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GEOTECHNICAL ENGINEERING REPORT

City of Tigard – Accessible Boat Launch at Cook Park

TIGARD, OREGON

January 2024
Shannon & Wilson No: 112406

SHANNON & WILSON

Submitted To: City of Tigard
13125 SW Hall Boulevard
Tigard, OR 97223
Attn: Jeff Peck

Subject: GEOTECHNICAL ENGINEERING REPORT, CITY OF TIGARD – ACCESSIBLE BOAT LAUNCH AT COOK PARK, TIGARD, OREGON

Shannon & Wilson, Inc. prepared this report and participated in this project as a consultant to the City of Tigard. Our scope of services was specified in Purchase Order Number P2400040, dated November 2, 2023. This report presents the results of our field exploration, laboratory testing, geotechnical design evaluations and recommendations, and construction considerations for the proposed project, and was prepared by the undersigned.

We appreciate the opportunity to be of service to you on this project. If you have questions concerning this report, or if we may be of further service, please contact us.

Sincerely,

SHANNON & WILSON, INC.

REGISTERED PROFESSIONAL ENGINEER
96994PE
OREGON
MARCH 09, 2021
KYLE ROMNEY
EXPIRES: 06/30/2024

Kyle Romney, PE
Senior Engineer

Stephen McLandrich

Stephen McLandrich, PE, GE
Associate

KTR::SMM/mmb

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ACRONYMS

FS	Factor of Safety
GDM	Geotechnical Design Manual
IBC	International Building Code
ODOT	Oregon Department of Transportation
OHW	Ordinary High Water
OLW	Ordinary Low Water
OSSC	Oregon Standard Specifications for Construction

1 INTRODUCTION

This report presents the results of our field exploration, laboratory testing, geotechnical design evaluations and recommendations, and construction considerations for the proposed Accessible Boat Launch project at Cook Park in Tigard, Oregon. The City of Tigard (the City), along with their engineering consultant Century West, Inc. (Century West), are planning to construct a new floating boat launch adjacent to the existing floating boat launch at Cook Park. As a consultant to the City, Shannon & Wilson, Inc. (Shannon & Wilson) is providing geotechnical engineering services to support the engineering design of the proposed new floating boat launch foundations. The location of the project site is shown on the Vicinity Map, Figure 1.

2 PROJECT UNDERSTANDING

2.1 Site Description

The proposed accessible boat launch will be located at the south end of the park between the existing Boat Launch and Shelter 3 at Cook Park in Tigard, Oregon. The project site is on the north side of the Tualatin River where it makes a large southern bend. Tigard High School is to the north of the site and Durham City Park to the east. The topography at the project site gradually slopes down to the river, increasing in proximity to the Tualatin River, and is currently a grass-covered lawn with tall bushes and grasses along the riverbank. An asphalt parking lot and paved walkway are located at the top of this slope above the river. A photograph of the site is shown in Exhibit 2-1, below.

Exhibit 2-1: View looking east from the parking lot at the area where the new accessible boat launch will be located.

2.2 Project Description

It is our understanding that a new concrete pathway will extend from the existing parking lot to a gangway which will connect to the accessible dock. Based on preliminary design drawings provided by Century West, the proposed accessible boat launch will consist of a gangway ramp connected to a floating dock with an accessible boat launch for pedestrian use. Design drawings indicate two 12-inch diameter steel pipe piles will secure the gangway ramp at the top and four 16-inch diameter steel pipe piles will secure the floating dock. The depth of the pile foundations has not been finalized but is anticipated to extend into the Hillsboro Formation. Vertical and lateral loading has not been provided at this time. We understand that the structural engineer will use the geotechnical allowable axial resistance and lateral loading properties of the subsurface soils, in the form of L-Pile parameters, for use in developing the foundation design.

2.3 Scope of Services

Shannon & Wilson's services were conducted in accordance with the scope of services defined in Purchase Order #P2400040, dated November 2, 2023. The completed geotechnical design services for the project consisted of the following tasks:

- Observe surface and geologic conditions of the site and identify site constraints, staging concerns for geotechnical exploration and construction, and submit utility locate ticket for proposed geotechnical exploration;
- Drill one geotechnical boring to evaluate the subsurface conditions at the proposed site;
- Perform lab testing on selected soil samples collected from geotechnical boring;
- Provide recommended axial resistance for 12- and 16-inch-diameter steel piles;
- Provide lateral loading soil properties in the form of L-Pile parameters for the driven steel piles;
- Evaluate slope stability of proposed earthwork slopes required for gangway construction; and
- Provide this Geotechnical Engineering Report summarizing foundation design recommendations and construction considerations for the proposed improvements.

3 GEOLOGIC SETTING

3.1 Regional and Local Geology

The project site lies in the eastern half of the Tualatin Basin: an approximately 35-mile-long by 20-mile-wide, northwest-trending, gently sloping synclinal valley (Madin, 1990). The Tualatin Basin is one of several localized sub-basins within the Willamette Lowland, which is a broader regional geologic depression (Gannett and Caldwell, 1998). The basins are structural depressions, created by complex folding and faulting of the basement rocks (Schlicker and Deacon, 1967). The basement, or floor, of the basins is made up of lava flows collectively referred to as the Columbia River Basalt Group (CRBG), which flowed into the area in the middle Miocene epoch, between about 17 and 6 million years ago.

Over the span of geologic time, sedimentary deposits consisting of clay, silt, sand, and gravel eroded from the surrounding uplands and settled into the basins that formed on the CRBG surface. In the Tualatin Basin, these sediments have historically been referred to by several names, including the Troutdale Formation (Schlicker and Deacon, 1967); the Sandy River Mudstone equivalent (Madin, 1990); and the Hillsboro Formation (Wilson, 1998). For the purposes of this report, Shannon & Wilson refer to these upper-Miocene to Pleistocene age (approximately 11- to 1-million-year-old) basin-fill sediments as Hillsboro Formation, after Wilson (1998).

The Hillsboro Formation varies in thickness, reaching up to 860 feet near the center of the basin and tapering out at the basin margins. It consists predominantly of clay and silt, with some thin sand layers and rare gravelly sands. It is not known to be exposed at the ground surface and is generally covered by younger alluvial deposits or rocks of the Boring volcanic field (on the west slopes of the Tualatin Mountains).

The Hillsboro Formation is extensively overlain by a layer of late-Pleistocene Missoula Flood sediment. During the late stages of the last great ice age, between about 18,000 and 15,000 years ago, a lobe of the continental ice sheet repeatedly blocked and dammed the Clark Fork River in western Montana, which then formed an immense glacial lake called Lake Missoula. The lake grew until its depth was sufficient to buoyantly lift and rupture the ice dam, which allowed the entire massive lake to empty catastrophically. Once the lake had emptied, the ice sheet again gradually dammed the Clark Fork Valley and the lake refilled, leading to 40 or more repetitive outburst floods at intervals of decades (Allen and others, 2009). During each short-lived episode, floodwaters washed across the Idaho panhandle, through the eastern Washington scablands, and through the Columbia River Gorge. When the floodwater emerged from the western end of the gorge, it spread out over the Portland Basin and up the Willamette Valley as far south as Junction City, depositing a Tremendous load of sediment (O'Conner and others, 2001). The floods deposited extensive gravel bars across east Portland and up to 50 feet of micaceous sand and clayey to fine sandy silt in the Tualatin Basin. Fine-grained Missoula Flood sediments deposited in the Tualatin Basin are commonly referred to as Willamette Silt (Wilson, 1998). For the purposes of this report, Shannon & Wilson refer to these sediments as Fine-grained Missoula Flood Deposits, after Ma and others (2012).

The Tualatin River and its many tributary creeks and streams have locally eroded the older sediments (principally Missoula Flood deposits) and re-deposited the sediment along their modern floodplains. These modern sedimentary deposits generally consist of clay, silt, fine-grained sand, and organic material with minor gravel. Thickness of the Holocene (recent) alluvium varies with location and is difficult to determine, because there is little distinction between it and the deposits from which it originated.

4 FIELD EXPLORATIONS

The geotechnical field exploration program included a single geotechnical boring, designated B-1. The geotechnical boring was performed near the top of the proposed gangway ramp. The approximate location of the boring is shown on Figure 2, Site and Exploration Plan.

It should be noted that a geotechnical boring was not also drilled at the proposed location for the floating dock piles. An in-water work permit would be required to drill a boring in the Tualatin River. However, the process of obtaining the in-water work permit and permitted work window would not have allowed us to meet the project schedule. After comparing subsurface conditions encountered in the single project geotechnical boring, B-1, to nearby geotechnical borings, the subsurface conditions were assumed to be consistent across the project site.

The geotechnical boring for this project was drilled on November 10, 2023, to an approximate depth of 61.5 feet below ground surface (bgs) using mud rotary techniques. The track-mounted CME-550 drill rig was provided and operated by Western States Soil Conservation, Inc., of Hubbard, Oregon.

A Shannon & Wilson geologist was present during the site reconnaissance to locate the boring and during drilling to collect and log the soil samples encountered.

5 LABORATORY TESTING

The samples obtained during drilling were transported to the Shannon & Wilson laboratory for further examination. The soil testing program included moisture content tests, particle-size analyses, and Atterberg limits tests. All samples were tested in Shannon & Wilson's laboratory. All test procedures were performed in accordance with applicable ASTM International standards. Results of the laboratory tests and brief descriptions of the test procedures are presented in Appendix B.

6 SUBSURFACE CONDITIONS

6.1 Geotechnical Units

Shannon & Wilson grouped the materials encountered in the geotechnical boring into the following geotechnical units, as described below. This interpretation of the subsurface conditions is based on the geotechnical exploration and regional geologic information from published sources. The geotechnical units are as follows:

- **Alluvium**: Loose/very soft Sandy Silt to Sandy Elastic Silt (ML/MH) with trace organics, and very loose Silty Sand (SM) with trace organics;
- **Marsh Deposits**: Very loose Silty Sand (SM) with some to mostly organics;
- **Fine-Grained Missoula Flood Deposits**: Stiff to very stiff Lean Clay (CL) and medium dense to very stiff Sandy Silt (ML);
- **Hillsboro Formation**: Very stiff to hard Lean Clay to Lean Clay with Sand (CL) and dense Silty Sand (SM).

