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GEOTECHNICAL ENGINEERING REPORT

I-5 Pedestrian Bridge: Barber St. to Wilsonville Town Center

WILSONVILLE, OREGON

SHANNON & WILSON

December 2020

Shannon & Wilson No: 103953

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Submitted To: DOWL, LLC
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Attn: Bob Goodrich, PE

Subject: DRAFT GEOTECHNICAL ENGINEERING REPORT, I-5 PEDESTRIAN BRIDGE: BARBER ST. TO WILSONVILLE TOWN CENTER, WILSONVILLE, OREGON

Shannon & Wilson prepared this report and participated in this project as a subconsultant to DOWL, LLC. Our scope of services was specified in Task Order No. 29, dated December 26, 2019. This report presents the results of our field explorations, 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 we may be of further service, please contact us.

Sincerely,

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Important Information

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AASHTO	American Association of State Highway and Transportation Officials
A_s	Site Peak Ground Acceleration (F_{pga} x PGA)
ASTM	American Society for Testing and Materials
bgs	below ground surface
bpf	blows per foot
CRBG	Columbia River Basalt Group
CSL	Crosshole Sonic Logging
CSZ	Cascadia Subduction Zone
CSZE	Cascadia Subduction Zone Earthquake
El	Elevation
FHWA	Federal Highway Administration
F_{pga}	Zero-Period Site Coefficient
fps	feet per second
FS	Factor of Safety
F_y	Yield Strength
g	acceleration due to gravity
GDM	Geotechnical Design Manual
GPS	Geographic Positioning System
H	Horizontal
ka	"Kilo-annum" or one thousand years ago
k_{AE}	Seismic active earth pressure coefficient
k_h	horizontal acceleration coefficient
k_v	vertical acceleration coefficient
LRFD	Load and Resistance Factor Design
m/sec	meters per second
Ma	"Mega-annum" or million years ago
mm	millimeters
Mw	Moment Magnitude
NAVD88	North American Vertical Datum of 1988
NB	Northbound
ODOT	Oregon Department of Transportation
OSHA	Occupational Safety and Health Administration
OSSC	Oregon Standard Specifications for Construction
pcf	pounds per cubic foot
PGA	Peak Ground Acceleration
psf	pounds per square foot
psi	pounds per square inch
S_a	Spectral Acceleration
SB	Southbound

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SPT	Standard Penetration Test
Sta	Station
V	Vertical
V_{s30}	average shear wave velocity in the upper 30 meters of soil
USGS	United States Geological Survey
yr	year

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1 INTRODUCTION

This report presents the results of our field explorations, laboratory testing, geotechnical design evaluations and recommendations, and construction considerations for the proposed Interstate 5 (I-5) Pedestrian Bridge: Barber St to Wilsonville Town Center project in Wilsonville, Oregon. The City of Wilsonville, along with their engineering consultant DOWL, LLC (DOWL), is planning to construct a pedestrian bridge connecting Barber Street on the west side of I-5 to the Wilsonville Town Center on the east side of I-5. The proposed bridge will cross over Boones Ferry Road, I-5, and Town Center Loop West. The general location of the project site is shown on the Vicinity Map, Figure 1. As a subconsultant to DOWL, Shannon & Wilson, Inc. (Shannon & Wilson), is providing geotechnical services to support engineering design for the project.

2 PROJECT UNDERSTANDING

2.1 Site Description

The proposed I-5 Pedestrian Bridge is located approximately 0.3 miles north of the I-5 and Southwest (SW) Wilsonville Road Intersection in Wilsonville, Oregon. The proposed bridge will connect Barber Street to the Wilsonville Town Center. The Willamette River runs west to east approximately 1 mile south of the project site. From west to east, the proposed I-5 Pedestrian Bridge will cross over Boones Ferry Road, I-5 SB, I-5 NB, and Town Center Loop West, including the unpaved medians between each roadway. All four roadways in the project vicinity run in parallel, north to south, however Town Center Loop West begins to curve eastwards north of the project site. West of Boones Ferry Road is a parking area for a Rite Aid Distribution Center. East of Town Center Loop West is an unpaved field surrounded by two parking areas to the south and east, and the newly constructed EyeHealth Northwest Clinic to the north at 29250 Town Center Loop West. In general, the topography in the project area is relatively flat except for a short berm, approximately 5 to 10 feet high and sloped at approximately 2.5H:1V, separating the Rite Aid Distribution Center parking area from Boones Ferry Road. The elevation slightly increases from west to east along the proposed alignment, from approximate elevation 165 feet west of Boones Ferry Road to elevation 175 feet at the median between I-5 NB and SB. The topography dips a few feet east of the I-5 median, before increasing again to an approximate elevation 175 feet east of Town Center Loop West. Existing ground surface contours are shown on the Site and Exploration Plan, Figure 2. All elevations are in North American Vertical Datum of 1988 (NAVD88).

