liquefiable (q_{c1N}<150) from a depth of 40 feet to a depth of at least 135 feet bgs (as observed at SCPT-5). We estimate the *in-situ* relative density (D_R) of this ESU ranges from 40 to 58 percent (loose to medium dense sand). For modeling purposes, we distinguish this ESU into three subgroups, namely ESU 3a (27 to 59 feet bgs), ESU 3b (59 to 97 feet bgs), and ESU 3c (greater than 97 feet bgs) to account for increasing relative density with depth. Each of the ESU 3 soils are considered to be relatively weak and susceptible to liquefaction and/or seismic strength loss.

This unit extends to depths of at least 150 feet bgs based on our exploration data, and likely terminates at a depth of approximately 180 feet bgs based on geophysical test results as described in the Geotechnical Data Report (Haley & Aldrich, 2023b). This unit is likely underlain by Troutdale Formation materials, followed by basalt bedrock at great depth.

3.2.4 Engineering Properties of ESUs

Estimated engineering soil properties for the three ESUs are provided in Table 1, Design Soil Profile, and Table 2, General Soil Properties. Table 1 indicates the name of the ESU, its general depth range, and representative $N1_{60}$ values, and Table 2 provides general engineering soil properties used in our analyses. Determination of these material properties were based on SPT relationships described in Bowles (1977) and our engineering judgement. Liquefied residual strength ratios and friction angles used for analysis of liquefiable conditions were generated from correlations by Robertson and Cabal (2010).

Table 1. Design Soil Profile

Soil Unit	Description	Typical Depths (Elevation[1]) (feet)	Average $N1_{60}$ (blows/foot)
ESU 1	Loose to Medium Dense Sand and Sandy Silt	0 to 16 (22 to 6)	15[2]
ESU 2	Very Soft Silt	16 to 27 (6 to –5)	0

Table 1. Design Soil Profile			
Soil Unit	**Description**	**Typical Depths (Elevation[1]) (feet)**	**Average $N1_{60}$ (blows/foot)**
ESU 3a	Medium Dense Interbedded Sand and Silt	27 to 59 (–5 to –37)	10
ESU 3b	Medium Dense Columbia River Sand	59 to 97 (–37 to –75)	21
ESU 3c	Medium Dense to Dense Columbia River Sand	97+ (-75 +)	26

Notes:

1. *The reference/assumed ground elevation at the site is 22 feet NAVD88.*
2. *SPT blow counts from this unit are not available due to "soft" digging during explorations. $N1_{60}$ is based on correlations with dynamic cone penetrometer tests.*

Table 2. General Soil Properties					
Parameter	**ESU 1**	**ESU 2**	**ESU 3a**	**ESU 3b**	**ESU 3c**
Total Unit Weight (pcf[a])	112	105	115	120	120
Friction Angle, ϕ' (degrees)	32	30	35	36	36
Liquefied Residual Shear Strength Ratio[1], s_r/σ'_{v0}	0.63	0.29	0.12	0.16	0.40
Liquefied Residual Friction Angle[1], ϕ'_r (degrees)	32	16	7	9	22

Notes:

1. *Liquefied Residual Shear Strength Ratio and Liquefied Residual Friction Angle values are provided for axial analyses and not intended for lateral pile analyses. Refer to Section 6.1.3 for lateral pile parameters.*

pcf = pounds per cubic foot

3.3 GROUNDWATER

Groundwater was encountered at depths ranging from approximately 5 to 14 feet bgs during our current and previous site explorations. Shallower measurements on the order of 5 to 7 feet bgs appear to represent a perched groundwater table within fill materials overlying the more fine-grained overbank deposits. Deeper groundwater level readings appear to be more indicative of the regional groundwater table. Cone penetration test pore pressure dissipation data collected during our current and previous site explorations indicate a regional groundwater level between approximately 10.5 to 14.5 feet bgs, as measured in June 2019 and February 2023. Historical groundwater elevations at the site reported by others were on the order of 8 to 10 feet mean sea level (MSL) in February 1999 (AGRA Earth & Environmental, Inc., 1999).

We anticipate that groundwater elevations will likely fluctuate over time based on the water level of the adjacent Columbia River. Fluctuations in groundwater levels may also occur due to variations in rainfall, temperature, seasons, and other factors. It is important that the contractor provide contingencies for addressing groundwater during construction on this project.

3.4 INFILTRATION TESTING

Three infiltration tests were conducted at the locations shown on Figure 2 labeled as IT-1, IT-2, and IT-3 in general accordance with the City of Portland's 2020 Stormwater Management Manual Section 2.3.2. Details surrounding the test procedure and collected infiltration data are presented in the Geotechnical Data Report (Haley & Aldrich, 2023b). Refer to Section 8.3, Infiltration Systems, of this report for a discussion of our findings and recommendations regarding the design of infiltration systems for this site.

4. Seismic Considerations

4.1 SEISMIC SETTING

Oregon is located near the contact between two large crustal tectonic plates. The Juan de Fuca Plate constitutes the floor of the Pacific Ocean off the northwestern coast of the United States and moves northeastward from its spreading ridge boundary with the Pacific Plate at an average rate of approximately 1.5 inches per year. As the Juan de Fuca Plate converges with continental North America, it subducts or dips below the North American Plate, forming a shallow, eastward-dipping contact interface. This boundary is referred to as the Cascadia Subduction Zone (CSZ) and is responsible for seismic activity in the western regions of Washington and Oregon. The CSZ gives rise to earthquakes associated with three types of source zones: subduction interface, subduction intraslab, and shallow crustal earthquakes.

The seismicity of the Pacific Northwest region is predominantly influenced by the CSZ. In this zone, the offshore Juan de Fuca Plate subducts beneath the continental North American Plate. Subduction zones typically exhibit three main types of earthquakes: crustal earthquakes, interface subduction earthquakes, and intraslab subduction earthquakes.

Intraslab and Interface Sources. A subduction zone is characterized by the interaction of a down-going oceanic plate, such as the Juan de Fuca Plate, and an overriding continental plate, such as the North American Plate. The displacement caused by the subduction of the Juan de Fuca Plate below the North American Plate does not generally manifest as slip between the two plates; rather, it is absorbed by compression of the North American Plate at the interface at relatively shallow depths. When the magnitude of the compression becomes large enough to overcome the stresses locking the plates together, the plates will suddenly rupture, causing an interface earthquake. Based on geologic and historical evidence, this compression is released about every 350 to 600 years on average in the form of magnitude 8.0 to 9.0 earthquakes.

The most recent CSZ interface event is thought to have occurred on 26 January 1700, based on paleoseismic evidence and historical records of an orphan tsunami along the Japanese coast (Atwater et al., 2005). Interface earthquakes (such as the 2011 magnitude 9.0 Tohoku earthquake in northern Japan) are some of the largest magnitude earthquakes on record. Characteristics of this type of earthquake may include very large ground accelerations, shaking durations in excess of 3 minutes, and particularly strong long-period ground motions, which may affect tall or long-period structures.

Intraslab earthquakes originate from a deeper zone of seismicity that is associated with bending and breaking of the subducting Juan de Fuca Plate. Intraslab earthquakes (such as the 2001 magnitude 7.0 Nisqually earthquake in west central Washington) occur at depths of 40 to 70 kilometers (130,000 to 230,000 feet) and can produce earthquakes with magnitudes greater than magnitude 7.0. Deep intraslab earthquakes tend to be felt over larger areas than shallower crustal events.

Crustal Sources. Shallow crustal faults are caused by cracking of the continental crust resulting from the stress that builds as the subduction zone plates remain locked together. Few surficial geologic traces exist of the shallow crustal faults in the Portland, Oregon area. The nearest series of known shallow crustal faults, including the Portland Hills Fault, East Bank Fault, Oatfield Fault, Lacamas Lake, and the Beaverton Fault Zone, have had their surface traces either eroded away or buried by ancient flood deposits, but have been mapped by seismic reflection and refraction studies and other geophysical methods. Therefore, less information is known about these faults than faults with distinct surface traces.

Crustal seismicity from known faults near the project site is generally dominated by the Portland Hills Fault, located approximately 6 miles from the project site. The Portland Hills, Oatfield, and East Bank faults run in a generally northwest-southeast direction through downtown Portland, and the Portland Hills Fault is generally believed to be capable of producing earthquake events with magnitude 7.0 or greater with a return period from 10,000 years to 20,000 years (Petersen et al., 2014). No estimates for the maximum expected earthquake magnitudes are available for the Beaverton Fault Zone and the Oatfield Fault (Peterson et. al., 2014); however, the East Bank Fault has a lower estimated slip rate and an expected maximum earthquake magnitude of 6.2. These faults and other crustal sources contribute significantly to the seismic hazard at all periods.

4.2 SEISMIC SITE CLASS

We determined the soil site class based on the foundation soil information following the guidelines of ASCE 7-16, as referenced by the current OSSC. The soil site class is determined by considering the soil characteristics and measured shear wave velocity data at the site up to a depth of 100 feet bgs. As presented in the Haley & Aldrich's Enhanced Seismic Design Considerations report (Haley & Aldrich, 2023a), the site is classified as seismic Site Class E, without accounting for the presence of liquefiable soils at the Project site. As a liquefaction hazard is determined to be present at the site, the site is classified as Site Class F and a site response analysis was completed.

4.3 DESIGN RESPONSE SPECTRA

A recommended surface response spectrum was developed based primarily on the results of the site response analysis, as discussed in Haley & Aldrich's Enhanced Seismic Design Considerations report (Haley & Aldrich, 2023a). The response spectrum is observed to be generally equal to or larger than the full ASCE 7-16 Chapter 21 code-based spectrum in the impulsive period range of interest. To facilitate design, the design earthquake (DE) spectrum is determined as 2/3 of the MCE_R spectrum. Tabular values for both the MCE_R and DE spectra are provided in Table 3. Additionally, the design acceleration parameters, S_{D1} and S_{DS}, are computed from the recommended design spectrum in accordance with Section 21.4 of ASCE 7-16. These design acceleration parameters are included in the notes section of Table 3. Refer to the Enhanced Seismic Design Considerations report for a full discussion of surface response spectrum development (Haley & Aldrich, 2023a).

Table 3. Recommended Surface Response Spectra

Period (seconds)	MCE_R (g)	DE (2/3 MCE_R) (g)
0.01	0.34	0.23
0.03	0.40	0.27
0.05	0.45	0.30
0.10	0.58	0.39
0.20	0.89	0.59
1.20	0.89	0.59
1.50	0.74	0.49

Table 3. Recommended Surface Response Spectra		
Period (seconds)	MCE_R (g)	DE (2/3 MCE_R) (g)
1.70	0.65	0.43
2.00	0.52	0.35
3.00	0.36	0.24
4.00	0.27	0.18
5.00	0.21	0.14
7.50	0.13	0.09
10.00	0.10	0.07
Note: $S_{DS} = 0.59g$, $S_{D1} = 0.74g$		

By utilizing the recommended design spectrum, along with the calculated design acceleration parameters, designers can appropriately incorporate the seismic loading considerations into the structural design process in accordance with ASCE 7-16 guidelines.

4.4 LIQUEFACTION

4.4.1 General

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

As presented in the Haley & Aldrich's Enhanced Seismic Design Considerations report (2023a), we performed simplified and more advanced two-dimensional (2D) numerical modeling to evaluate the liquefaction potential analysis of the site soils. Based on our analyses, we anticipate the saturated ESU 2 and ESU 3 soils are liquefiable to a depth of at least 150 feet bgs. The analyses estimate liquefaction-induced total vertical settlements at the site range from approximately 8 to 12 inches, with an average estimated total settlement of approximately 10 inches. An average estimated vertical settlement profile is presented as Figure 7, Estimated Vertical Seismic Settlement, and tabulated values for this profile are presented as Table 4, Tabulated Values for Estimated Liquefaction-Induced Vertical Settlement (attached). The recommended design liquefaction-induced differential settlement at the site can be taken as 5 inches over a distance of 50 feet, corresponding to about half of the estimated total seismic settlement.