These geotechnical units were grouped based on their engineering properties, geologic origins, and their distribution in the subsurface. Contacts between the units may be more gradational than shown in the boring log in Appendix A. The Standard Penetration Test (SPT) N-values shown on the boring logs are as recorded in the field (uncorrected).

As aforementioned, subsurface conditions based on boring B-1 were assumed across the project site. Additional geotechnical borings would be required to determine subsurface conditions at other locations within the project site.

6.2 Groundwater

The geotechnical boring B-1 was advanced using mud rotary drilling techniques that introduce fluids into the borehole. This makes it difficult to discern the depth of groundwater if it is encountered during drilling.

Groundwater is interpreted to be near or slightly above the elevation of the Tualatin River and is expected to fluctuate at the site with fluctuations in the Tualatin River. Locally, groundwater highs typically occur in the late fall to spring, and groundwater lows typically occur in the late summer and early fall. However, for design purposes, the groundwater was assumed to be at the Ordinary High Water (OHW) elevation of 108 feet, based on preliminary drawings.

7 BOAT LAUNCH DESIGN RECOMMENDATIONS

7.1 General

Based on preliminary drawings provided by Century West, it is our understanding that the new accessible boat launch will be founded/secured on two 12-inch-diameter driven steel pipe piles at the top of the gangway and four 16-inch-diameter driven steel pipe piles at the floating dock. No seismic hazard analysis is required. The design recommendations contained within the subsequent sections of this report are based on these assumptions.

7.2 Gangway Cut Slope Global Stability

Global stability was evaluated at the proposed cut slope to accommodate the gangway assuming groundwater at both Ordinary High Water (OHW) and Ordinary Low Water (OLW) elevations. Generalized subsurface conditions were estimated based on boring B-1, presented in Appendix A. Soil parameters were estimated from the results of field explorations and laboratory testing.

We conducted global stability analyses for the proposed cut slope using the computer program SLOPE/W, Version 10 (Geo-Slope International, 2019). This program employs limit-equilibrium methods in accordance with the ODOT GDM (ODOT, 2019). The Morgenstern-Price slope stability analysis method was used for rotational and irregular surface failure mechanisms. The analysis was performed for static conditions only.

The global stability analyses for the proposed cut slope for groundwater conditions at the OHW and OLW elevations resulted in a satisfactory Factor of Safety (FS) of 1.5 and 1.9, respectively. However, only the results of the global stability analysis for OHW, presented in Figure 3, are included at the end of this report.

7.3 Driven Pile Design Recommendations

7.3.1 General

The following sections provide our recommendations for axial resistance and lateral resistance soil properties of driven steel pipe piles. Based on preliminary design information provided by Century West, we evaluated 12-inch- and 16-inch-diameter steel pipe piles. Pile wall thickness is not available at this time so, for design purposes, a wall thickness of 0.375 inches was assumed. Based on the subsurface conditions, we recommend the piles be driven or vibrated open-ended.

7.3.2 Driven Pile Axial Resistance

We recommend that the steel pipe piles conform to the requirements of ASTM A252, Grade 3. Mill certification of the steel should be provided by the supplier. All portions of pile design and construction should meet the requirements of Oregon Standard Specifications for Construction (OSSC) Section 00520 (ODOT, 2024) and its project special provisions. Exhibit 7-1 presents the typical pile section design properties.

Exhibit 7-1: Steel Pipe Pile Section Properties

Pile Type	Steel Grade (kips/inch^2)	Section Area (inch^2)	Nominal Structural Capacity (kips)
PP 12x0.375	A252 Grade 3	13.7	615
PP 16x0.375	A252 Grade 3	18.4	825

We recommend the ultimate and allowable compressive resistance of the piles is established using an FS of 3.0.

Our axial resistance analysis results are presented on Figures 4 and 5. These results are presented as plots of ultimate and allowable axial resistance versus depth for static conditions only. The resistances presented are based on a single pile and do not consider axial group effects due to our understanding that the piles will be spaced at least 2.5 pile diameters (2.5D) apart (center-to-center).

7.3.3 Driven Pile Lateral Resistance Soil Properties

The driven pile foundations will be subjected to lateral loads resulting from live loading. We understand that the laterally loaded pile analyses will be performed by the structural engineer responsible for the design of the piles with the aid of the L-Pile computer program.

Table 1, included at the end of this report, presents the recommended static L-Pile geotechnical input parameters for driven piles with center-to-center spacing greater than five pile diameters (5D) and in a single row.

7.3.4 Driven Pile Foundation Construction Considerations

7.3.4.1 Pile Driving Criteria

As previously stated, we recommend steel piles be installed using impact or vibratory techniques. However, if piles are installed using a vibratory hammer, we recommend using an impact hammer to confirm axial resistance of the pile. Additionally, we recommend that pile driving and installation of piles follow the OSSC, Section 00520 (ODOT, 2024), and its project special provisions. If splicing pile lengths in the leads is necessary to install the piles, then splicing locations should be approved by the structural engineer. All pile splices should be made according to the OSSC and procedures for piling with lateral and tension loading conditions. Also, the piles should be driven no closer together than 2.5 pile diameters (2.5D), measured from center-to-center and within 6 inches of locations shown on the plans. The pile driving alignment tolerance should follow the OSSC.

We recommend that the piles be driven to a minimum pile length of 30 feet below ground surface (bgs) considering the potential scour depth. However, the final minimum pile tip depth should be determined based on the geotechnical and structural design recommendations. If the specified bearing resistance is reached before the minimum tip elevation or minimum pile embedment, driving should continue until the minimum design requirements are reached. If driving must be terminated before the minimum requirements are achieved because driving stresses are greater than 90 percent of the yield strength, F_y (of the steel pile), or to prevent other damage to the pile or hammer, the driving records should be reviewed by the professional geotechnical engineer of record to evaluate both compressive and lateral resistance of the pile. In the case that a pile meets obstructions or terminal driving resistance with less than the minimum required pile embedment, the pile may need to be relocated, or the addition of piles to the pile group may be required. In such case, the pile cap design may need to be reevaluated by the structural engineer.

Prior to construction, driving criteria including the "last set" should be established for the specific pile driving equipment proposed for use. The hammer selected by the contractor should be capable of achieving the required nominal resistance at a blow count between 2 and 10 blows per inch, as determined by a WEAP analysis specified in the OSSC, Section 00520.42(c) (ODOT, 2024). This analysis should include the specific hammer, helmet, and cushion characteristics proposed by the contractors for the project.

Exhibit 7-2 presents the recommended WEAP input parameters. If Dynamic Pile Monitoring, or Pile Driving Analyzer® (PDA) testing, is used with the signal matching software program, CAPWAP, then an decreased FS may be used to establish factored pile resistance.

Exhibit 7-2: Recommended Input Parameters for WEAP Analysis

	Quake (in)		Damping (s/ft)			
Pile Section	Skin	Toe	Skin	Toe	Friction Distribution Shape	Percent Skin Friction (%)
PP12x0.375	0.10	0.20	0.20	0.15	triangular	85
PP16x0.375	0.10	0.20	0.20	0.15	triangular	70

During pile driving, a continuous record of pile driving resistance (bpf) should be maintained for the full length of each pile driven, as well as other pertinent information, including observed hammer performance. If an open-end diesel hammer is used to drive the piles, the pile driving contractor should supply a Saximeter™ during pile driving to determine an actual stroke height and pile driving energy. We recommend that an engineering staff representative, under the guidance of a professional geotechnical engineer, monitor pile driving in order to evaluate the suitability of each pile driven.

If piles do not meet driving criteria when driven to the specified lengths, redriving may be performed in accordance with OSSC, Section 00520.42(d) (ODOT, 2024). The piles should be allowed to stand for a "set period" of at least 24 hours without driving, then redriving should be performed. If the piles do not meet driving criteria during redriving, all piles should be further driven until the required bearing resistance is attained.

7.3.4.2 Pile Driving Vibration Impacts

We understand Shelter 3 is the nearest existing structure, approximately 100 feet from the proposed pile driving location for the gangway anchor piles. Therefore, pile driving vibration on the nearby shelter may be a concern. We recommend the contractor perform a pre-construction condition survey of any existing structure within 150 feet of the pile driving area to document pre-pile driving conditions and evaluate any possible post-pile driving damage. The pre-construction condition survey may include photographs, survey points, or other information that would allow the contractor to document pre-pile driving conditions and evaluate any potential building damage during pile driving. In addition, the design should evaluate whether any vibration-sensitive facilities, such as the nearby country club, are within 1/4-mile of the pile driving area.

7.3.4.3 Pile Driving Noise

Pile driving will be very noisy and may disturb nearby residents and the environment. The Agency should evaluate any potential noise impacts to surrounding residents. In addition, construction noise should meet local noise ordinances.

8 LIMITATIONS

The analyses, interpretations, conclusions, and recommendations contained in this report are based on site conditions as they presently exist and further assume that the exploration is representative of the subsurface conditions throughout the site; that is, the subsurface conditions everywhere are not significantly different from those disclosed by the past and current explorations. If future phases of work at the site uncover subsurface conditions different from those encountered in these explorations, we should be advised at once so that we can review these encountered conditions and reconsider our interpretations and conclusions, where necessary. If there is a substantial lapse of time between the submission of this report and the start of future phases of work at the site, or if conditions have changed because of natural forces or construction operations at or adjacent to the site, we recommend that we review our report to determine the applicability of the conclusions and recommendations.

Within the limitations of scope, schedule, and budget, the analyses, interpretations, conclusions, and recommendations presented in this report were prepared in accordance with generally accepted professional geotechnical engineering principles and practice in this area at the time this report was prepared. We make no other warranty, either express or implied. These interpretations and conclusions were based on our understanding of the project as described in this report and the site conditions as observed previously and at the time of our current explorations.