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Exhibit 2-1 through Exhibit 2-3 present site photographs showing several views of the site and existing structures.

Exhibit 2-1: View of Rite Aid Distribution parking area near the proposed I-5 Pedestrian Bridge western approach. Berm separating the parking area from Boones Ferry Road is seen in the right of the picture. Photograph taken facing north.

Exhibit 2-2: View of I-5 median. The field where the I-5 Pedestrian Bridge east approach is proposed is behind the array of trees, and the EyeHealth Northwest clinic is seen in the left of the picture. Photograph taken facing southeast.

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Exhibit 2-3: Field where the I-5 Pedestrian Bridge east approach is proposed. Photograph taken facing southwest.

2.2 Project Description

The proposed project will construct a new, eight-span bridge structure to connect Barber Street to the Wilsonville Town Center. According to preliminary plans, provided by DOWL on November 4, 2020, we understand the proposed bridge will have an abutment to abutment length of 770 feet and a width of about 20 feet. The bridge section crossing Boones Ferry Road, I-5, and Town Center Loop West will run perpendicular to the roadways. West of Boones Ferry Road, the bridge will curve towards the north, terminating on what is now the short berm area between the Rite Aid Distribution Parking Area and Boones Ferry Road. East of Town Center Loop West, the proposed bridge will curve towards the northeast before terminating at the northeast corner of the field. The proposed bridge alignment with approximate bent locations is shown on Figure 2.

We understand the bridge will likely be supported on drilled shafts and spread footings. The interior bents, Bents 2 through 8, are anticipated to be supported on dual 5- or 6.5-foot diameter drilled shafts. The abutments are proposed to be supported on spread footings constructed upon back-to-back Mechanically Stabilized Earth (MSE) walls for the bridge approaches. For the remainder of this report, MSE abutments refers to the preferred alternative of bridge abutment spread footings constructed upon the MSE walls. The West and East abutments are designated Bent 1 and Bent 9, respectively. DOWL provided factored design loads per bent for the proposed I-5 Pedestrian Bridge on November 9, 2020. Exhibit 2-4 presents the provided factored design loads, per bent, at each bent location.

Exhibit 2-4: Estimated Factored Design Loads for I-5 Pedestrian Bridge

		Factored Design Load Per Bent (kips)	
Location	Foundation Configuration	Service Limit	Strength Limit
Bent 1 (West Abutment)	Spread Footing on MSE Wall (MSE Abutment)	135	185
Bent 2	Dual 5- or 6.5-foot-diameter drilled shafts	641	861
Bent 3	Dual 5- or 6.5-foot-diameter drilled shafts	628	843
Bent 4	Dual 6.5-foot-diameter drilled shafts	924	1245
Bent 5	Dual 6.5-foot-diameter drilled shafts	1324	1790
Bent 6	Dual 6.5-foot-diameter drilled shafts	1045	1406
Bent 7	Dual 5- or 6.5-foot-diameter drilled shafts	742	998
Bent 8	Dual 5- or 6.5-foot-diameter drilled shafts	906	1217
Bent 9 (East Abutment)	Spread Footing on MSE Wall (MSE Abutment)	196	268

Based on the preliminary plans provided by DOWL, approach fill heights of approximately 11 feet are anticipated at both abutments. The approach fills will be retained with back-to-back MSE walls tapering to the existing ground surface away from the abutment.