4.4.2 Lateral Spreading

Lateral spreading commonly occurs on mildly sloping ground and involves lateral displacement caused by the accumulation of cyclic shear strain during earthquake. As the soil undergoes cyclic loading, excess pore pressure builds up, reducing the effective stress and gradually leading to a reduction in shear strength. This accumulation of shear strain ultimately results in permanent lateral deformation. Excessive lateral displacement resulting from lateral spreading can impact the fuel tank facility area by increasing the lateral force and displacement exerted on the tank foundation. Given the proximity of the project site to the Columbia River and the presence of liquefiable soil, we conducted an evaluation of the potential geotechnical impact on fuel tank facilities due to lateral deformation caused by liquefaction-induced lateral spreading during a design-level event. However, it is possible that the upslope geometry from the project site toward the levee may help reduce lateral displacement.

As presented in the Haley & Aldrich's Enhanced Seismic Design Considerations report (2023a), we used a free field 2D model to predict the behavior of the site under seismic loading. With respect to lateral deformation and spreading, seismic analyses performed on this model showed that the levee at the northern end of the PDX site experienced significant lateral deformation (more than 10 feet) due to high shear strain accumulation within the toe region. Considering a 2,475-year hazard level, the numerical model estimated that the existing fuel tank facilities area could experience lateral displacement, either toward the north or south direction, ranging from several inches to up to 6 feet, depending on the input ground motion selected; however, the average predicted lateral displacement using eleven input ground motions ranges from 18 inches at the northern end of the tank area to 32 inches at the southern end. Analyses indicate that the general trend for lateral spreading-induced movement is to the south in the direction away from the Columbia River, as topography at the site and in the surrounding area gently slopes towards the south.

4.4.3 Seismic Strength Loss

Our analyses, as presented in the Haley & Aldrich's Enhanced Seismic Design Considerations report (2023a), indicate site soils below the groundwater table will undergo liquefaction and cyclic softening and lose strength during the design level earthquake. This loss of strength was accounted for and factored into design parameters used in our global stability and foundation design analyses.

4.5 FAULT SURFACE RUPTURE

There are no mapped crustal faults are present at the site, with the closest known quaternary-age fault mapped approximately 5 miles to the southwest (Personius, 2019). Therefore, we consider the hazard from fault surface rupture at the site to be low, although unmapped or otherwise unknown faults may be present that could result in a higher hazard.

5. Discussion

Based on our review of subsurface information for the site, we have formulated geotechnical recommendations for use in project design. We offer the following general summary of our conclusions.

- Site soils generally consist of up to 7 feet of dredge sand fill overlying overbank deposits of Columbia River Sand Alluvium up to 50 feet bgs, which then overlies sand of the Columbia River Sand Aquifer to approximately 180 feet bgs. Shallow groundwater conditions are expected, with perched groundwater zones identified within the upper 5 feet bgs, and an estimated depth to the local groundwater table as shallow as about 10 to 12 feet bgs. Subsurface materials at the site to 180 feet are considered weak and susceptible to liquefaction and/or seismic strength loss where saturated.
- Near-surface soils may be prone to disturbance and loss of support under loading from heavy construction equipment such as pile drivers and cranes. Grading of working pads to support these loads should be expected. Wet soil grading methodologies may be appropriate for work during wet months.
- Due to presence of liquefaction and related hazards, there is a likelihood for excessive vertical and/or lateral movements of existing and proposed building foundations, utilities, and other site improvements. Proposed, critical, displacement-sensitive improvements should be designed to resist or account for these seismically induced ground movements. This could be achieved through support of proposed improvements on deep foundations designed to resist seismic loading. Shallow foundations may be considered for other cases. Ground improvement presents another viable alternative, though we understand the project team is not considering this approach at this time.
- Several existing structures on-site are supported on shallow foundations and may experience distress due to seismic ground deformations.
- Abrupt differential settlements may occur between improvements supported on deep foundations and those supported on shallow foundations and existing subgrade.

The remainder of this report presents our specific recommendations for foundations, pavements, drainage facilities, earthwork, and utilities.

6. Foundation Design Recommendations

This section of the report presents our conclusions and recommendations for the geotechnical aspects of design and construction of foundations for structures on the Project site. We have developed our recommendations based on our current understanding of the project and the subsurface conditions encountered by our explorations. We understand that the proposed fuel tanks, dike walls, operations building, truck offload and HCTS facility, and piping will all be above ground improvements supported on deep foundations. Non-critical improvements not designed per OSCC may potentially be supported on shallow foundations bearing on unimproved subgrade. Current designs do not include use of ground improvement for support of proposed improvements due to potential damage to existing facilities, though this approach is potentially feasible for this project.

If the nature or location of the facilities is different than we have assumed, we should be notified so we can change or confirm our recommendations.

6.1 DEEP FOUNDATIONS

6.1.1 General

Deep foundations are recommended for support of the proposed tank structures to mitigate the potential for large static total and differential settlements, to provide support for the proposed tanks under seismic shaking, and to supply resistance against seismic hazards including liquefaction-induced vertical settlements and lateral spreading.

In addition to conventional structure loads on the piles, additional soil-related loads will include seismically induced downdrag and lateral spreading forces. Deep foundations should be designed to resist a bearing capacity type failure while under downdrag caused by liquefaction-induced settlements. Lateral loads due to seismic lateral displacements of the ground, including the non-liquefied crust and deeper liquefied soils, will induce large moments and displacement in deep foundation elements. To support the structural and soil-related loads, we understand that pipe piles on the order of 18 inches in diameter are proposed.

Driven piles installed using vibratory methods or conventional drop hammers may induce development of elevated pore water pressures within the liquefiable soil layers at the site. This could lead to localized liquefaction occurring around each driven pile, resulting in significant ground settlements during construction. While the amount of settlement and the lateral distance from each pile to which the settlement will occur is not well understood, available analysis methods predict that settlements could be as high as several feet directly adjacent to the pile and may not taper out to less than 1 inch until a distance of several hundred feet from the pile is reached (Massarsch, 2004). These settlements may severely impact the performance of the existing buried pipelines and infrastructure at the site.

Where driving-induced settlements are a concern, piles may be installed using a torque-down method, which will greatly reduce the potential for pore water pressure buildup during installation.
Steel-encased torque-down piles perform similarly to conventional driven piles but are installed by screwing or torquing the pile into place using proprietary equipment, means, and methods. The piles are installed with a helical tip that allows the pile to advance through the subsurface through a combination of crowd pressure and torque. These piles are installed as full-displacement elements, similar to plugged

driven piles, causing soils surrounding the piles to densify during installation. Installation using this method also produces much less noise compared to conventional pile driving.

The recommendations provided in this section, including recommended pile capacities, apply to both conventional driven piles and those installed using a torque-down method.

6.1.2 Vertical Pile Resistance (Compressive and Uplift)

Vertical compressive loads to be supported by piles can be resisted by a combination of end bearing support at the tip (bottom) and side friction between the pile material and the soil along the axial length embedded into the bearing stratum. The ultimate uplift resistance of a pile is generally considered to be equal to the component of vertical resistance resulting from the friction of soil against the surface length embedded into the bearing stratum.

Our pile capacity analyses were conducted in general accordance with the methods contained in *Design and Construction of Driven Pile Foundations* (FHWA, 2016) using the computer program APile (ver. 2019.9.10) by Ensoft, Inc. We used the API RP2A method which is typically used for large diameter open-ended steel pipe piles bearing in cohesionless soil and relates soil density to a dimensionless bearing capacity factory, N_q.

The results of our vertical pile capacity analyses for 18-inch-diameter open-ended steel pipe piles are plotted on charts included on Figures 4 through 6. The charts show plots of ultimate resistance (capacity) versus length for various scenarios including static, liquefied, and post-liquefied conditions. These capacities are unfactored and appropriate resistance factors or factors of safety should be applied to the values. See Section 6.1.5.3, Quality Assurance/Quality Control (QA/QC) and Resistance Factors, for additional recommendations on this subject. We recommend that we work closely with the design team to discuss these factors depending upon the design methodology used.

For the liquefied condition, we recommend that bearing resistance be ignored within the upper 60 feet bgs to account for full liquefaction within the ESU 2 and 3a layers, as shown on Figure 5, 18-Inch-Diameter Pile Capacities Liquefied Condition. Portions of deep foundations embedded at depths greater than 60 feet bgs may gain support as shown on Figure 5, as soils at these depths are expected to undergo only partial liquefaction (see Enhanced Seismic Design Considerations report).

We recommend that design for the Extreme Event loading condition be performed considering the effects of post-seismic downdrag following the methodology presented for liquefaction-induced downdrag presented by Caltrans (2020), with minor modifications to account for increased settlement tolerances (between approximately 3 and 4 inches) for the proposed structures. The Caltrans methodology presents a settlement-based approach to determining minimum pile lengths to satisfy the axial bearing stability; however, it is acceptable to modify the analysis to account for increased settlement tolerances as applicable to the relevant site. The settlement profile presented on Figure 7, Estimated Vertical Seismic Settlement, should be used for determining ground settlements for this procedure. The strength of resettled liquefied soils may be taken as 50 percent of the static strength from 0 to 60 feet bgs. The full static strength should be used at depths greater than 60 feet bgs, based on the reduced Ru-max values in ESU 3b and 3c, indicating only partial liquefaction in these units (see Enhanced Seismic Design Considerations report). The axial loads corresponding to these recommendations are depicted graphically on Figure 6, 18-Inch-Diameter Pile Capacities Post-Liquefied Condition.

6.1.3 Lateral Deep Foundation Capacity

Lateral loads on the deep foundations are resisted primarily by the horizontal bearing support of near-surface soils adjacent to the pile. The lateral capacity of a pile depends on its length, stiffness in the direction of loading, and degree of fixity at the head, as well as on the engineering properties of the soils. The design lateral capacity of the deep foundations will depend largely on the allowable lateral deflection, shear, and moment of the shafts.

We performed analyses of the lateral capacity of the deep foundations completed using the 2D commercial code LPile 2019 by Ensoft, Inc. The LPile software computes deflection, shear, bending moment, and soil response due to lateral loads with respect to depth under several types of shaft-head loading conditions.

Table 5 shows recommended input parameters for performing LPile analyses for deep foundations.

Table 5. LPile Parameters for Analysis of Deep Foundations

Parameter	Engineering Soil Unit (ESU)				
	ESU 1	ESU 2	ESU 3a	ESU 3b	ESU 3c
Elevation (ft)	22 to 6	6 to –5	–5 to –37	–37 to –75	–75+
Static p-y Curve Type	API Sand	API Sand	API Sand	API Sand	API Sand
Static p-y Modulus, k (pci)	102	34.6	85.7	96.6	96.6
Static Friction Angle (°)	32	30	35	36	36
Liquefied p-y Curve Type	API Sand	Soft Clay (Matlock)	API Sand	API Sand	API Sand
Liquefied p-y Modulus, k (pci)	102	n/a	85.7	96.6	96.6
Liquefied p-multiplier[1]	--	--	0.10	0.60	0.72
Liquefied Friction Angle (°)	32	n/a	35	36	36
Undrained Shear Strength at Top of Layer (psf)	n/a	520	n/a	n/a	n/a
Undrained Shear Strength at Bottom of Layer (psf)		655			
Liquefied Strain Factor, ε_{50}	n/a	0.05	n/a	n/a	n/a
Effective Unit Weight (psf)	112 / 49.6[2]	42.6	52.6	57.6	57.6

Note:

1. *Lateral group p-multipliers should be combined with the liquefied p-multipliers as appropriate based on pile spacing.*
2. *Lower effective unit weight should be used for portions of ESU 1 beneath the design water table elevation of 10 feet.*

The proposed deep foundations supporting the structures and improvements will be subjected to large lateral displacements in the extreme limit state due to liquefaction-induced lateral spreading.
A relationship of average lateral displacements versus depth was developed as a result of 2D numerical model analyses, as summarized in our report on Enhanced Seismic Design Considerations (Haley &

Aldrich, 2023a). This lateral displacement will induce deflection and loading on the piles, which should be incorporated into lateral pile design. Recommended lateral displacement profiles for the northern, middle, and southern ends of the proposed facility are presented as Figure 8, Estimated Lateral Seismic Displacements.