This report was prepared for the exclusive use of the City of Tigard. During future phases of work involving construction, the data contained in this report should be provided to the contractors for their information, but our report interpretations and conclusions should not be construed as a warranty of subgrade conditions as included in this report.

The scope of our present services did not include environmental assessments or evaluations regarding the presence or absence of wetlands, or hazardous or toxic substances in the soil, surface water, groundwater, or air, on or below or around this site, or for the evaluation or disposal of contaminated soils or groundwater should any be encountered.

Shannon & Wilson, Inc., has prepared and included, "Important Information About Your Geotechnical/Environmental Report," to assist you and others in understanding the use and limitations of our reports.

9 REFERENCES

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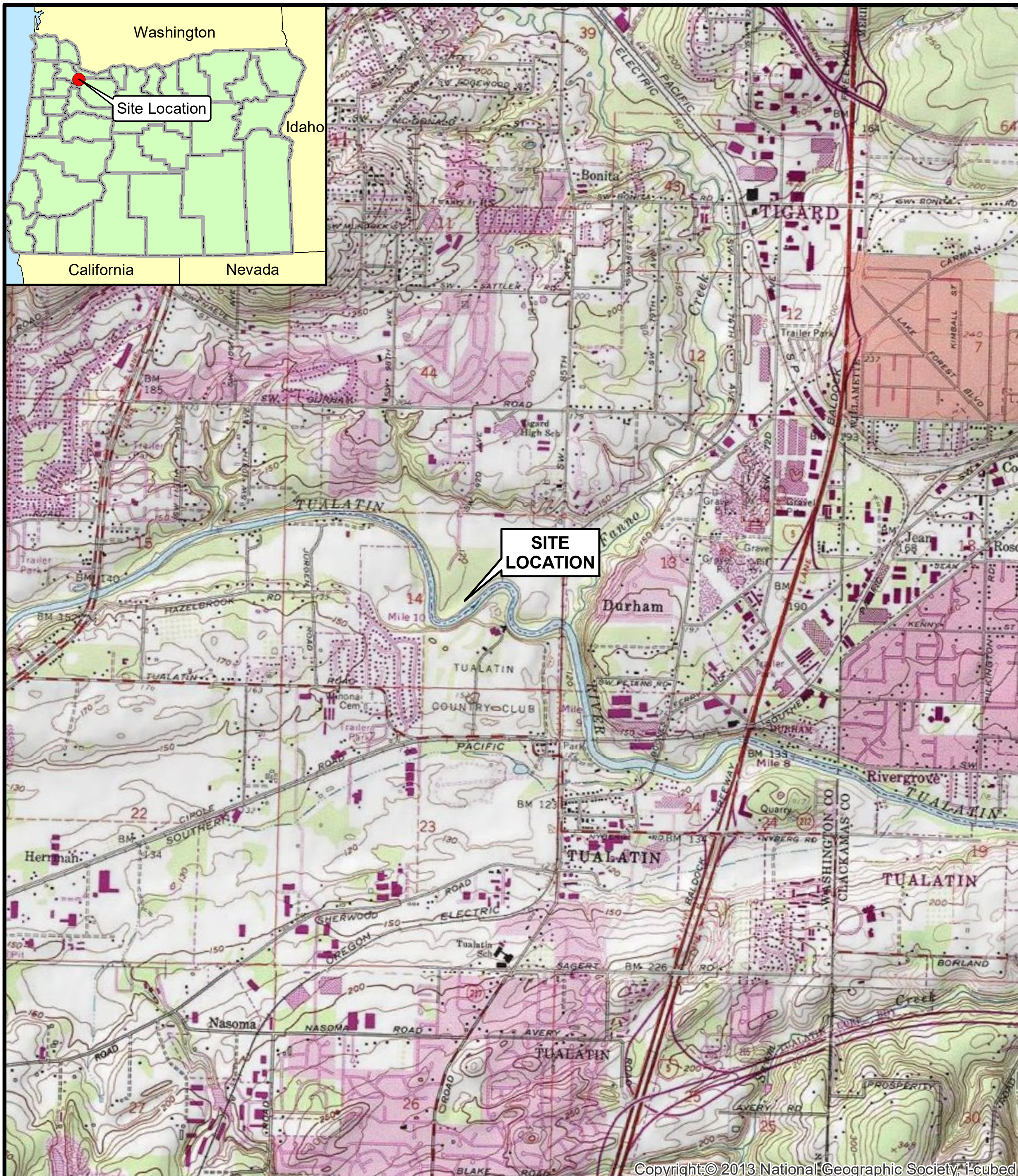
Schlicker, H.G. and Deacon, R.J., 1967, Engineering Geology of the Tualatin Valley region, Oregon: Oregon Department of Geology and Mineral Industries Bulletin B-60, 103 p., 5 app., 45 figs., 5 tables, 4 pls., scale 1:48,000.

Wilson, D.C., 1998, Post-middle Miocene geologic evolution of the Tualatin Basin, Oregon: Oregon Geology, v. 60, no. 5.

Table 1 - Recommended Static LPILE Geotechnical Input Parameters for Deep Foundations

	Depth[a] (feet)					Sand (Reese)		Clay	
	From	To	Soil Unit	Recommended P-Y Curve	Effective Unit Weight (pcf)	Friction Angle (deg)	k (pci)	Undrained Cohesion (psf)	Strain Factor, ε_{50}
Boring B-1	0	3.5	Alluvium / Marsh Deposits	Sand (Reese)	110	26	20	--	--
Boring B-1	3.5	23	Alluvium / Marsh Deposits	Sand (Reese)	43	26	10	--	--
Boring B-1	23	29	Fine-grained Missoula Flood Deposits	Stiff Clay w/o free water (Reese)	53	--	--	1,900	0.006
Boring B-1	29	33	Fine-grained Missoula Flood Deposits	Stiff Clay w/o free water (Reese)	53	--	--	2,900	0.005
Boring B-1	33	38	Fine-grained Missoula Flood Deposits	Sand (Reese)	53	29	60	--	--
Boring B-1	38	48	Hillsboro Formation	Stiff Clay w/o free water (Reese)	58	--	--	3,250	0.0045
Boring B-1	48	53	Hillsboro Formation	Stiff Clay w/o free water (Reese)	58	--	--	4,000	0.004
Boring B-1	53	58	Hillsboro Formation	Sand (Reese)	58	38	105	--	--
Boring B-1	58	Pile or Shaft Tip	Hillsboro Formation	Stiff Clay w/o free water (Reese)	58	--	--	2,350	0.0065

NOTES:
a Depth = 0 feet corresponds to ground surface
deg = degrees; pcf = pounds per cubic foot; pci = pounds per cubic inch; psf = pounds per square foot; psi = pounds per square inch; UCS = uniaxial compressive strength



0 0.25 0.5 1

Scale in Miles

N

Cook Park Accessible Boat Launch
Tigard, Oregon

VICINITY MAP

January 2024 112406

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GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

FIG. 1

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LEGEND

B-1 Designation and Approximate Location of Boring

0 50 100 200

Scale in Feet

NOTES

1. 2019 aerial imagery and taxlots obtained through Metro RLIS.
2. Existing contours and features from file 0330-024T1_z1.dwg, provided by Century West Engineering on December 7, 2023.
3. Proposed features from file Design_Base.dwg, provided by Century West Engineering on December 7, 2023.

Cook Park Accessible Boat Launch
Tigard, Oregon

SITE AND EXPLORATION PLAN

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FIG. 2

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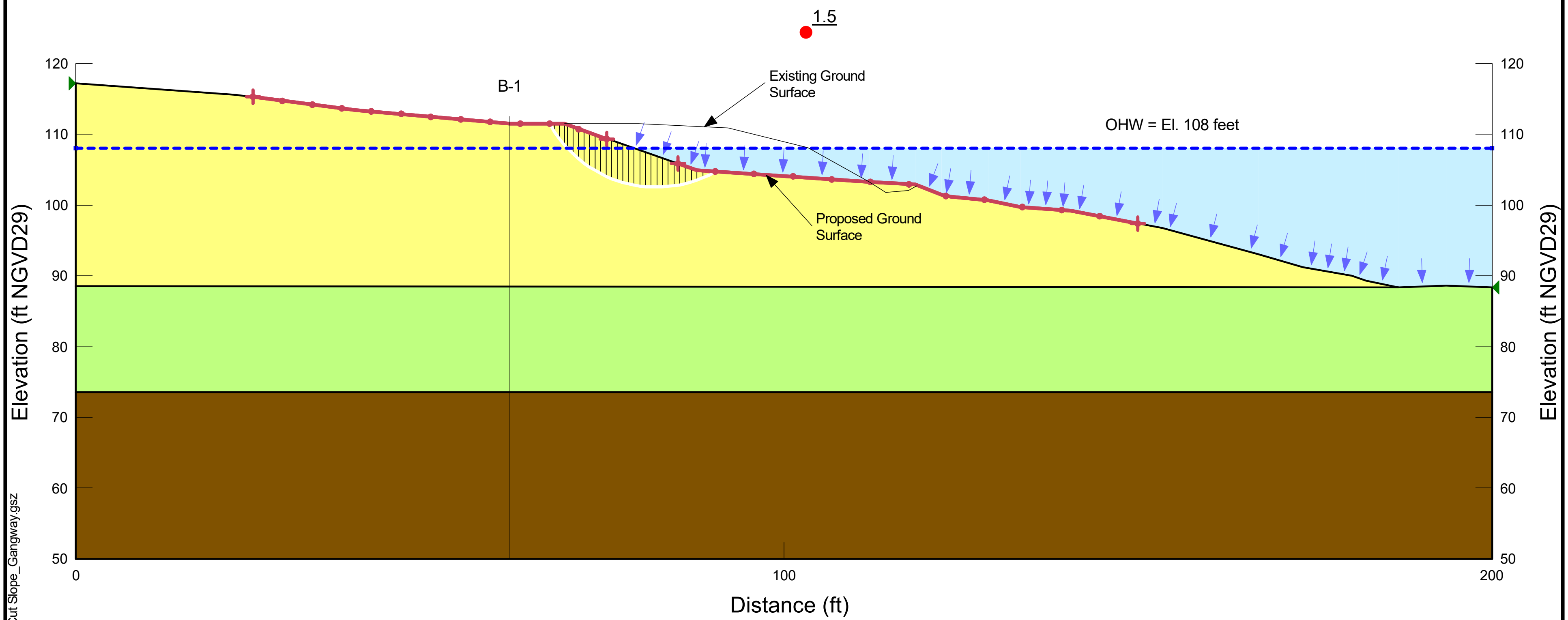

Color	Name	Material Model	Unit Weight (pcf)	Cohesion (psf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	01 Alluv/Marsh Deposits	Mohr-Coulomb	110		0	26
	02 MFDFG	Mohr-Coulomb	110		0	29
	03 Hillsboro Form	Undrained (Phi=0)	120	4,000		

NOTES

1. Critical failure surface estimated using the entry and exit search criteria and the Morgenstern and Price (1965) analysis method.
2. See report text for additional information about analyses and assumptions.