2.3 Scope of Services

Shannon & Wilson's services were conducted in accordance with the scope of services defined in Task Order No. 29, dated December 26, 2019. The completed geotechnical design services for the project consisted of the following tasks:

- Review available existing information and visit the site to observe existing site conditions, geologic hazards, site access for the field explorations, site constraints and staging concerns for construction, and mark proposed exploration locations;
- Develop a field exploration and testing work plan and obtain drilling permits from ODOT and the City of Wilsonville;
- Explore the subsurface conditions with five (5) geotechnical borings and two (2) in-situ infiltration tests, with collection of soil samples in the geotechnical borings;
- Conduct laboratory testing on selected soil samples to characterize soils and develop soil properties for evaluation;
- Develop seismic design response spectrum curves utilizing ODOT's Design Response Spectrum Program;

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- Evaluate the site-specific seismic hazards, including ground motion, liquefaction potential, and other seismic-related hazards, and their effects on the proposed bridge foundations and retaining walls;
- Evaluate bridge foundation design alternatives and provide design recommendations for the selected foundation type;
- Provide lateral earth pressures (both dynamic and static), bearing resistance, and retaining wall geotechnical design parameters for bridge abutment design use;
- Evaluate settlement due to consolidation of the foundation soils for the fill retaining walls;
- Evaluate global stability of retaining walls and bridge abutments;
- Provide geotechnical construction considerations for earthwork, including site preparation, excavation, temporary shoring and retaining wall types, cut and fill slopes, structural fill material, fill placement, compaction, and wet weather construction; and
- Prepare this geotechnical report summarizing our explorations, lab testing, geotechnical design recommendations, and construction considerations.

3 GEOLOGIC AND SEISMIC SETTING

3.1 Regional Geology

The project site is located in the Willamette Lowland at the northern end of the Central Willamette Valley (Gannett and Caldwell, 1998). Regional and local geology of the Wilsonville area has been mapped by Schlicker and Deacon (1967), Walker and MacLeod (1991), and by O'Connor and others (2001).

The Willamette Lowland is a structural depression created by tectonic forces acting on basalt flows of the middle Miocene age (approximately 17 to 6 million years old) Columbia River Basalt Group (CRBG) and older underlying basement rock. The once relatively uniform lava surface is now extensively folded and faulted such that it lies both above and below the general elevation of the Central Willamette Valley floor. CRBG forms Parrett Mountain and Petes Mountain northwest and northeast of the site, respectively.

In the Wilsonville and Portland area, the CRBG is overlain by Upper Miocene age (approximately 10.8 to 5.3 million years old) deposits of fine grained micaceous fluvial sediments derived from the Columbia and Willamette Rivers collectively termed Sandy River Mudstone which have an approximate thickness of up to 1000 feet (Orr and Orr, 2000). The Sandy River Mudstone is described by Gannett and Caldwell as a micaceous arkosic siltstone, mudstone, and claystone. Overlying the Sandy River Mudstone is the

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Pliocene age (approximately 5.3 to 1.8 million years old) Troutdale Formation which is described as a quartzite bearing basaltic conglomerate, vitric sandstone, and micaceous sandstone (Gannett and Caldwell, 1998). The total thickness of the Troutdale Formation is approximately 700 feet (Orr and Orr, 2000). Mapping in the Wilsonville area by Schlicker and others (1967) collectively includes the Sandy River Mudstone with the Troutdale Formation and describes the overall unit as poorly indurated silt, clay, and silty sand with occasional pebble conglomerate beds. More recent studies of the Portland and Tualatin Basins northwest and northeast of Wilsonville (Wilson, 1998 and Peterson and others, 2011) discuss Pliocene and Pleistocene age sediments which overlie the CRBG in the Tualatin Basin and the Troutdale Formation in the Portland Basin. The authors term the sediments Hillsboro Formation in the Tualatin Basin and Pleistocene alluvial sand and gravel in the Portland Basin. In the context of this report, we collectively term the sediments "Pliocene / Pleistocene Sediments".