6.1.4 Pile Corrosion Considerations

A suite of tests was completed as part of prior geotechnical work at the site (Hart Crowser, 2020) to evaluate the corrosion potential of the site soils. The results of the laboratory tests are provided within the Geotechnical Design Report (Haley & Aldrich, 2023b).

Based on the laboratory test results and comparisons to standards in the Ductile Iron Pipe Research Association (2006), we conclude that there is a low risk of corrosion to steel, iron, and concrete within the on-site fine- and coarse-grained soils. Additionally, the Soil Survey (USDA, 2023) indicates that there is a low risk of corrosion for both uncoated steel and concrete.

Based on the laboratory testing and guidance from the above documents, we consider it prudent to follow the guidelines set forth in Section 1.26.5 of Oregon Department of Transportation *Bridge Design Manual* (ODOT; 2020) for general corrosion protection measures due to the relatively low corrosive environment, which estimates an annual loss of steel of 0.001 inches/year. For open-ended piles, this loss of steel should be assumed to occur on the inside and outside of the pile (e.g., double the loss), whereas closed-ended piles only need to assume material loss on the pile exterior. ODOT (2020) indicates that corrosion can be controlled by relatively simple means, such as thicker walls or galvanizing steel. Use of Type I or II Portland cement for concrete is allowable.

6.1.5 Pile Installation Considerations

6.1.5.1 Driven Piles

We understand that driven piles, while feasible for use at this site, have the potential for inducing large settlements during pile driving, as described in Section 6.1.1. Should the design team elect to use driven piles, a pre-production indicator pile program with vibration and settlement monitoring should be formulated to establish action limits and strategies to mitigate excessive vibration and/or settlement.

Based on our explorations and experience with similar geologic units, excess pore pressure buildup is anticipated during pile installation using conventional driving. We recommend that after reaching the desired tip elevation under these conditions, piles should be allowed to “set up” for a minimum of 24 hours to allow for excess pore pressure dissipation. Following the set-up period, a restrike should occur and the restrike resistance should be verified in general accordance with Section 8.12.3 in the ODOT *Geotechnical Design Manual* (2023).

6.1.5.2 Torque-down Piles

Torque-down piles are typically installed using proprietary driving rigs capable of rotating closed-end pipe piles into place under variable crowd force. We recommend that the specialty contractor selected for this project provide a minimum of five references for previous installation of torque-down pile or equivalent type projects of similar length and diameter.

Deep foundations for this project are required to attain a minimum tip elevation to withstand seismic forces. Since torque-down piles are by necessity installed with closed-end tips without percussive force to hammer the piles into place, relatively dense soil conditions can cause torque-down piles to encounter refusal conditions; where driven open-ended piles would be able to advance. We recommend that the contractor independently assess the geotechnical data available for the site and attest to the capabilities of achieving minimum tip elevations using torque-down methods. Additionally, the contractor should propose a remedial action should an individual torque-down pile reach early refusal.

6.1.5.3 QA/QC and Resistance Factors

We recommend that a program of pile driving analysis be performed to verify the soil resistances and required depths of embedment, regardless of whether driven piles or torque-down piles are used. We recommend that load testing consist of static load testing for determination of compression and uplift resistances. Static load testing to determine lateral resistance may also be considered. Additionally, we recommend that at least 2 percent of production piles be tested using high-strain dynamic testing with signal matching (Pile Driving Analyzer® [PDA] and Case Pile Wave Analysis Program [CAPWAP]). Required driving resistances should include both anticipated structural loads and downdrag loads. Piles designed using this QA/QC approach may be designed using a Load and Resistance Factor Design resistance factor of as high as 0.80 for bearing resistance, and as high as 0.60 for uplift resistance, or an Allowable Stress Design factor of safety of 2.0 for uplift and compressive capacity.

6.2 SHALLOW FOUNDATIONS

6.2.1 General

Shallow foundations without ground improvement are generally not recommended for support of planned critical site improvements and those designed following OSSC, due to excessive predicted static and seismic settlements and lateral spread displacements. However, use of shallow foundations for support of improvements may be considered where the expected displacements have been accounted for in the facility design. Structures designed per OSSC must also meet the requirements of ASCE 7-16 Section 12.13. Shallow foundation systems should consist of elements structurally tied to each other, such as via a mat foundation system or via spread footings with grade beam ties.

Additionally, we understand that numerous existing, to-remain site improvements are supported on shallow foundations. Recommendations pertaining to new and existing shallow foundation supported structures and improvements are provided in this section.

6.2.2 New Shallow Foundations

Where shallow foundations are appropriate for support of select planned improvements, we recommend the following for design:

- Use a mat foundation or grid-style foundation to interlock all interior and perimeter footings. Use of isolated footings is not recommended.
- Design individual footings/strip footings for a maximum allowable bearing pressure of 1,500 psf. This represents a maximum pressure for any specific foundation element but does not consider consolidation settlements. The above bearing pressure values represent net bearing pressures; the weight of the footings and overlying backfill can be ignored in calculating footing sizes. The recommended allowable bearing pressures apply to the total of dead plus long-term live loads and may be increased by one-third for short term wind and seismic loads.

- Smaller structures (up to 75 kips total foundation load) supported on shallow foundations may experience about 2 inches of total static settlement due to consolidation of underlying compressible soils. This may result in about 1 inch of static differential settlement over a distance of 50 feet.
- Assume lateral ground displacement of 15 to 45 inches and vertical settlement of 8 to 12 inches as shown on Figures 7 and 8 for evaluation of ASCE-16, Section 12.13 criteria.

All shallow foundations should be underlain by 2 feet of medium dense granular material. This may consist of the on-site granular fill overlying the soft overbank deposits, imported structural fill, or a combination thereof.

6.2.2.1 Footing Dimension Recommendations

Structurally interconnected continuous wall footings and interior grade beams should be at least 18 and 12 inches wide, respectively. The bottom of exterior footings should be at least 18 inches below the adjacent finished exterior grade. Interior grade beams should be embedded at least 12 inches below the adjacent interior grade (e.g., base of slab).

6.2.2.2 Lateral Resistance

Lateral loads on footings can be resisted by passive earth pressures on the sides of footings and by friction on the bearing surface. We recommend that passive earth pressures be calculated using an equivalent fluid density of 250 pcf. We recommend using an ultimate friction coefficient of 0.30 for foundations placed on compacted, existing silty sand fill or 0.45 for foundations placed on an aggregate base subgrade. The passive earth pressure and friction components may be combined provided that the passive component does not exceed two-thirds of the total. The lateral resistance values do not include safety factors.

6.2.2.3 Uplift Resistance

Uplift forces on shallow foundations may be resisted using a combination of two methods. The first is to use the weight of the foundation element itself to resist uplift forces. Additionally, the weight of the overlying backfill soils may be used to assist in uplift resistance. Overlying soil weight may be calculated as a prism overlying the footing defined by a plane acting at 20 degrees from vertical extending from the upper perimeter of the foundation element. Assume the unit weight of the overlying soil is approximately 110 pcf.

6.2.2.4 Construction Considerations

Prior to the placement of reinforcing steel in the footing excavations, all loose or disturbed soils should be removed. If water infiltrates and pools in the excavation, the water, along with any disturbed soil, should be removed before placing the reinforcing steel. If construction is undertaken during periods of rain, we recommend that a concrete “rat slab” or “mud slab” or imported granular material be placed over the bases of footing excavations. The protective layer reduces subgrade disturbance from standing water and from foot traffic during forming and tying of reinforcing steel. Typically, 3 to 4 inches of concrete or granular material that is lightly compacted until well-keyed provides sufficient protection from disturbance.

6.2.3 Existing Shallow Foundations

Several existing site structures and equipment pads are reportedly supported on shallow mat foundations with bearing pressures on the order of 500 to 750 psf. These improvements include the following:

- Pump pad with footprint area of about 35 by 70 feet;
- Maintenance building with footprint area of about 20 by 25 feet;
- Testing lab with footprint area of about 10 by 30 feet;
- Generator pad with footprint area of about 20 by 30 feet;
- PDC with footprint area of about 30 by 45 feet; and
- Product tank with footprint area of about 10 by 35 feet.

These existing improvements are expected to experience large total vertical settlements averaging 8 to 12 inches resulting from liquefaction trigged by the design seismic event, with estimated vertical differential settlements averaging 5 inches over a distance of 50 feet. However, differential settlement equal to the total settlement (8 to 12 inches) should be anticipated between existing systems and proposed improvements supported on deep foundations. Seismic-induced lateral displacements averaging 15 to 45 inches are also anticipated and will result in lateral separation to varying degrees between existing and proposed improvements. These displacements should be considered in design.

6.3 BUILDING SLABS

We understand that nearly all new building slabs will be supported on deep foundations as opposed to slabs-on-grade. One slab adjacent to an existing hydrant pump pad will be supported on-grade. Building slabs, whether pile-supported on supported on-grade, may be constructed over new structural fills or native subgrade prepared in accordance with Section 9.0, Earthwork Recommendations, including reworking of the loose/soft surficial soils.

A minimum 6-inch-thick layer of aggregate base should be placed over the prepared subgrade to assist as a capillary break. Aggregate base material placed directly below the slab should be 3/4- to 1-inch maximum size. Flooring manufacturers often require vapor barriers to protect flooring and flooring adhesives. Many flooring manufacturers will warrant their product only if a vapor barrier is installed according to their recommendations. Selection and design of an appropriate vapor barrier, if needed, should be based on discussions among members of the design team.

6.3.1 Subdrainage Considerations

We generally recommend that slab-on-grade buildings with moisture sensitive interiors be constructed with a perimeter drain system to reduce the risk of future slab or below-grade wall moisture problems. Such intrusion may occur due to water perching in the near surface sands over the fine-grained overbank deposits or if the ground surface is not properly draining away from the building (e.g., trapped planters are present). Given the relatively flat grade and the presence of perched groundwater relatively close to ground surface in our explorations, the risk of water or moisture intrusion inside the building envelope is considered low to moderate. Provided that the surrounding ground surface is properly sloped away from the building (e.g., no trapped planters), a perimeter drainage system may be considered prudent but not required. However, the final decision whether to include a perimeter building drainage system should be based on what level of risk the owner and building designer is willing to accept.

Installation of perimeter drain system should consist of a minimum 2-foot-deep by 1-foot-wide trench filled with drainage material with a 4-inch-diameter perforated collection pipe. The drainage material should consist of a free draining, well-graded sand and gravel, such as ODOT Standard Specifications for Construction (OSS) Gravel Granular Drainage Blanket, Section 00360.11, with the additional criteria of containing less than 3 percentffines. All drainage pipes should be installed at least 1 foot beneath the adjacent floor slab subgrade and be sloped to drain away from the footings and be hydraulically connected to a suitable discharge outlet point. Cleanouts should be installed for maintenance purposes.

Roof and surface water runoff should not discharge into the perimeter drain system. Rather, these sources should discharge into separate tightline pipes and be routed away from the building to a storm drain or other appropriate location.

6.4 LIGHT POLE STRUCTURE FOUNDATIONS

We understand that luminaire structures will be constructed at various points around the site. We anticipate that these structures will typically be supported by square or round shafts that vary in diameter from 2.5 to 3 feet and in length from 6 to 11.5 feet, depending upon the configuration of the structure and soil strength. Design of luminaires is commonly controlled not by vertical capacity but by overturning and/or rotation of the pole. Due to the relatively short and wide nature of luminaire foundations, LPile analysis is generally not considered a valid method for design of luminaire foundations. Design of this type of foundation is more commonly performed using Broms method as presented in Design of Structural Supports for Highway Signs, Luminaires, and Traffic Signals (American Association for State Highway and Transportation Officials [AASHTO], 2011).