Cook Park Accessible Boat Launch Project
Tigard, Oregon

GLOBAL STABILITY ANALYSIS
GANGWAY CUT SLOPE
STATIC CONDITION

January 2024 112406

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 3

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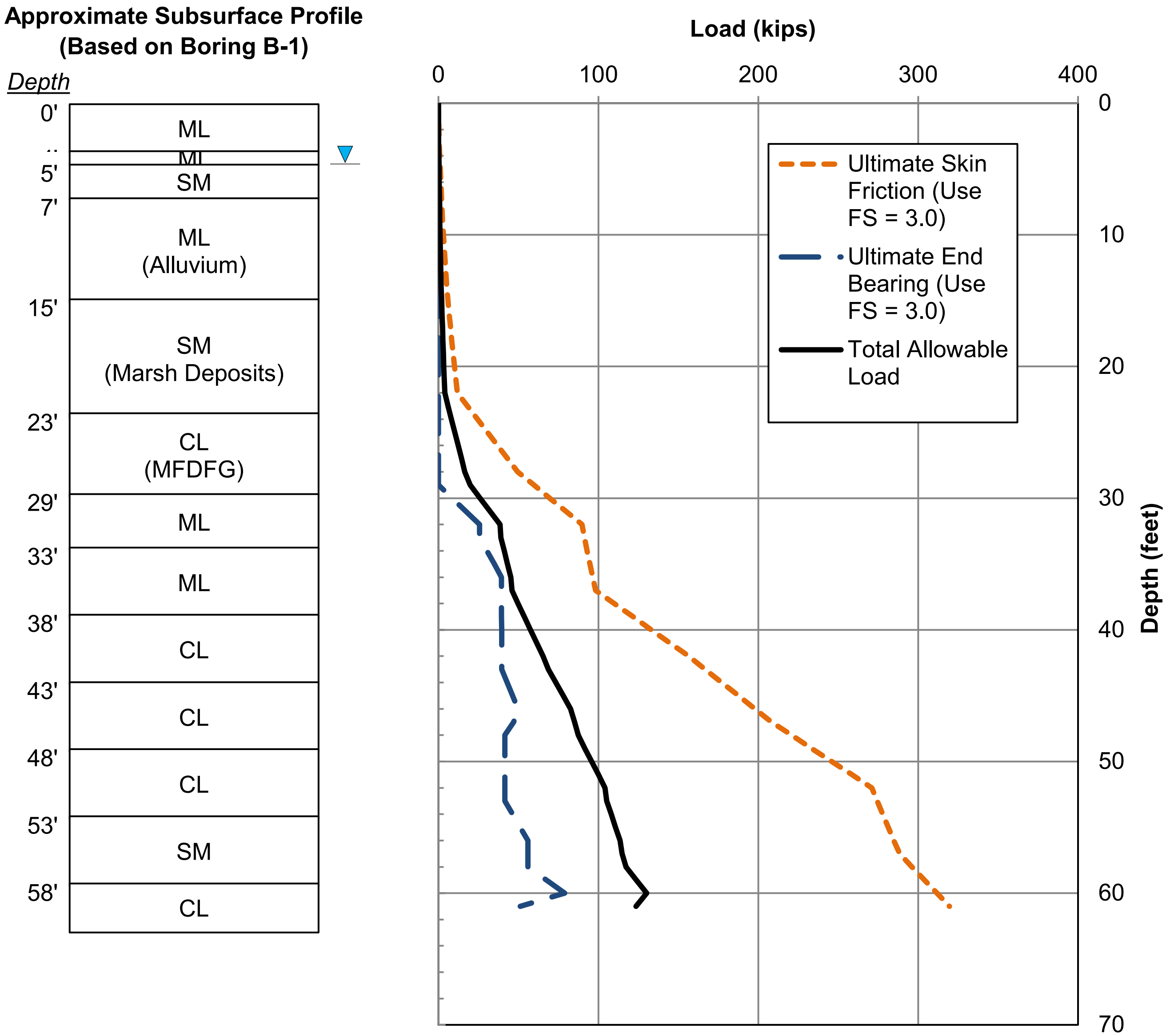

NOTES:

1. We performed the analyses based on guidelines in the OSSC and our local experience. The analyses consider a single pile, not group action of closely spaced piles (closer than 2.5 diameters, center to center).

2. Total pile capacity is a summation of its skin friction and end bearing. Ultimate capacity shown on plots above are to be divided by the a FS = 3.0 for skin friction and end bearing to calculate the total allowable load for static conditions.

3. Per the ODOT GDM (ODOT, 2019), we did not consider potential liquefaction below a depth of 75 feet.

4. We estimate an unfactored downdrag load of 4 kips due to liquefaction.

ABBREVIATIONS:

ODOT = Oregon Department of Transportation
OSSC = Oregon Structural Specialty Code
GDM = Geotechnical Design Manual
FS = Factor of Safety

Cook Park Accessible Boat Launch
Tigard, Oregon

ESTIMATED AXIAL RESISTANCE
12-INCH-DIAMETER PIPE PILE

January 2024 | 112406

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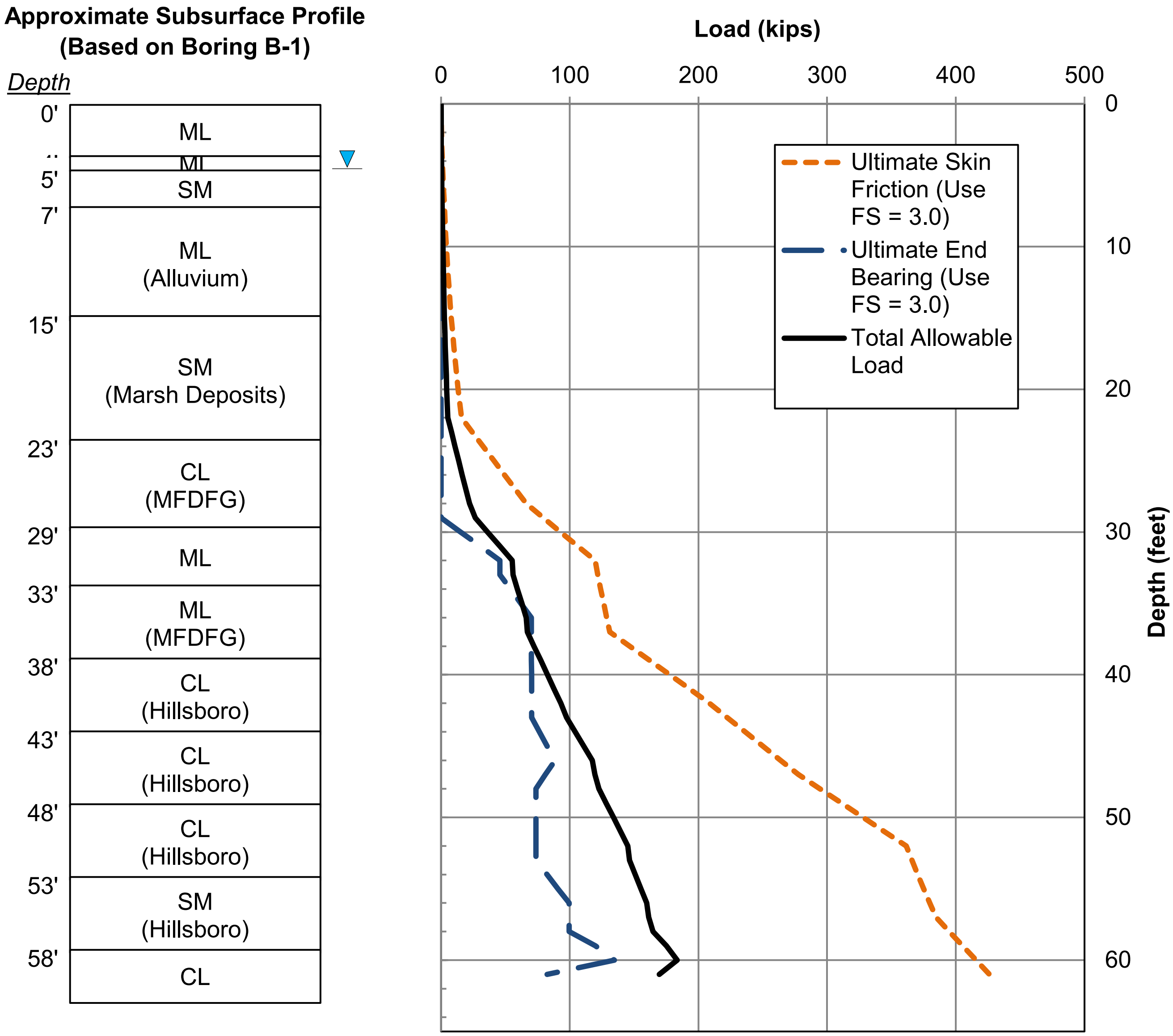
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

FIG. 4

Printed: 12-20-23 13:36 File: P:\PDX\112000s\112406 Tigard - Cook Launch\Analysis\Deep Foundations\[Deep_Foundation_v1.0.0_12OE.xlsm]PILE PLOT UNFAC-KTR



NOTES:

1. We performed the analyses based on guidelines in the OSSC and our local experience. The analyses consider a single pile, not group action of closely spaced piles (closer than 2.5 diameters, center to center).