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). These repeated floods are collectively referred to as the Missoula Floods. During each short-lived Missoula Flood episode, floodwaters washed across the Idaho panhandle, through eastern Washington's 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 pooled to elevations of about 400 feet, depositing a tremendous load of sediment. Boulders, cobbles, and gravel were deposited nearest the mouth of the gorge and along the main channel of the Columbia River. Cobble-gravel bars reached westward across the basin, grading to thick blankets of micaceous sand and silt (Allen and others, 2009). Ma and others (2012) divided the Missoula Flood Deposits into two groups; Fine-Grained Deposits consisting of sand and silt and Coarse-Grained Deposits consisting mostly of gravel with cobbles and boulders. In the context of this report we term the coarse-grained deposits "Missoula Flood Deposits - Coarse".

The Tonquin Scablands Channels and Rock Creek Gap, north of the Wilsonville area, constricted flows from the Missoula Floods, creating a high-energy water surge from the Tualatin Basin in the north emptying into the Central Willamette Valley to the south. The high-velocity water flowing through the gap entrained coarse gravels, cobbles, and boulders that were dropped out of suspension when the surge lost energy (Thompson, 2012). As a result, much of the Wilsonville area is underlain by Missoula Flood Deposits - Coarse. In more recent times, the Willamette River and its tributaries have deposited alluvial sediments in and along their channels and floodplains (Ma and others, 2012; Smith and Roe, 2015).

It is also important to note that within the Willamette Valley and Portland area, artificial fill has become an important soil unit at some locations. Fill is generally placed to provide smoothed or raised ground surfaces for urban or industrial/commercial development. The fills are composed of various earth materials, compacted to varying degrees of density, and make up the upper geotechnical soil unit in areas of the project site.

3.2 Local Geology

The project site is located approximately 3 miles southeast of the Tonquin Scablands and Rock Creek Gap on outwash from the Missoula Floods. Geologic mapping by O'Connor and others (2001) and Ma and others (2012) indicate the area of the I-5 Wilsonville Pedestrian Bridge is underlain by Missoula Flood Deposits - Coarse. They describe the material as boulder, cobble, sandy gravel fans deposited by the Missoula Floods as they spilled into the northern Willamette Valley through the Rock Creek Gap. The gravel is described as poorly sorted and ranges from open-matrix gravel to gravel with considerable fine-grained matrix. The clasts are generally basalt, but other compositions may dominate downstream from bedrock exposures. Boulders or cobbles encased in the breached glacial ice during the Missoula Floods were rafted or carried in the massive floods and dropped along the way as the ice melted. These glacial erratic boulders and cobbles are found throughout the Portland Basin and the Tualatin and Willamette Valleys. Therefore, it is possible for boulders and cobbles to be found in the Missoula Flood Deposits - Coarse. Based on mapping by Schlicker and Deacon (1967), the Missoula Flood Deposits are underlain at depth by Troutdale Formation consisting of silt and clay with occasional pebble conglomerate beds. The Troutdale Formation of Schlicker and Deacon (1967) was later designated the Hillsboro Formation by Wilson (1998) which in this report we refer to as Pliocene / Pleistocene Sediments.

3.3 Seismic Setting

3.3.1 Earthquake Sources

The contemporary tectonics and seismicity of the region are the result of oblique, northeastward subduction at a rate of about 40 millimeters per year (mm/yr) (Personius and Nelson, 2006) of the Juan de Fuca oceanic plate beneath the North American continental plate (e.g., Wells and others, 1998; Wells and Simpson, 2001). This complex tectonic setting produces east-west compressive strain along the Cascadia Subduction Zone (CSZ), as well as northward translation and rotation of the mobile, crustal, Cascadia fore-arc blocks that

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span the leading edge of the North America plate (Wells and others, 1998; McCaffrey and others, 2007, 2013). Rotation of the Sierra-Nevada block and expansion of the Basin and Range drive the northward migration and clockwise rotation of the Cascadia fore-arc blocks (e.g., Pezzopane and Weldon, 1993; Wells and others, 1998; Wells and Simpson, 2001). As a result, the southern portion of the fore-arc, the Oregon Coast block, is impinging on western Washington at a rate of about 8 to 12 mm/yr causing crustal shortening in northwest Oregon and western Washington (Wells and others, 1998; Wells and Simpson, 2001; Mazzotti and others, 2002).