We understand that luminaire structures are not being designed to withstand the design seismic event. Recommendations provided in this section address static conditions only.

6.4.1 Lateral Capacity (Broms Method) Recommendations

The Broms Method outlined in AASHTO (2011) may be followed using the parameters listed in Table 6.

Table 6. Broms Method Soil Parameters

Parameter	Value
Effective Soil Unit Weight (γ') (above groundwater/below groundwater)	112 pcf/49.6 pcf
Soil Friction Angle (ϕ)	32
Lateral Bearing Coefficient (K_p)	3.3

Notes:

The values presented assume that the ground around the foundation is generally level (3H:1V or flatter)

Assume groundwater at a depth of 10 feet bgs.

A soil-to-foundation contact friction angle of 25 degrees may be used for torsional analysis.

6.4.2 Shaft Foundations

Shaft foundations may be designed using either a friction or end-bearing approach.

- **Friction-Based Design:** Vertical capacity may be derived by applying an allowable 250 psf skin friction in both the surficial sand and underlying overbank deposits for both compressive and uplift forces. Friction-based design should not be used for shaft foundations constructed using corrugated metal pipe or Sonotube forms below grade.
- **End-Bearing Based Design:** Vertical capacity may be derived using an end-bearing based approach using an allowable end bearing stress of 6 kips per square foot. If this approach is selected, shafts should be overdrilled a depth of 2 feet and 2 feet of stabilization material as defined in Section 9.5.4, Stabilization Material, should be placed at the base of the excavation prior to concrete placement.

6.4.3 Spread Footings

Alternatively, luminaires may be founded on shallow spread footings. Design for shallow foundations for luminaires should be performed using design parameters presented in Section 6.2, Shallow Foundations, of this report.

6.4.4 Construction Considerations

The bottoms of the drilled pier holes for luminaires should be free of debris and water before placing concrete. Concrete should be placed the same day the holes are drilled to limit relaxation of the supporting soil. As an alternative approach, concrete can be placed using tremie methods if water is present in the pier holes.

The sand layers at the site could cave during drilling. The contractor should plan for this condition and select the appropriate means and methods of drilling the pier holes and placing the reinforcing steel and concrete.

7. Pavement Design Recommendations

Paving for this project includes new flexible asphaltic concrete (AC) to be used for drive aisles and parking areas around the facility. Our design thicknesses assume that new pavements will be founded on the *in-situ* silty sand subgrade.

7.1 ASSUMPTIONS AND DESIGN PARAMETERS

We made the following assumptions regarding, and used the following parameters for, the design of the pavement.

- A 20-year design life of approximately 60,000 equivalent single-axle loads (ESALs). (These ESALs were calculated using ESAL factors provided in ODOT's Pavement Design Guide (ODOT, 2019) and traffic count values provided by Burns & McDonnell, as follows:
  - Tanker Truck (14k Gallons) – 2 per week
  - Ford F700 – 8 per day
  - Isuzu NPR – 8 per day
  - Ford F450 – 10 per day
  - Ford F150 – 15 per day
  - Utility Van – 15 per day
  - Vacuum Truck (5k Gallons) – 1 per week
  - Employee Vehicle – 20 per day
- A subgrade modulus of 10,000 pounds per square inch (psi) was assumed for a compacted *in-situ* fill subgrade.
- A resilient modulus of 20,000 psi was estimated for the base rock.
- Reliability and standard deviation of 85 and 0.45, respectively.
- Initial and terminal serviceability indices of 4.2 and 2.5, respectively.
- Structural coefficients of 0.42 and 0.10 for new asphalt and aggregate base layers, respectively.

7.2 PAVEMENT SECTION

Using the parameters provided above, we analyzed various pavement sections, including pavement constructed on *in-situ* fill material. If these or other assumptions are inaccurate, we should be contacted to develop updated recommendations. The recommended pavement section is provided in Table 7.

Table 7. Recommended Pavement Sections

Pavement Type	Pavement Thickness (inches)	Aggregate Base Thickness (inches)
Flexible Asphaltic Concrete	4	6
Aggregate Roadway	0	21

Notes:
1. *The aggregate base should be underlain by a separation geotextile unless pre-existing aggregate is exposed in the subgrade.*
2. *These values represent the minimum recommended material thicknesses.*

7.3 PAVEMENT MATERIALS

7.3.1 Flexible Asphaltic Concrete

The AC should be Level 2, 12.5-millimeter, dense hot mix asphalt concrete according to OSS 00744, Minor Hot Mixed Asphalt Concrete Pavement.

The asphalt cement binder should be PG 64-22 Performance Grade Asphalt Cement. The minimum AC lift thickness for the base lift should be 2 inches. Subsequent lifts should be a minimum of 1.5 inches thick. The AC should be compacted to 91 percent of Rice Density of the mix, as determined in accordance with ASTM International (ASTM) D 2041.

7.3.2 Aggregate Base

Imported granular material used as base aggregate (base rock) should meet the criteria specified in Section 9.5, Structural Fill and Backfill, of this report. The base aggregate should be compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D 1557. We recommend placement of a geotextile separation fabric beneath the aggregate base if the base is placed on existing sandy fill soils (as opposed to existing base rock). The geotextile should meet the specifications provided in OSS 02320.20, Geotextile Property Values, for soil separation. The geotextile should be installed in conformance with the specifications provided in OSS 00350, Geosynthetic Installation.

If the existing base rock that blankets the site is documented to be free of debris and other deleterious materials and is of sufficient thickness after site grading (at least 8 inches), the existing rock may be used to support new pavement. If sufficient rock thickness is not present, and if grades allow, additional rock can be placed atop the existing rock, otherwise the existing rock will need to be removed and new rock placed.

7.3.3 Subgrade

The pavement design assumes the soil subgrade consists of well compacted subgrade that has been stripped of organics. The subgrade should be compacted to a firm and unyielding condition and evaluated per the recommendations of Section 9, Earthwork Recommendations.

8. Drainage and Infiltration

8.1 TEMPORARY DRAINAGE

During demolition, stripping and mass grading at the site, the contractor should be made responsible for temporary drainage of surface water as necessary to prevent standing water and/or erosion at the working surface. During rough and finished grading of the building site, the contractor should keep all footing excavations, building pads, tank pads, and other subgrades free of water.

8.2 SURFACE DRAINAGE

The finished ground surface around buildings and tanks should be sloped away from the foundations at a minimum 2 percent gradient for a distance of at least 5 feet. Downspouts or roof scuppers should discharge into a storm drain system that carries the collected water to an appropriate stormwater system.

8.3 INFILTRATION SYSTEMS

The results of our field infiltration testing are described in the Geotechnical Data Report (Haley & Aldrich, 2023b). Infiltration rates in the near surface fill soils were found to be highly variable with silty fill materials having an infiltration rate of less than ¼ inches per hour, while sandy fill materials had an unfactored infiltration rate of approximately 10 to 20 inches per hour.

Perched groundwater was encountered in our explorations at depths as shallow as 5 feet bgs, corresponding to elevations of about 14.5 to 16.2 feet (North American Vertical Datum [NAVD] 88). These higher-elevation groundwater measurements likely represent zones of perched water, with the regional groundwater table at depths of approximately 10 to 12 feet bgs. Perched and regional groundwater could have a significant effect on design of stormwater disposal systems. Per the City of Portland 2020 Stormwater Management Manual, new surface infiltration facilities are required to have a minimum separation distance of 5 feet between the bottom of the facility and the seasonal high groundwater level unless otherwise approved by the City Bureau of Environmental Services.

Should surface infiltration features be permitted for use at the site, we recommend using a design unfactored infiltration rate of no greater than 10 inches per hour assuming that infiltration features encounter sandy fill materials. Due to extreme variability of infiltration rates in the fill, we recommend that supplemental field exploration be completed prior to completion of design to confirm soil conditions at the proposed locations of new infiltration systems, or that early in construction the soil conditions at proposed infiltration feature locations be assessed and that confirmation infiltration testing be conducted in the field during construction.

9. Earthwork Recommendations

We understand that mass grading across the site will generally be limited, with localized fill up to approximately 2 feet thick. All earthwork activities should be conducted in accordance with the OSS, particularly OSS 00330, Earthwork; OSS 00400,Drainage and Sewers; and OSS 02600, Aggregates, depending upon the application (ODOT, 2018).

9.1 DEMOLITION

Demolition should include complete removal of existing site improvements within areas to receive new pavements, structures, or engineered fill. Materials generated during demolition of existing improvements should be transported off-site for disposal or stockpiled in areas designated by the owner. In general, these materials will not be suitable for reuse as engineered fill. However, concrete, embankment fill, and base rock materials may be crushed and recycled for use as general fill. Such recycled materials should meet the specifications for imported granular material, as described in Section 9.5, Structural Fill and Backfill.

9.2 EXCAVATION

9.2.1 General

Excavations, shoring, and dewatering should be completed in accordance with the specifications provided in OSS 00330, Earthwork and OSS 00405, Trench Excavation, Bedding, and Backfill; the guidelines provided in Section 15.3.26, Temporary Shoring and Cut Slopes of the ODOT Geotechnical Design Manual (ODOT 2018); the City of Portland Standard Construction Specifications (Portland, 2020); and the City of Portland Stormwater Management Manual (Portland, 2020).

Site soils within expected excavation depths generally consist of sandy fill materials and silty alluvial materials. It is our opinion that conventional earthmoving equipment in proper working condition should be capable of making necessary general excavations. However, caving and/or sloughing conditions are likely to be present in the loose sands and soft silts.

The earthwork contractor should be responsible for providing equipment and following procedures as needed to excavate the site soils, as described in this report, while protecting the subgrade.

9.2.2 Temporary Open Cuts

Temporary soil cuts for site excavations that are more than 4 feet deep should be adequately sloped back to prevent sloughing and collapse, in accordance with Occupational Safety and Health Administration (OSHA) guidelines.

The stability and safety of cut slopes depend on a number of factors, including:

- Type and density of the soil;
- Presence and amount of groundwater seepage;
- Depth of cut;

- Proximity and magnitude of the cut to any surcharge loads, such as stockpiled material, traffic loads, or structures;
- Duration of the open excavation; and
- Care and methods used by the contractor.

Because of the variables involved, actual slope angles required for stability in temporary cut areas can only be estimated before construction. It is the responsibility of the contractor to ensure that the excavation is properly sloped or braced for worker protection, in accordance with OSHA guidelines. Most of the near-surface site soils generally consist of loose fill and very soft alluvial soils that would be classified as OSHA Class C for excavation purposes.

If site constraints do not allow proper excavation slopes for trenching, shoring may be used to support trench excavations. Shoring selection and design should be the responsibility of the contractor. If shored excavations are left open for extended periods of time, caving of the sidewalls may occur between the cut and shoring if voids between the shoring and cut are not filled. The presence of caved material will limit the ability to properly backfill cuts. The voids between box shoring and the sidewalls of cuts should be properly filled with sand or gravel before caving occurs. It should be the contractor's responsibility to employ trenching, excavation, and shoring methods that ensure proper compaction will be achieved and protect adjacent facilities.

9.3 SUBGRADE PREPARATION

The site should be rough graded to accommodate the proposed grading plan. In non-foundation areas that will receive new fills, building loads, and site improvements, such as pavements, sidewalks, and slabs, the exposed soil subgrade should be prepared by scarifying to a depth of at least 8 inches, moisture-conditioning to the optimum moisture content, and compacting to at least 90 percent relative compaction. In proposed building areas, subgrade preparation should extend at least 5 feet beyond the limits of the proposed building slabs and any adjoining flatwork. In exterior concrete slab and pavement areas, subgrade preparation should extend at least 2 feet beyond the limits of these improvements.