2. Total pile capacity is a summation of its skin friction and end bearing. Ultimate capacity shown on plots above are to be divided by the a FS = 3.0 for skin friction and end bearing to calculate the total allowable load for static conditions.

3. Per the ODOT GDM (ODOT, 2019), we did not consider potential liquefaction below a depth of 75 feet.

4. We estimate an unfactored downdrag load of 5 kips due to liquefaction.

ABBREVIATIONS:

ODOT = Oregon Department of Transportation
OSSC = Oregon Structural Specialty Code
GDM = Geotechnical Design Manual
FS = Factor of Safety

Cook Park Accessible Boat Launch
Tigard, Oregon

ESTIMATED AXIAL RESISTANCE
16-INCH-DIAMETER PIPE PILE

January 2024 | 112406

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GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

FIG. 5

Printed: 12-21-23 14:43 File: P:\PDX\112000s\112406 Tigard - Cook Launch\Analysis\Deep Foundations\[Deep_Foundation_v1.0.0_16OE.xlsm]SOIL INPUT-KTR

APPENDIX A: FIELD EXPLORATIONS

Appendix A

Field Explorations

CONTENTS

Figures

APPENDIX A: FIELD EXPLORATIONS

A.1 GENERAL

The field exploration program for this project consisted of performing a single geotechnical boring designated B-1. The approximate location of the completed boring was measured in the field and is shown on the Site and Exploration Plan, Figure 2. A Shannon & Wilson geologist was present during the drilling of the geotechnical boring to locate the drilling site, log the material encountered, and collect disturbed and undisturbed soil samples.

This appendix describes the techniques used to advance and sample the boring and presents a log of the materials encountered.

A.2 GEOTECHNICAL BORINGS

The geotechnical boring was drilled and sampled on November 10, 2023 by Western States Soil Conservation Inc. out of Hubbard, Oregon using a track mounted CME-550 drilling rig.

A.2.1 Disturbed Sampling

Disturbed samples were collected in the boring, typically at 2.5- to 5-foot depth intervals, using a standard 2-inch-outside-diameter (O.D.) split spoon sampler in conjunction with Standard Penetration Testing. In a Standard Penetration Test (SPT), ASTM D1586, the sampler is driven 18 inches into the soil using a 140-pound hammer dropped 30 inches. The number of blows required to drive the sampler the last 12 inches is defined as the standard penetration resistance, or N-value. The SPT N-value provides a measure of in situ relative density of cohesionless soils (silt, sand, and gravel), and the consistency of cohesive soils (silt and clay). All disturbed samples were visually identified and described in the field, sealed to retain moisture, and returned to our laboratory for additional examination.

SPT N-values can be significantly affected by several factors, including the efficiency of the hammer used. Automatic hammers generally have higher energy transfer efficiencies than cathead-driven (manual) hammers. For reference, cathead hammers are typically assumed to have an average energy efficiency of 60 percent. All N-values presented in this report are in blows per foot, as counted in the field. No corrections of any kind have been applied.

A.2.2 Undisturbed Sampling

Undisturbed samples were collected in a 3-inch O.D. thin-wall Shelby tubes which was hydraulically pushed into the undisturbed soil at the bottoms of boreholes. The soil exposed at the end of the tube was examined and described in the field. After examination, the ends of the tube were sealed to preserve the natural moisture of the samples. The sealed tube was stored in the upright position and care was taken to avoid shock and vibration during its transport and storage in our laboratory.

A.3 MATERIAL DESCRIPTIONS

In the field, soil samples were identified visually in general accordance with ASTM D2488 Standard Practice for Description and Identification of Soils (Visual-Manual Procedure). Consistency, color, relative moisture, degree of plasticity, peculiar odors and other distinguishing characteristics of the samples were noted. Once returned to the laboratory, soil samples were re-examined, and field identifications were modified as necessary. We refined our visual-manual soil identifications based on additional observation using elements of the Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System), ASTM D2487. The specific terminology used in the soil identifications is defined on the Soil Description and Log Key, Figure A1.

A.4 LOGS OF BORINGS

A summary log of the boring is presented in the Log of Boring B-1, Figure A2. Material descriptions and interfaces on the logs are interpretive, and actual changes may be gradual. The left-hand portions of the logs show individual sample intervals, percent recovery, SPT data, and natural moisture content measurements. Material descriptions and geotechnical unit designations are shown in the center of the boring logs, and right-hand portion of the boring logs shows a graphic log, sample locations and designations, backfill details, and a graphical representation of N-values, natural water contents, Atterberg limits, and sample recovery.

A.5 BOREHOLE ABANDONMENT

The boring was backfilled with bentonite chips in accordance with Oregon Water Resource Department regulations, up to a depth of approximately 1 foot. Native soil was used as backfill from approximately 1 foot up to the ground surface.

Shannon & Wilson, Inc. (S&W), uses a soil identification system modified from the Unified Soil Classification System (USCS). Elements of the USCS and other definitions are provided on this and the following pages. Soil descriptions are based on visual-manual procedures (ASTM D2488) and laboratory testing procedures (ASTM D2487), if performed.

S&W INORGANIC SOIL CONSTITUENT DEFINITIONS

CONSTITUENT[2]	FINE-GRAINED SOILS (50% or more fines)[1]	COARSE-GRAINED SOILS (less than 50% fines)[1]
Major	***Silt, Lean Clay, Elastic Silt,*** or ***Fat Clay***[3]	***Sand*** or ***Gravel***[4]
Modifying (Secondary) Precedes major constituent	30% or more coarse-grained: ***Sandy*** or ***Gravelly***[4]	More than 12% fine-grained: ***Silty*** or ***Clayey***[3]
Minor Follows major constituent	15% to 30% coarse-grained: ***with Sand*** or ***with Gravel***[4]	5% to 12% fine-grained: ***with Silt*** or ***with Clay***[3]
	30% or more total coarse-grained *and* lesser coarse-grained constituent is 15% or more: ***with Sand*** or ***with Gravel***[5]	15% or more of a second coarse-grained constituent: ***with Sand*** or ***with Gravel***[5]

[1]All percentages are by weight of total specimen passing a 3-inch sieve.
[2]The order of terms is: *Modifying Major with Minor*.
[3]Determined based on behavior.
[4]Determined based on which constituent comprises a larger percentage.
[5]Whichever is the lesser constituent.

MOISTURE CONTENT TERMS

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

STANDARD PENETRATION TEST (SPT) SPECIFICATIONS

Hammer:	140 pounds with a 30-inch free fall. Rope on 6- to 10-inch-diam. cathead 2-1/4 rope turns, > 100 rpm
Sampler:	10 to 30 inches long Shoe I.D. = 1.375 inches Barrel I.D. = 1.5 inches Barrel O.D. = 2 inches
N-Value:	Sum blow counts for second and third 6-inch increments. Refusal: 50 blows for 6 inches or less; 10 blows for 0 inches.

NOTE: Penetration resistances (N-values) shown on boring logs are as recorded in the field and have not been corrected for hammer efficiency, overburden, or other factors.

PARTICLE SIZE DEFINITIONS

DESCRIPTION	SIEVE NUMBER AND/OR APPROXIMATE SIZE
FINES	< #200 (0.075 mm = 0.003 in.)
SAND	
Fine	#200 to #40 (0.075 to 0.4 mm; 0.003 to 0.02 in.)
Medium	#40 to #10 (0.4 to 2 mm; 0.02 to 0.08 in.)
Coarse	#10 to #4 (2 to 4.75 mm; 0.08 to 0.187 in.)
GRAVEL	
Fine	#4 to 3/4 in. (4.75 to 19 mm; 0.187 to 0.75 in.)
Coarse	3/4 to 3 in. (19 to 76 mm)
COBBLES	3 to 12 in. (76 to 305 mm)
BOULDERS	> 12 in. (305 mm)

RELATIVE DENSITY / CONSISTENCY

COHESIONLESS SOILS

N, SPT, BLOWS/FT.	RELATIVE DENSITY
< 4	Very loose
4 - 10	Loose
10 - 30	Medium dense
30 - 50	Dense
> 50	Very dense

COHESIVE SOILS

N, SPT, BLOWS/FT.	RELATIVE CONSISTENCY
< 2	Very soft
2 - 4	Soft
4 - 8	Medium stiff
8 - 15	Stiff
15 - 30	Very stiff
> 30	Hard

WELL AND BACKFILL SYMBOLS

- Bentonite Cement Grout
- Bentonite Grout
- Bentonite Chips
- Silica Sand
- Gravel
- Perforated or Screened Casing
- Surface Cement Seal
- Asphalt or Cap
- Slough
- Inclinometer or Non-perforated Casing
- Vibrating Wire Piezometer

PERCENTAGES TERMS [1, 2]

Trace	< 5%
Few	5 to 10%
Little	15 to 25%
Some	30 to 45%
Mostly	50 to 100%

[1]Gravel, sand, and fines estimated by mass. Other constituents, such as organics, cobbles, and boulders, estimated by volume.

[2]Reprinted, with permission, from ASTM D2488 - 09a Standard Practice for Description and Identification of Soils (Visual-Manual Procedure), copyright ASTM International, 100 Barr Harbor Drive, West Conshohocken, PA 19428. A copy of the complete standard may be obtained from ASTM International, www.astm.org.

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SOIL DESCRIPTION AND LOG KEY

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FIG. A1
Sheet 1 of 3

UNIFIED SOIL CLASSIFICATION SYSTEM (USCS)
(Modified From USACE Tech Memo 3-357, ASTM D2487, and ASTM D2488)

MAJOR DIVISIONS			GROUP/GRAPHIC SYMBOL	TYPICAL IDENTIFICATIONS
COARSE-GRAINED SOILS *(more than 50% retained on No. 200 sieve)*	Gravels *(more than 50% of coarse fraction retained on No. 4 sieve)*	Gravel *(less than 5% fines)*	GW	Well-Graded Gravel; Well-Graded Gravel with Sand
			GP	Poorly Graded Gravel; Poorly Graded Gravel with Sand
		Silty or Clayey Gravel *(more than 12% fines)*	GM	Silty Gravel; Silty Gravel with Sand
			GC	Clayey Gravel; Clayey Gravel with Sand
	Sands *(50% or more of coarse fraction passes the No. 4 sieve)*	Sand *(less than 5% fines)*	SW	Well-Graded Sand; Well-Graded Sand with Gravel
			SP	Poorly Graded Sand; Poorly Graded Sand with Gravel
		Silty or Clayey Sand *(more than 12% fines)*	SM	Silty Sand; Silty Sand with Gravel
			SC	Clayey Sand; Clayey Sand with Gravel
FINE-GRAINED SOILS *(50% or more passes the No. 200 sieve)*	Silts and Clays *(liquid limit less than 50)*	Inorganic	ML	Silt; Silt with Sand or Gravel; Sandy or Gravelly Silt
			CL	Lean Clay; Lean Clay with Sand or Gravel; Sandy or Gravelly Lean Clay
		Organic	OL	Organic Silt or Clay; Organic Silt or Clay with Sand or Gravel; Sandy or Gravelly Organic Silt or Clay
	Silts and Clays *(liquid limit 50 or more)*	Inorganic	MH	Elastic Silt; Elastic Silt with Sand or Gravel; Sandy or Gravelly Elastic Silt
			CH	Fat Clay; Fat Clay with Sand or Gravel; Sandy or Gravelly Fat Clay
		Organic	OH	Organic Silt or Clay; Organic Silt or Clay with Sand or Gravel; Sandy or Gravelly Organic Silt or Clay
HIGHLY-ORGANIC SOILS	Primarily organic matter, dark in color, and organic odor		PT	Peat or other highly organic soils (see ASTM D4427)

FILL	Placed by humans, both engineered and nonengineered. May include various soil materials and debris.		The Fill graphic symbol is combined with the soil graphic that best represents the observed material

NOTE: No. 4 size = 4.75 mm = 0.187 in.; No. 200 size = 0.075 mm = 0.003 in.

NOTES

1. Dual symbols *(symbols separated by a hyphen, i.e., SP-SM, Sand with Silt)* are used for soils with between 5% and 12% fines or when the liquid limit and plasticity index values plot in the *CL-ML* area of the plasticity chart.
2. Borderline symbols *(symbols separated by a slash, i.e., CL/ML, Lean Clay to Silt; SP-SM/SM, Sand with Silt to Silty Sand)* indicate that the soil properties are close to the defining boundary between two groups.
3. The soil graphics above represent the various USCS identifications (i.e., *GP*, *SM*, etc.) and may be augmented with additional symbology to represent differences within USCS designations. *Sandy Silt (ML)*, for example, may be accompanied by the *ML* soil graphic with sand grains added. Non-USCS materials may be represented by other graphic symbols; see log for descriptions.

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SOIL DESCRIPTION AND LOG KEY

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FIG. A1
Sheet 2 of 3

GRADATION TERMS

Poorly Graded	Narrow range of grain sizes present or, within the range of grain sizes present, one or more sizes are missing (Gap Graded). Meets criteria in ASTM D2487, if tested.
Well-Graded	Full range and even distribution of grain sizes present. Meets criteria in ASTM D2487, if tested.

CEMENTATION TERMS[1]

Weak	Crumbles or breaks with handling or slight finger pressure
Moderate	Crumbles or breaks with considerable finger pressure
Strong	Will not crumble or break with finger pressure

PLASTICITY[2]

DESCRIPTION	VISUAL-MANUAL CRITERIA	APPROX. PLASITICTY INDEX RANGE
Nonplastic	A 1/8-in. thread cannot be rolled at any water content.	< 4%
Low	A thread can barely be rolled and a lump cannot be formed when drier than the plastic limit.	4 to 10%
Medium	A thread is easy to roll and not much time is required to reach the plastic limit. The thread cannot be rerolled after reaching the plastic limit. A lump crumbles when drier than the plastic limit.	10 to 20%
High	It take considerable time rolling and kneading to reach the plastic limit. A thread can be rerolled several times after reaching the plastic limit. A lump can be formed without crumbling when drier than the plastic limit.	> 20%

ADDITIONAL TERMS

Mottled	Irregular patches of different colors.
Bioturbated	Soil disturbance or mixing by plants or animals.
Diamict	Nonsorted sediment; sand and gravel in silt and/or clay matrix.
Cuttings	Material brought to surface by drilling.
Slough	Material that caved from sides of borehole.
Sheared	Disturbed texture, mix of strengths.

PARTICLE ANGULARITY AND SHAPE TERMS[1]

Angular	Sharp edges and unpolished planar surfaces.
Subangular	Similar to angular, but with rounded edges.
Subrounded	Nearly planar sides with well-rounded edges.
Rounded	Smoothly curved sides with no edges.
Flat	Width/thickness ratio > 3.
Elongated	Length/width ratio > 3.

ACRONYMS AND ABBREVIATIONS

ATD	At Time of Drilling
approx.	Approximate/Approximately
Diam.	Diameter
Elev.	Elevation
ft.	Feet
FeO	Iron Oxide
gal.	Gallons
Horiz.	Horizontal
HSA	Hollow Stem Auger
I.D.	Inside Diameter
in.	Inches
lbs.	Pounds
MgO	Magnesium Oxide
mm	Millimeter
MnO	Manganese Oxide
NA	Not Applicable or Not Available
NP	Nonplastic
O.D.	Outside Diameter
OW	Observation Well
pcf	Pounds per Cubic Foot
PID	Photo-Ionization Detector
PMT	Pressuremeter Test
ppm	Parts per Million
psi	Pounds per Square Inch
PVC	Polyvinyl Chloride
rpm	Rotations per Minute
SPT	Standard Penetration Test
USCS	Unified Soil Classification System
q_u	Unconfined Compressive Strength
VWP	Vibrating Wire Piezometer
Vert.	Vertical
WOH	Weight of Hammer
WOR	Weight of Rods
Wt.	Weight

STRUCTURE TERMS[1]

Interbedded	Alternating layers of varying material or color with layers at least 1/4-inch thick; singular: bed.
Laminated	Alternating layers of varying material or color with layers less than 1/4-inch thick; singular: lamination.
Fissured	Breaks along definite planes or fractures with little resistance.
Slickensided	Fracture planes appear polished or glossy; sometimes striated.
Blocky	Cohesive soil that can be broken down into small angular lumps that resist further breakdown.
Lensed	Inclusion of small pockets of different soils, such as small lenses of sand scattered through a mass of clay.
Homogeneous	Same color and appearance throughout.

[1]Reprinted, with permission, from ASTM D2488 - 09a Standard Practice for Description and Identification of Soils (Visual-Manual Procedure), copyright ASTM International, 100 Barr Harbor Drive, West Conshohocken, PA 19428. A copy of the complete standard may be obtained from ASTM International, www.astm.org.
[2]Adapted, with permission, from ASTM D2488 - 09a Standard Practice for Description and Identification of Soils (Visual-Manual Procedure), copyright ASTM International, 100 Barr Harbor Drive, West Conshohocken, PA 19428. A copy of the complete standard may be obtained from ASTM International, www.astm.org.

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SOIL DESCRIPTION AND LOG KEY

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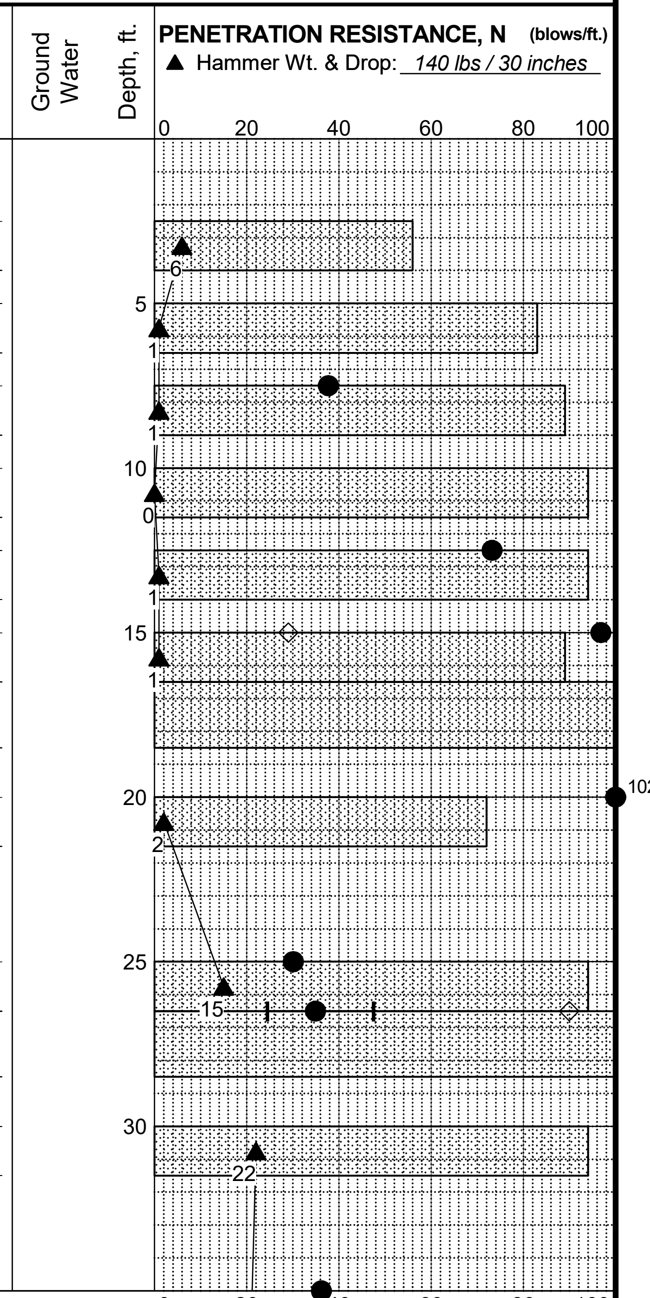
FIG. A1
Sheet 3 of 3