The near surface soils primarily consist of sands but include fine-grained layers that may be susceptible to disturbance during periods of wet weather. We recommend wet soil construction practices be implemented throughout construction. Wet soil construction practices include using equipment, such as smooth excavator buckets and tracked equipment, to limit subgrade disturbance. During wet weather or when the exposed subgrade is wet, the prepared subgrade should be evaluated by observing excavation activity and probing with a steel foundation probe. Observations and probing should be performed by Haley & Aldrich representatives.

Existing near-surface soils are not expected to provide a suitable working surface for heavy construction equipment including cranes and pile driving rigs, meaning that ground improvement or grading of a working platform will be required. Design of the working platform should be the responsibility of the contractor.

Outside of crane and pile driver working pad areas, the contractor may consider the use of granular haul roads and staging areas to reduce subgrade disturbance. Based on our experience, between 12 and 18 inches of imported granular material is generally required to provide stable staging and haul roads areas. However, the actual thickness will depend on the contractor's means and methods, and accordingly, should be the contractor's responsibility. Additionally, a geotextile separation fabric should be placed as a barrier between the subgrade and imported granular material in areas of repeated construction traffic. The granular material and geotextile separation fabric materials should conform to specifications provided in Section 9.5.4, Stabilization Material, of this report. If stabilized haul roads or staging areas are constructed, the contractor should verify if site restoration requirements require such features to be removed at the end of construction.

9.4 DEWATERING

Dewatering systems should be designed for the highest anticipated groundwater elevation during the construction period. Perched groundwater may be present as shallow as 5 feet bgs seasonally based on our explorations. These perched groundwater zones may produce a significant volume and flow rate of water into temporary excavations.

Construction of utilities that extend below groundwater will require dewatering or water control systems. Groundwater seepage rates into excavations may vary across the site, with high flow rates possible in areas with more granular soil layers. Pumping from sumps may only be effective in removing water from localized sections of trenches and open excavations. If excavations extend more than a few feet below a groundwater level or expose large areas below the groundwater, then large volumes, possibly combined with relatively high flow rates of water should be expected, and the use of well points or a robust series of collection trenches and sumps may be required. (This is particularly true for excavations that extend into the regional water table, generally expected to be found at a depth of 10 feet below the existing ground surface).

We note that dewatering of excavations with sump pumps will not prevent or reduce the greater risk of trench wall caving, sloughing, or basal instability caused by seepage.

Excavation or hauling equipment should not track below the groundwater table without dewatering systems in place. Also, fills, topsoil, etc. should not be placed in ponded water. Therefore, dewatering points or trenches may be required to prevent water from ponding in excavations during construction. The contractor should be made responsible for temporary drainage of surface water and groundwater as necessary to prevent standing water and/or erosion at the working surface or in excavations.

9.5 STRUCTURAL FILL AND BACKFILL

Structural fill should include fill intended to support structures, such as buried structures or new buildings, or which exist within the influence zone of structures. Structural fill should only be placed over a subgrade that has been prepared in conformance with the prior sections of this report. A variety of material may be used as structural fill. However, all material used as structural fill should be free of debris, clay balls, roots, organic matter, frozen soil, man-made contaminants, particles with greatest dimension exceeding 4 inches, and other deleterious materials and should meet the appropriate specification provided in OSS 00330.12, Borrow Material; 00330.13, Selected General Backfill; or 00330.14, Selected Granular Backfill.

Fill and backfill materials should be placed and compacted in lifts with maximum uncompacted thicknesses and relative densities as recommended in the table in Section 9.6, Fill Placement and Compaction of this report.

9.5.1 On-Site Soils

On-site soils encountered at shallow depths in our explorations consist of dredge sand with variable silt content used for the original construction of the airport. This fill material can be used as borrow material, provided it is properly moisture conditioned, free of organics, and has oversize materials removed. If earthwork is completed during periods of wet weather, then the excavated soil intended for reuse should be protected with plastic sheeting or other methods employed to maintain suitable moisture content. Even with these measures, such soils may be difficult or impossible to use during wet weather of it is wet at the time of placement.

Below the fill materials, native soils are fine-grained, very soft and wet. These fine-grained soils will not be suitable for reuse as fill.

9.5.2 Imported Select Structural Fill

Imported granular material used as structural fill should be pit or quarry run rock, crushed rock, or crushed gravel and sand and should meet the specifications provided in OSS 00330.14, Selected Granular Backfill; 00330.15, Selected Stone Backfill; or 00330.16, Selected Stone Embankment. The imported granular material should also be angular, fairly well graded between coarse and fine material, have less than 5 percent by dry weight passing the United States (U.S.) Standard Number (No.) 200 Sieve, and have at least two mechanically fractured faces.

9.5.3 Aggregate Base/Base Rock

Imported granular material used as aggregate base (base rock) beneath pavements or structures should be clean, crushed rock or crushed gravel and sand that is fairly well graded between coarse and fine. The base aggregate should meet the specifications provided in OSS 02630.10, Dense Graded Base Aggregate, depending upon application, with the exception that the aggregate has less than 5 percent by dry weight passing a U.S. Standard No. 200 Sieve, and at least two mechanically fractured faces. For use beneath abutment wall footings, the aggregate base should have a maximum particle size of 1.5 inches, while for use beneath pavements and sidewalks or other slabs (if needed) should have a maximum particle size of 1 inch.

9.5.4 Stabilization Material

If imported granular material is used to create haul roads for construction traffic or is required for stabilization of the bases of excavations, we recommend that material consist of pit or quarry run rock or crushed rock. The material should generally be sized between 2 and 6 inches, have less than 5 percent by dry weight passing the U.S. Standard No. 4 Sieve, and have at least two mechanically fractured faces. The material should be free of organic matter and other deleterious material. Material meeting the specifications of OSS 00330.15 - Stone Backfill Material is acceptable for use, excepting recycled glass shall not be used.

A geotextile should be placed as a barrier between the native soil subgrade and the stabilization material. The stabilization material should be placed in conformance with the specifications provided in OSS 00331, Subgrade Stabilization. The geotextile should meet the specifications provided in OSS 02320.20, Geotextile Property Values for soil separation. The geotextile should be installed in conformance with the specifications provided in OSS 00350, Geosynthetic Installation.

Stabilization material should be placed in lifts between 12 and 18 inches thick and be compacted to a well-keyed condition with appropriate compaction equipment without using vibratory action.

9.5.5 Drain Rock

Drain rock used for subsurface drainage systems should meet the specifications provided in OSS 0430.11, Granular Drain Backfill Material. The drain rock should be wrapped in a geotextile fabric that meets the specifications provided in OSS 02320, Geosynthetics for Drainage Geotextiles.

9.6 FILL PLACEMENT AND COMPACTION

Structural fill should be placed and compacted in accordance with OSS 00330.43, Earthwork Compaction requirements and the following guidelines.

- Place fill and backfill on a prepared subgrade that consists of firm, inorganic on-site soils or approved structural fill.
- Place fill or backfill in uniform horizontal lifts with a thickness appropriate for the material type and compaction equipment. Table 8 provides general guidance for uncompacted lift thicknesses.
- Do not place fill and backfill until the required tests and evaluation of the underlying materials have been made and the appropriate approvals have been obtained.
- Limit the maximum particle size within the fill to two-thirds of the loose lift thickness.
- Control the moisture content of the fill to within 3 percent of the optimum moisture content based on laboratory modified Proctor tests. The optimum moisture content corresponds to the maximum attainable modified Proctor dry density.

During structural fill placement and compaction, a sufficient number of in-place density tests should be completed by Haley & Aldrich to verify that the specified degree of compaction is being achieved. For structural fill with more than 30 percent retained on the 3/4-inch sieve, proper compaction should be verified with a proof roll or other performance methods.

Table 8. Guidelines for Uncompacted Lift Thickness

Compaction Equipment	Native Soils	Granular and Crushed Rock Maximum Particle Size ≤ 1½ inch	Crushed Rock Maximum Particle Size > 1½ inch
Plate Compactors and Jumping Jacks	4 to 8	4 to 8	Not Recommended
Rubber-Tire Equipment	6 to 8	8 to 12	6 to 8
Light Roller	8 to 10	8 to 12	8 to 10
Heavy Roller	10 to 12	12 to 18	12 to 16
Hoe Pack Equipment	12 to 16	18 to 24	12 to 16

Note:

The above table is based on our experience and is intended to serve as a guideline. The information provided in this table should not be included in the project specifications.

10. Additional Geotechnical Services

Before construction begins, we recommend that we review the final design plans and specifications to verify the geotechnical engineering recommendations have been properly interpreted and implemented into the design.

During the construction phase of the project, we recommend that we be retained to review contractor submittals and conduct the following activities:

- Observe subgrade preparation for foundations and pavements;
- Observe deep foundation installation;
- Observe load testing of deep foundations and review GRL WEAP analysis results;
- Review or provide PDA/CAPWAP data;
- Review contractor submittals for pile driving, dewatering, materials, and other geotechnically relevant items;
- Observe construction of pavements; and
- Perform confirmatory infiltration testing.

The purpose of these observations and services is to note compliance with the design concepts, specifications, or recommendations, and to allow design changes or evaluation of appropriate construction measures in the event that subsurface conditions differ from those anticipated prior to the start of construction.

11. Limitations

This report has been prepared for specific application to the proposed construction as understood at this time. In the event that changes in the nature, design, or location of the project are planned, the conclusions and recommendations contained in this report should not be considered valid, unless the changes are reviewed by Haley & Aldrich and the conclusions of this report modified or verified in writing.

The geotechnical analyses and recommendations are based, in part, upon the data obtained from the referenced subsurface exploration. The nature and extent of variations between explorations may not become evident until construction. If variations appear at that time, it may be necessary to re-evaluate the recommendations of this report.

This report is prepared for the exclusive use of Burns & McDonnell, JH Kelly, and their consultants in pursuit of the proposed PDX Fuel Tank Design in Portland, Oregon. There are no intended beneficiaries other than Burns & McDonnell, JH Kelly, and their consultants. Haley & Aldrich shall owe no duty whatsoever to any other person or entity on account of the Agreement or the report. Use of this report by any person or entity other than Burns & McDonnell, JH Kelly, and their consultants for any purpose whatsoever is expressly forbidden unless such other person or entity obtains written authorization from Burns & McDonnell, JH Kelly, and Haley & Aldrich. Use of this report by such other person or entity without the written authorization of Burns & McDonnell, JH Kelly, and Haley & Aldrich shall be at such other person's or entity's sole risk and shall be without legal exposure or liability to Haley & Aldrich.