Total Depth:	61.5 ft.	Northing:	~ 125,554 ft.	Drilling Method:	Mud Rotary	Hole Diam.:	5" in.
Top Elevation:	~ 111 ft.	Easting:	~ 322,559 ft.	Drilling Company:	Western States	Rod Type:	NWJ
Vert. Datum:	NGVD29	Station:	~	Drill Rig Equipment:	CME-550 Track Rig #2	Hammer Type:	Automatic
Horiz. Datum:	OCRS-Portland	Offset:	~	Other Comments:			

SOIL DESCRIPTION *Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.*	Elev. Depth (ft.)	Samples
TOPSOIL	*111.0* 0.5	
Loose, brown, *Sandy Silt (ML)*; moist; fine to medium sand; nonplastic; trace organics; micaceous.	*107.0* 4.5	S-1
Very loose, brown, *Silty Sand (SM)*; moist; fine to medium sand; nonplastic fines; trace organics; micaceous.	*104.5* 7.0	S-2
Very soft, gray, *Sandy Silt to Sandy Elastic Silt (ML/MH)*; moist; fine sand; low plasticity; trace organics; micaceous. **ALLUVIUM**	*97.0* 14.5	S-3, S-4, S-5
Very loose, gray and brown, *Silty Sand (SM)*; moist; fine to coarse sand; nonplastic to low plasticity fines; some to mostly organics; micaceous. **MARSH DEPOSITS**	*88.5* 23.0	S-6, S-7, S-8
Stiff to very stiff, gray, *Lean Clay (CL)*; moist; fine sand; medium to high plasticity; trace organics; micaceous. **FINE-GRAINED MISSOULA FLOOD DEPOSITS**	*82.5* 29.0	S-9, S-10
Very stiff, gray and brown, *Sandy Silt (ML)*; moist; fine to medium sand; low plasticity; micaceous; iron oxidation and staining.	*78.5* 33.0	S-11
Medium dense, gray, *Sandy Silt (ML)*; moist; fine to medium sand; nonplastic; micaceous.		



CONTINUED NEXT SHEET

LEGEND

Standard Penetration Test
3" O.D. Shelby Tube

Recovery (%)
% Fines (<0.075mm)
% Water Content
Plastic Limit — Liquid Limit

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations, and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. Group symbol is based on visual-manual identification and selected lab testing.
4. The hole location and elevation should be considered approximate.

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LOG OF BORING B-1

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FIG. A2
Sheet 1 of 2

REV 2

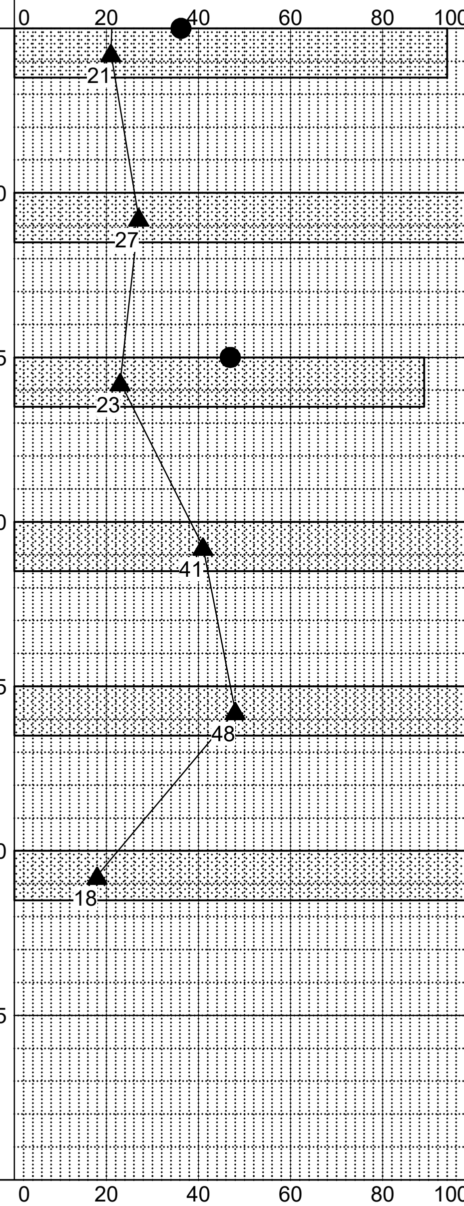
MASTER_LOG_E 112406.GPJ SW2013LIBRARYPDX.GLB SHANWIL_PDX.GDT 12/8/23 Log: GAF Rev: DSJ Typ: DSJ

Total Depth:	61.5 ft.	Northing:	~ 125,554 ft.	Drilling Method:	Mud Rotary
Top Elevation:	~ 111 ft.	Easting:	~ 322,559 ft.	Drilling Company:	Western States
Vert. Datum:	NGVD29	Station:	~	Drill Rig Equipment:	CME-550 Track Rig #2
Horiz. Datum:	OCRS-Portland	Offset:	~	Other Comments:	

Hole Diam.:	5" in.
Rod Type:	NWJ
Hammer Type:	Automatic

SOIL DESCRIPTION *Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.*	*Elev.* Depth (ft.)	Samples
Continued: Medium dense, gray, *Sandy Silt (ML)*; moist; fine to medium sand; nonplastic; micaceous.	*73.5* 38.0	S-12
FINE-GRAINED MISSOULA FLOOD DEPOSITS Very stiff, gray and brown, *Lean Clay with Sand (CL)*; moist; trace fine subangular gravel; fine to coarse sand; medium to high plasticity; micaceous; iron oxidation and staining.	*68.5* 43.0	S-13
Very stiff to hard, gray and brown, *Lean Clay with Sand (CL)*; moist; fine sand; medium to high plasticity; micaceous; iron oxidation and staining. **HILLSBORO FORMATION**	*58.5* 53.0	S-14, S-15
Dense, brown, *Silty Sand (SM)*; moist; fine to coarse sand; nonplastic to low plasticity fines.	*53.5* 58.0	S-16
Very stiff, brown and gray, *Lean Clay (CL)*; moist; trace fine sand; medium plasticity; micaceous.	*50.0* 61.5	S-17
Completed: November 10, 2023		

PENETRATION RESISTANCE, N (blows/ft.)
▲ Hammer Wt. & Drop: 140 lbs / 30 inches



LEGEND

- Standard Penetration Test
- 3" O.D. Shelby Tube

- Recovery (%)
- ◇ % Fines (<0.075mm)
- ● % Water Content
- Plastic Limit ├──┤ Liquid Limit

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations, and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. Group symbol is based on visual-manual identification and selected lab testing.
4. The hole location and elevation should be considered approximate.

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LOG OF BORING B-1

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FIG. A2
Sheet 2 of 2

APPENDIX B: LABORATORY TEST RESULTS

Appendix B
Laboratory Test Results

CONTENTS

Figures

APPENDIX B: LABORATORY TESTING RESULTS

B.1 GENERAL

Soil samples obtained during the field explorations were described and identified in the field in general accordance with the Standard Practice for Description and Identification of Soils (Visual-Manual Procedure), ASTM D2488. The specific terminology used is presented on Appendix A, Figure A1. The samples were reviewed in the Shannon & Wilson laboratory. The physical characteristics of the samples were noted, and the field descriptions and identifications were modified where necessary in accordance with terminology presented in Appendix A, Figure A1. Representative samples were selected for various laboratory tests. We refined our visual-manual soil descriptions and identifications based on the results of the laboratory tests, using elements of the Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System), ASTM D2487. The refined descriptions and identifications were then incorporated into the Log of Boring, presented in Appendix A. Note that ASTM D2487 was not followed in full because it requires that a suite of tests be performed to fully classify a single sample.

The soil testing program included moisture content, particle size analyses, and Atterberg limits. Laboratory testing was performed by Shannon & Wilson. All test procedures were performed in accordance with applicable ASTM International standards. Test procedures are summarized in the following paragraphs.

B.2 SOIL TESTING

B.2.1 Moisture (Natural Water) Content

Natural moisture content determinations were performed in accordance with ASTM D2216, on selected soil samples. The natural moisture content is a measure of the amount of moisture in the soil at the time of exploration. It is defined as the ratio of the weight of water to the dry weight of the soil, expressed as a percentage. The results of moisture content determinations are presented on the Boring Log in Appendix A.

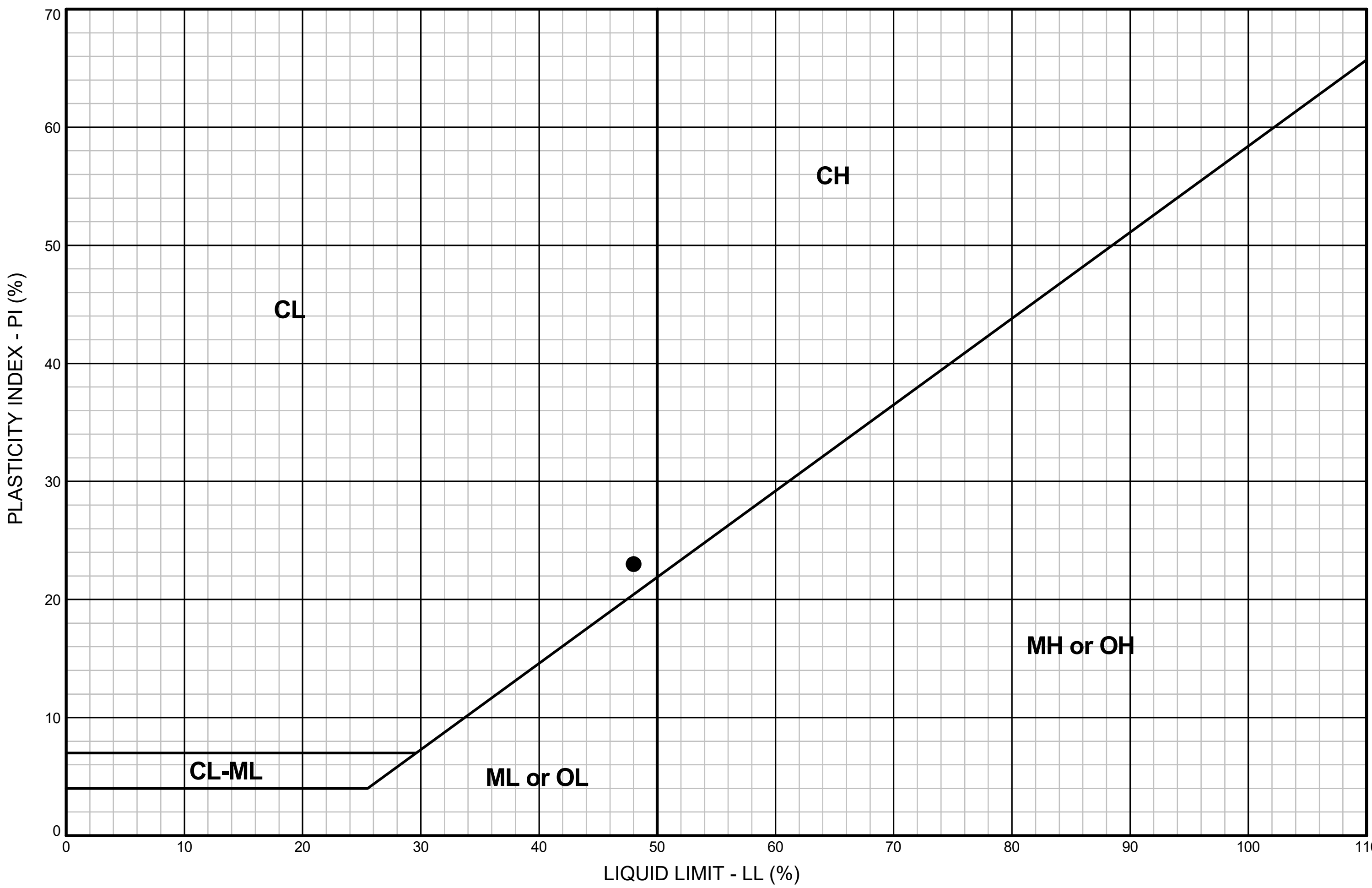
B.2.2 Atterberg Limits

Atterberg limits were determined for select samples in accordance with ASTM D4318. This analysis yields index parameters of the soil that are useful in soil identification, as well as in a number of analyses, including liquefaction analysis. An Atterberg limits test determines a soil's liquid limit (LL) and plastic limit (PL). These are the maximum and minimum moisture contents at which the soil exhibits plastic behavior. A soil's plasticity index (PI) can be determined by subtracting PL from LL. The LL, PL, and PI of tested samples are presented on Figure B1, Atterberg Limits Results. The results are also presented on the Drill Logs in Appendix A.

For the purposes of soil description, the ODOT Soil and Rock Classification Manual (1987) uses the term nonplastic to refer to soils with a PI less than 3, low plasticity for soils with a PI range of 3 to 15, medium plasticity for soils with a PI range of 15 to 30, and high plasticity for soils with a PI greater than 30.

B.2.3 Particle-Size Analyses

Particle-size analyses was conducted on a single sample to determine the grain-size distributions. Grain size distributions were determined in accordance with ASTM D6913 and D1140 as applicable. A wet sieve analysis was performed to determine the percentage (by weight) of each sample passing the No. 200 (0.075 mm) sieve. The material retained on the No. 200 sieve was then shaken through a series of sieves to determine the distribution of the plus No. 200 fraction. Hydrometer testing (ASTM D422) was used to identify the amount of silt and clay present. Results of all particle-size analyses are presented on Figure B1, Grain Size Distribution. The resulting percent fine-grained constituents are also presented on the Logs of Borings in Appendix A, and in Figure B2, Grain Size Distribution.



NOTES

1) Atterberg limits tests were performed in general accordance with ASTM D4318 unless otherwise noted in the report.

2) Group Name and Group Symbol are in accordance with ASTM D2488 and are refined in accordance with ASTM D2487 where appropriate laboratory tests are performed.

3) Plasticity adjectives used in sample descriptions correspond to plasticity index as follows:
- Nonplastic (NP) (< 4%)
- Low Plasticity (4 to 10%)
- Medium Plasticity (10 to 20%)
- High Plasticity (> 20%)

BORING AND SAMPLE NO.	DEPTH (feet)	GROUP SYMBOL[2]	GROUP NAME[2]	LL %	PL %	PI %[3]	NAT. W.C. %	FINES %
● B-1, S-10	26.5			48	25	23	35	90

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ATTERBERG LIMITS RESULTS

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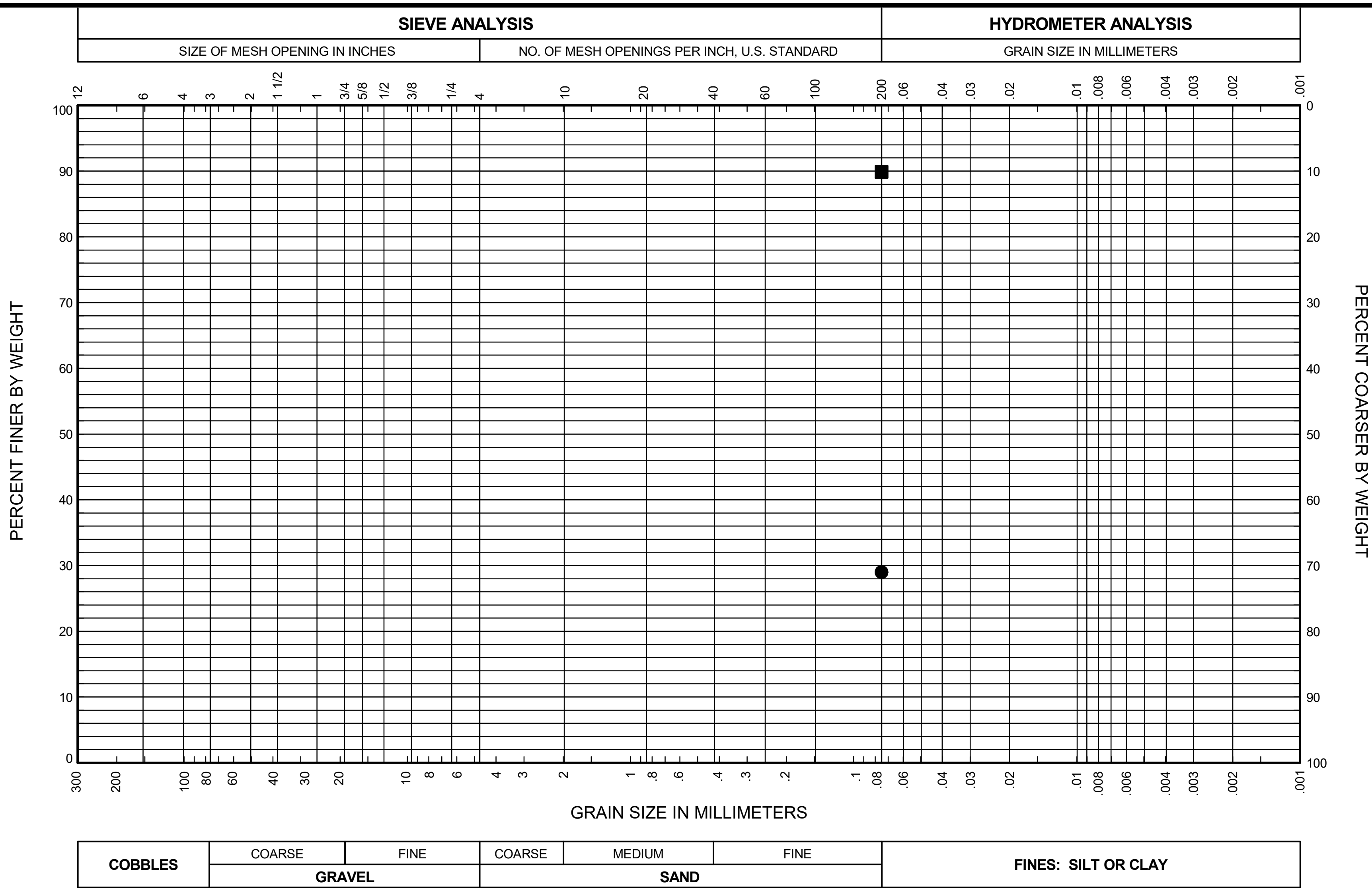
FIG. B1

FIG. B1



BORING AND SAMPLE NO.	DEPTH (feet)	GROUP SYMBOL[2]	GROUP NAME[2]	GRAVEL %	SAND %	FINES %	NAT. W.C. %	DRY DENSITY PCF
● B-1, S-6	15.0			-	-	29	97	
■ B-1, S-10	26.5			-	-	90	35	

NOTES:
1) Sieve analyses were performed in general accordance with ASTM D6913, sieve with hydrometer analyses were performed in general accordance with ASTM D422, and amount finer than #200 sieve analyses were performed in general accordance with ASTM D1140 unless otherwise noted in the report.
2) Group Name and Group Symbol are in accordance with ASTM D2488 and are refined in accordance with ASTM D2487 where appropriate laboratory tests are performed.

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GRAIN SIZE DISTRIBUTION

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FIG. B2

GSA MAIN COLOR 112406.GPJ SHAN WIL.GDT 12/20/23

Important Information

About Your Geotechnical/Environmental Report

IMPORTANT INFORMATION

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

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

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

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

SUBSURFACE CONDITIONS CAN CHANGE.

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

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

MOST RECOMMENDATIONS ARE PROFESSIONAL JUDGMENTS.

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

A REPORT'S CONCLUSIONS ARE PRELIMINARY.

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

THE CONSULTANT'S REPORT IS SUBJECT TO MISINTERPRETATION.

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

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

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

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

READ RESPONSIBILITY CLAUSES CLOSELY.

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

The preceding paragraphs are based on information provided by the Geoprofessional Business Association (https://www.geoprofessional.org)