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

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TABLE

TABLE 4

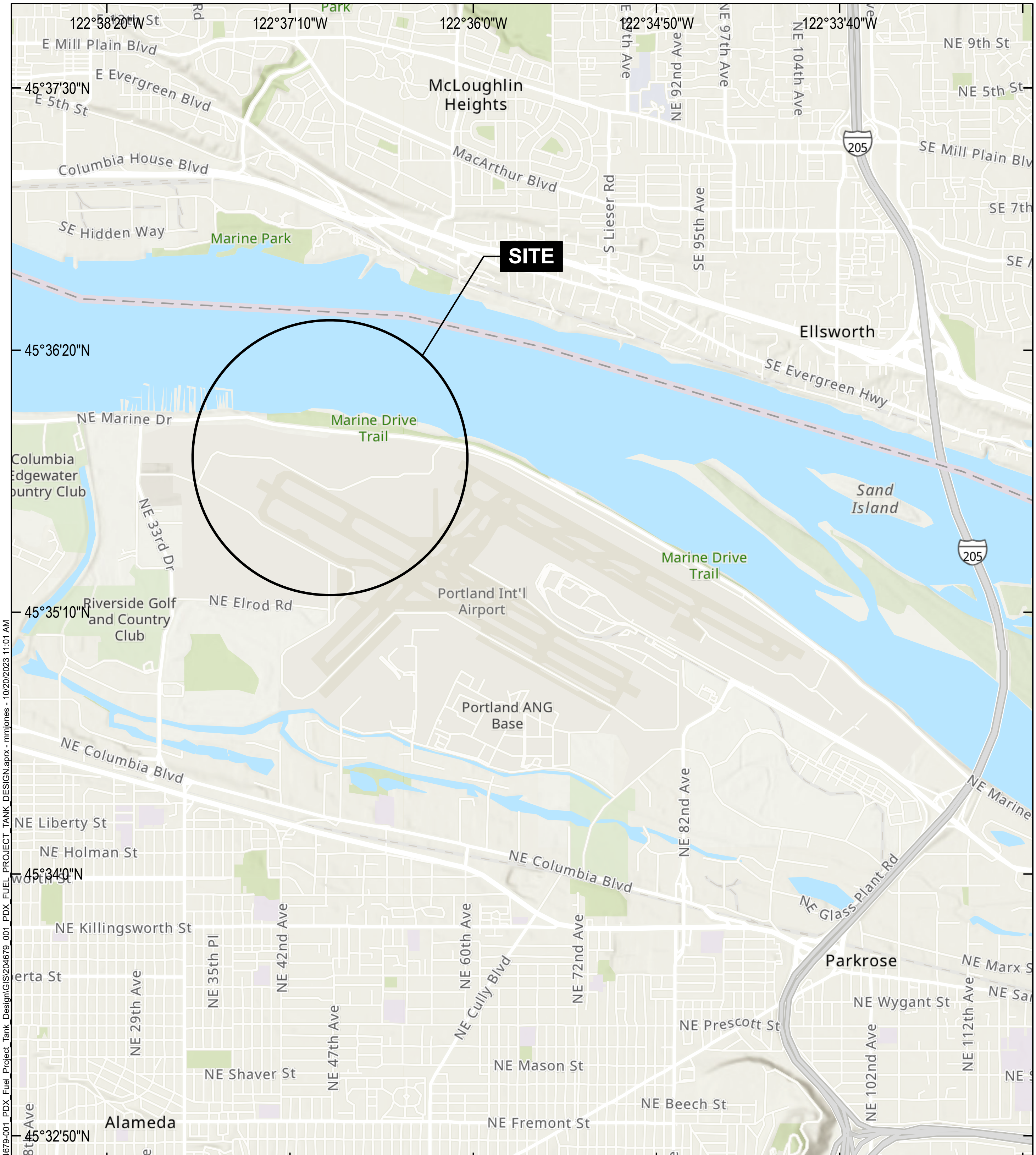
TABULATED VALUES FOR ESTIMATED LIQUEFACTION-INDUCED VERTICAL SOIL SETTLEMENT

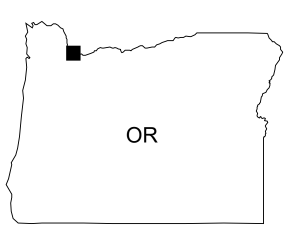
PDX Fuel Tank Design

Portland International Airport

Elevation (feet)	Settlement (inches)	Elevation (feet)	Settlement (inches)
21.3	10.2	-55.4	2.9
19.4	10.2	-57.2	2.8
17.5	10.1	-59.1	2.7
15.7	10.1	-61.0	2.7
13.8	10.1	-62.8	2.6
11.9	10.0	-64.7	2.5
10.1	10.0	-66.6	2.4
8.2	10.0	-68.4	2.3
6.3	9.9	-70.3	2.2
4.4	9.9	-72.2	2.0
2.6	9.8	-74.0	1.8
0.7	9.7	-75.9	1.7
-1.2	9.5	-77.8	1.6
-3.0	9.3	-79.6	1.5
-4.9	9.1	-81.5	1.5
-6.8	8.8	-83.4	1.4
-8.6	8.5	-85.3	1.4
-10.5	8.2	-87.1	1.3
-12.4	7.9	-89.0	1.3
-14.2	7.6	-90.9	1.2
-16.1	7.2	-92.7	1.2
-18.0	7.0	-94.6	1.2
-19.8	6.6	-96.5	1.1
-21.7	6.3	-98.3	1.1
-23.6	6.0	-100.2	1.0
-25.5	5.6	-102.1	1.0
-27.3	5.2	-103.9	0.9
-29.2	4.9	-105.8	0.9
-31.1	4.5	-107.7	0.8
-32.9	4.3	-109.6	0.7
-34.8	4.1	-111.4	0.7
-36.7	3.9	-113.3	0.6
-38.5	3.8	-115.2	0.6
-40.4	3.6	-117.0	0.5
-42.3	3.5	-118.9	0.5
-44.1	3.4	-120.8	0.4
-46.0	3.3	-122.6	0.4
-47.9	3.2	-124.5	0.3
-49.7	3.2	-126.4	0.3
-51.6	3.1	-128.2	0.2
-53.5	3.0	-130.1	0.2

FIGURES





N

MAP SOURCE: ESRI
SITE COORDINATES: 45°35'06"N, 122°35'39"W

HALEY ALDRICH

PDX FUEL TANK DESIGN
PORTLAND INTERNATIONAL AIRPORT
PORTLAND, OREGON

VICINITY MAP

APPROXIMATE SCALE: 1 IN = 1 MILE
OCTOBER 2023

FIGURE 1

GIS: \\haleyaldrich.com\share\pdx_data\Notebooks\0204679-001_PDX_Fuel_Project_Tank_Design\GIS\204679_001_PDX_FUEL_PROJECT_TANK_DESIGN.aprx - mmjones - 10/20/2023 11:01 AM

FOR REFERENCE ONLY

GIS FILE PATH: \\haleyaldrich.com\share\pdx_data\Notebooks\0204679-001_PDX_Fuel_Project_Tank_Design\GIS\204679_001_PDX_FUEL_PROJECT_TANK_DESIGN.aprx - USER: mmjones - LAST SAVED: 10/20/2023 10:53 AM

LEGEND

- CURRENT EXPLORATION (H&A, 2023)
- PREVIOUS EXPLORATION (GRI, 2017)
- PREVIOUS EXPLORATION (H&A, 2019)
- TEST PITS CURRENT EXPLORATION (H&A, 2023)
- CROSS SECTION
- BATHYMETRIC ELEVATION CONTOUR, 5-FT INTERVAL (NAVD 88)
- TOPOGRAPHIC ELEVATION CONTOUR, 5-FT INTERVAL (NAVD 88)

NOTES

1. ALL LOCATIONS AND DIMENSIONS ARE APPROXIMATE.
2. TOPOGRAPHY/BATHYMETRY SOURCE: US ARMY CORPS OF ENGINEERS, 2010.
3. AERIAL IMAGERY SOURCE: NEARMAP, 14 AUGUST 2022.
4. NORTH AMERICAN VCERTICAL DATUM OF 1988 (NAVD88)

N

0 800 1,600

SCALE IN FEET

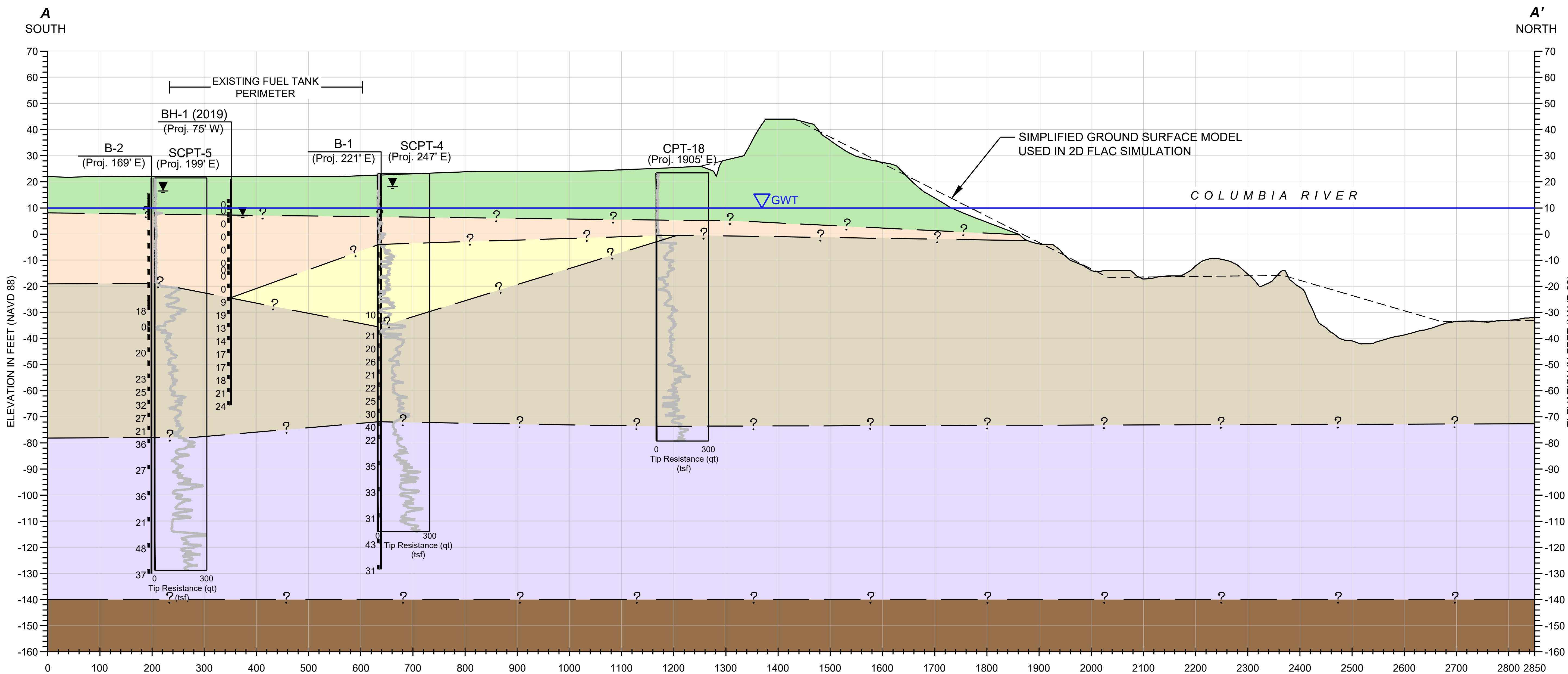
HALEY ALDRICH

PDX FUEL TANK DESIGN
PORTLAND INTERNATIONAL AIRPORT
PORTLAND, OREGON

SITE PLAN

OCTOBER 2023

FIGURE 2



LEGEND

EXPLORATION

B-1
(Proj. 221' E)

STANDARD PENETRATION RESISTANCE IN BLOWS PER FOOT

WATER LEVEL

USCS CLASSIFICATION

EXPLORATION DEPTH IN FEET

T.D.= 151.5

CPT EXPLORATION

CPT-18
(Proj. 1905' E)

Tip Resistance (qt) (tsf)

ESU-1 - FILL & OVERBANK DEPOSIT (ABOVE G.W.T)

ESU-2 - OVERBANK DEPOSIT

ESU-3A - COLUMBIA RIVER SAND (DR =40%)

ESU-3B - COLUMBIA RIVER SAND (SP)(DR = 50%)

ESU-3C - COLUMBIA RIVER SAND (SP)(DR = 58%)

VERY DENSE SANDY GRAVEL (ELASTIC HALFSPACE VS = 1271 FT/SEC)

NOTES

1. CONTACTS BETWEEN SOIL UNITS ARE BASED UPON INTERPOLATION BETWEEN BORINGS AND REPRESENT OUR INTERPRETATION OF SUBSURFACE CONDITIONS BASED ON CURRENTLY AVAILABLE DATA.
2. PROFILE SURFACE CREATED FROM A DIGITAL ELEVATION MODEL (DEM) CREATED BY THE LOWER COLUMBIA ESTUARY PARTNERSHIP.

HORIZ. 0 200 400
VERT. 0 40 80
SCALE IN FEET

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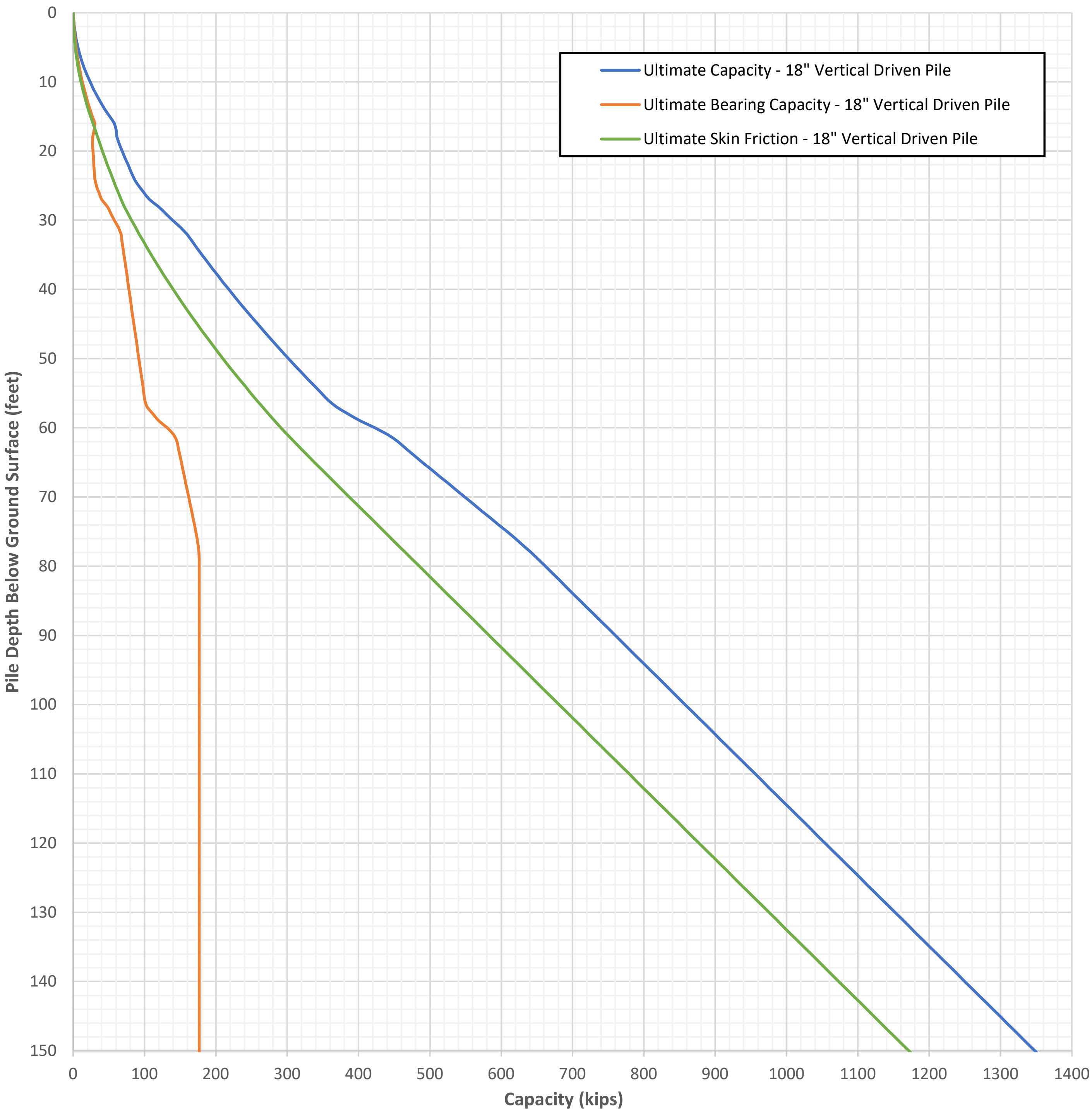
PDX FUEL TANK - SEISMIC VULNERABILITY ASSESSMENT
PORTLAND INTERNATIONAL AIRPORT
5000 NE MARINE DRIVE
PORTLAND, OREGON 97218

SUBSURFACE CROSS SECTION A-A'

DECEMBER 2023

FIGURE 3



NOTES

1. PROVIDED CAPACITIES ARE ULTIMATE (UNFACTORED). DESIGNER IS RESPONSIBLE FOR APPLYING FACTORS OF SAFETY / RESISTANCE FACTORS APPROPRIATE FOR DESIGN METHOD.

2. END BEARING CAPACITY IS CALCULATED ASSUMING EITHER A PLUG IS FORMED (FOR DRIVEN OPEN-ENDED PIPE PILES) OR CLOSED-END CONDITIONS FOR TORQUE-DOWN PILES

3. RECOMMENDATIONS ARE FOR VERTICAL 18-INCH-DIAMETER STEEL PIPE PILES

4. GROUND SURFACE ELEVATION ASSUMED AS EL. 22 FT (NAVD88).

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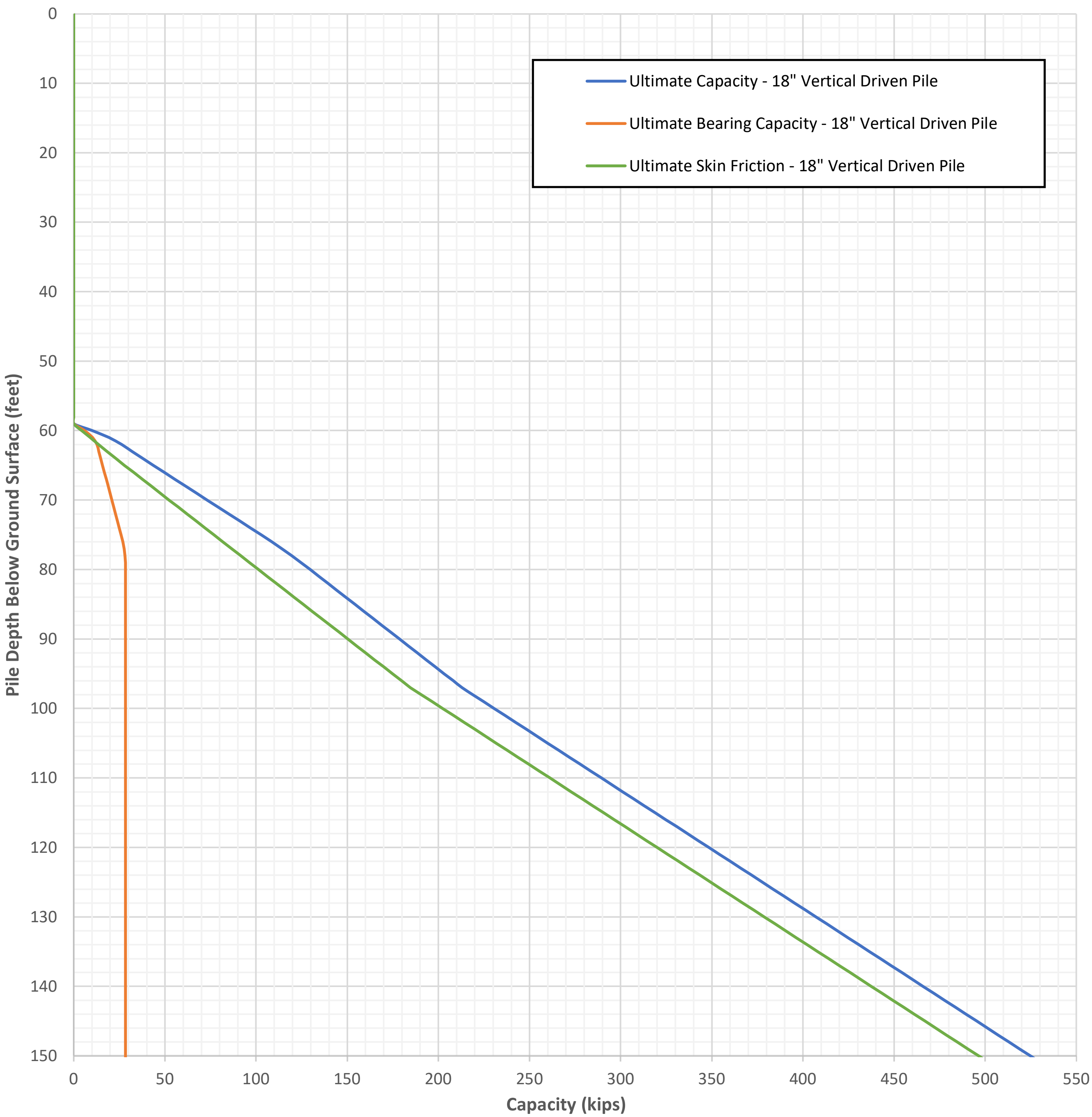
PDX FUEL TANK DESIGN
PORTLAND INTERNATIONAL AIRPORT
PORTLAND, OREGON

18-INCH-DIAMETER PILE CAPACITIES STATIC CONDITION

SCALE: AS SHOWN
DECEMBER 2023

FIGURE 4



NOTES

1. PROVIDED CAPACITIES ARE ULTIMATE (UNFACTORED). DESIGNER IS RESPONSIBLE FOR APPLYING FACTORS OF SAFETY / RESISTANCE FACTORS APPROPRIATE FOR DESIGN METHOD.

2. END BEARING CAPACITY IS CALCULATED ASSUMING EITHER A PLUG IS FORMED (FOR DRIVEN OPEN-ENDED PIPE PILES) OR CLOSED-END CONDITIONS FOR TORQUE-DOWN PILES

3. RECOMMENDATIONS ARE FOR VERTICAL 18-INCH-DIAMETER STEEL PIPE PILES

4. GROUND SURFACE ELEVATION ASSUMED AS EL. 22 FT (NAVD88).

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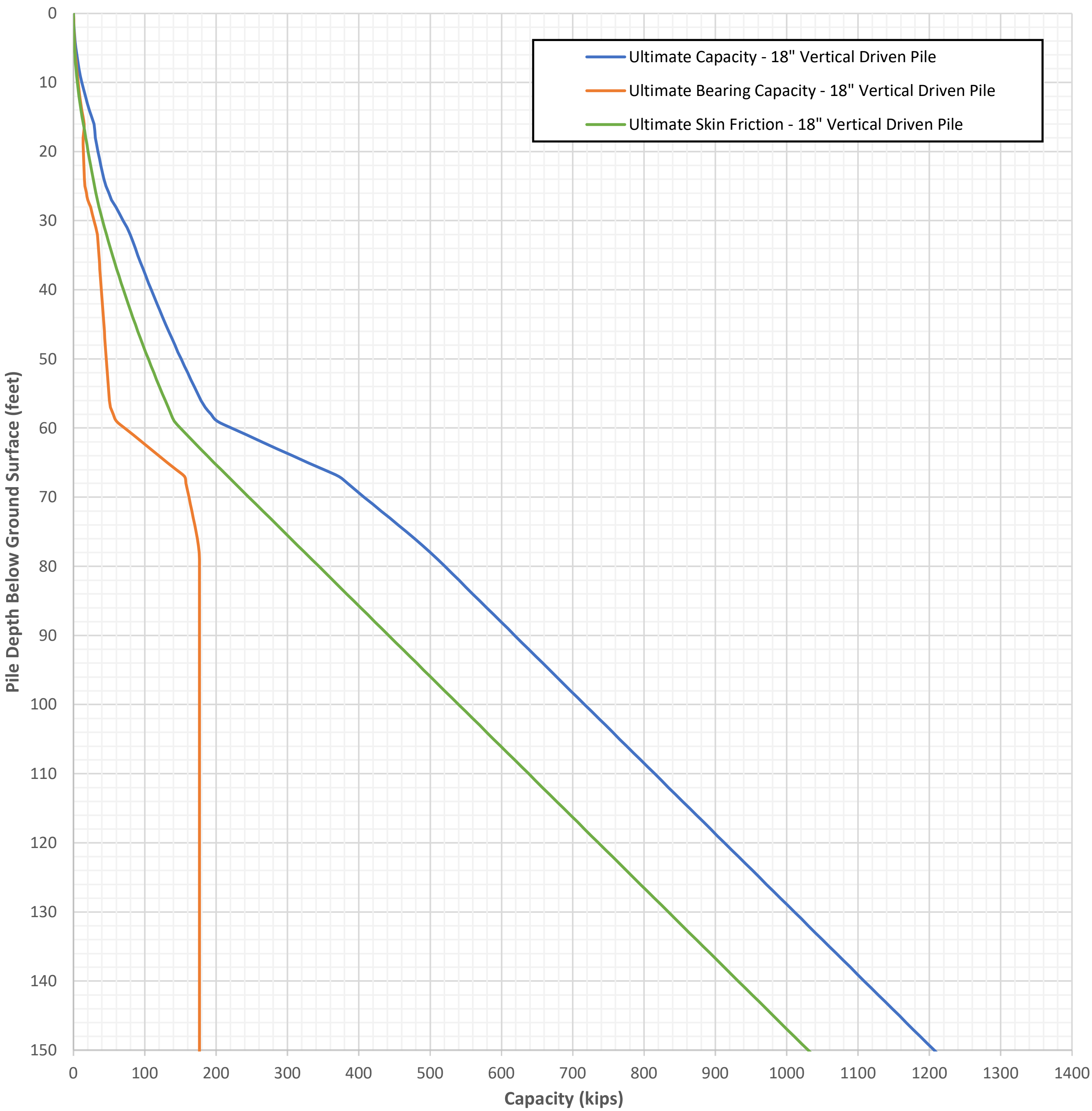
PDX FUEL TANK DESIGN
PORTLAND INTERNATIONAL AIRPORT
PORTLAND, OREGON

18-INCH-DIAMETER PILE CAPACITIES
LIQUEFIED CONDITION

SCALE: AS SHOWN
DECEMBER 2023

FIGURE 5



NOTES

1. PROVIDED CAPACITIES ARE ULTIMATE (UNFACTORED). DESIGNER IS RESPONSIBLE FOR APPLYING FACTORS OF SAFETY / RESISTANCE FACTORS APPROPRIATE FOR DESIGN METHOD.

2. LIQUEFACTION-INDUCED DOWNDRAG ANALYSIS MUST BE PERFORMED WHEN USING VALUES ON THIS FIGURE. SEE REPORT SECTION 6.1.2.

3. END BEARING CAPACITY IS CALCULATED ASSUMING EITHER A PLUG IS FORMED (FOR DRIVEN OPEN-ENDED PIPE PILES) OR CLOSED-END CONDITIONS FOR TORQUE-DOWN PILES

4. RECOMMENDATIONS ARE FOR VERTICAL 18-INCH-DIAMETER STEEL PIPE PILES

5. GROUND SURFACE ELEVATION ASSUMED AS EL. 22 FT (NAVD88).

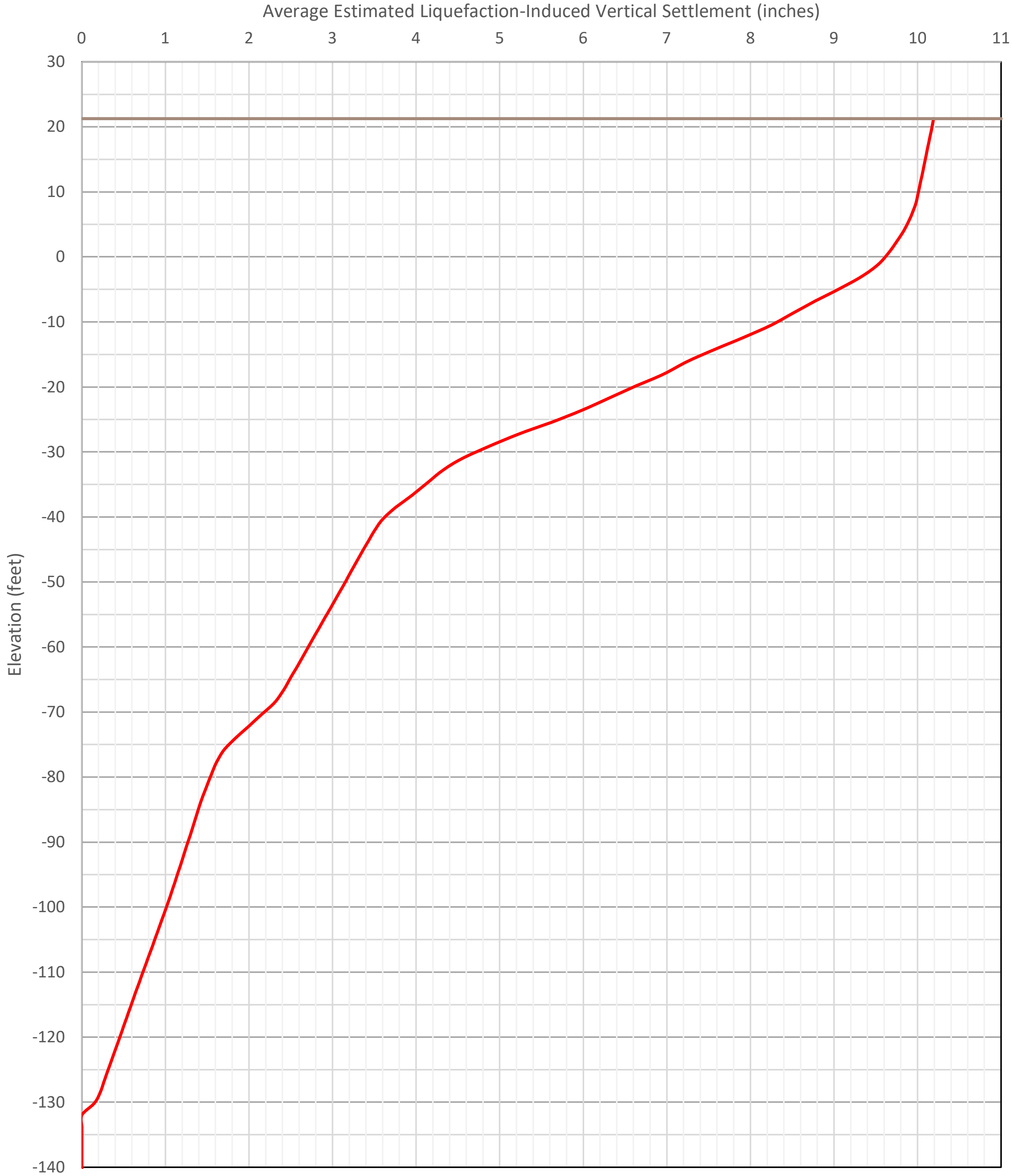
HALEY ALDRICH

PDX FUEL TANK DESIGN
PORTLAND INTERNATIONAL AIRPORT
PORTLAND, OREGON

18-INCH-DIAMETER PILE CAPACITIES
POST-LIQUEFIED CONDITION

SCALE: AS SHOWN
DECEMBER 2023

FIGURE 6



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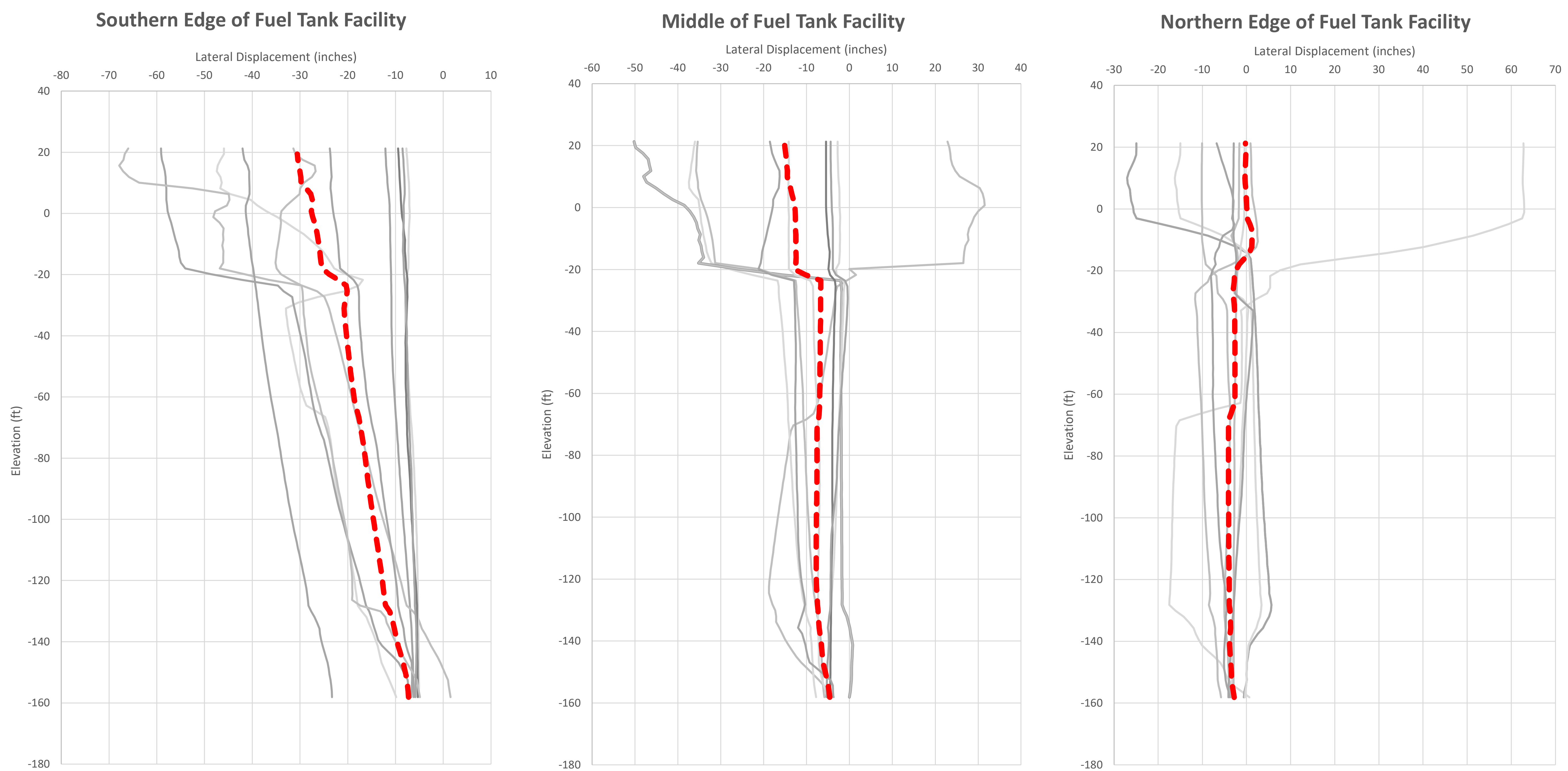
PORTLAND INTERNATIONAL AIRPORT
500 NE MARINE DRIVE
PORTLAND, OREGON 97218

ESTIMATED VERTICAL SEISMIC SETTLEMENT

SCALE: AS SHOWN
DECEMBER 2023

FIGURE 7



LEGEND

- NGAsubRSN6001811_MET-EW
- RSN6887_DARFIELD_CBGSS01W
- NGAsubRSN4000035_522-NS
- RSN5478_IWATE_AKT023EW
- NGAsubRSN4000113_904-NS
- Recommended Design Average
- NGAsubRSN6001803_SLUC090
- NGAsubRSN4000040_527-NS
- RSN1197_CHICHI_CHY028-N
- NGAsubRSN2000001_OLY0086
- NGAsubRSN7006531_KAU080--E

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ESTIMATED LATERAL SEISMIC DISPLACEMENTS

SCALE: AS SHOWN
DECEMBER 2023

FIGURE 8