The combined effect of margin-normal subduction and margin-parallel shortening produces complex and diverse deformation within the northern edge of the Cascadia fore-arc and triggers large (greater than magnitude [Mw] 6.0), damaging earthquakes from three seismogenic source zones:

- The locked zone of the CSZ fault interface, which produces great mega-thrust earthquakes;
- The deep intraslab portion of the CSZ (i.e., the subducted portion of the Juan de Fuca Plate), the source off Wadati-Benioff zone earthquakes; and
- The overriding North American Plate, where shallow crustal faults rupture.

All three sources potentially produce earthquakes that impact the ground motion hazards at the project site. Offshore, elastic release of strain accumulated in the locked plate interface of the CSZ produces great megathrust earthquakes (greater than Mw 8.0) occurring at irregular intervals that span from about 100 to more than 1,200 years, with an average recurrence interval of about 300 to 500 years (Atwater and Hemphill-Haley, 1997; Clague, 1997; Goldfinger and others, 2003 and 2012); and the most recent rupture occurred in A.D. 1700 (Satake and others, 1996; Atwater and Hemphill-Haley, 1997; Clague, 1997; Yamaguchi and others, 1997; Goldfinger and others, 2003 and 2012). Onshore, migration and rotation of tectonic blocks produce deformation along shallow faults within the upper part of the crust. At depth, rupture within the subducting slab, referred to as the intraslab, has produced some of the largest recorded earthquakes (Mw 6.5 to 7.0) to strike the Pacific Northwest, the northern California Coast, and Western Washington. However, over the past century, intraslab earthquakes have been markedly infrequent in Oregon. The following sections briefly describe the location, characteristics, and seismicity of each of the sources.

3.3.1.1 Cascadia Subduction Zone: Mega-Thrust Source

CSZ mega-thrust earthquakes originate along the interface between the subducting oceanic plates and the North American plate. Because of the significant uncertainty of the landward extent of a potential rupture surface, estimates of the closest distance between the project and potential rupture surface range from about 65 to 140 horizontal miles. Focal depths for

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mega-thrust earthquakes are commonly on the order of about 15 to 25 miles. Rupture of the interface could result in earthquakes with Mw on the order of 8.5 to over 9.0, with strong shaking that lasts for several minutes. No large earthquakes have occurred in this zone during historic times (in the last 170 years). However, geologic evidence suggests that coastal estuaries have experienced rapid subsidence at various times within the last 2,000 years (e.g., Atwater, 1987; Atwater and Hemphill-Haley, 1997) as a result of tectonic movement associated with mega-thrust earthquakes on the CSZ. It appears that ruptures of this zone have occurred at irregular intervals that span from about 100 to more than 1,200 years, with an average recurrence interval of about 300 to 500 years (Atwater and Hemphill-Haley, 1997). Based on historical tsunami records in Japan (Satake and others, 1996) the most recent interplate event on the CSZ was a Mw 9.0 event on January 26, 1700.

3.3.1.2 Cascadia Subduction Zone: Intraslab Source

CSZ intraslab earthquakes originate from within the subducting oceanic plates as a result of down-dip tensional forces and bending caused by mineralogical and density changes in the plates at depth. These earthquakes typically occur 28 to 37 miles beneath the surface. The nearest seismogenic intraslab portion of the Juan de Fuca plate is approximately 30 to 60 miles below the Portland area. Ludwin and others (1991) estimate that the maximum Mw from this source zone would be about 7.5. Ground shaking produced by intraplate earthquakes would be less intense and less prolonged in the Portland area than ground motions generated by large subduction zone interface earthquake events. Historic seismicity from this source zone includes the 1949 Mw 6.7 Olympia earthquake, the 1965 Mw 6.7 earthquake between Tacoma and Seattle, and the 2001 M 6.8 Nisqually earthquake. While intraslab events have occurred frequently in the Puget Sound area, they are historically rare in Oregon.

3.3.1.3 Shallow Crustal Source

Shallow crustal earthquakes within the North American Plate have historically occurred in a diffuse pattern within Pacific Northwest, typically within the upper 4 to 19 miles of the continental crust. Mabey and others (1993) concluded from their analysis of local geologic features that a crustal earthquake of up to Mw 6.5 could occur virtually anywhere in the Portland area. Based on their fault model, Wong and others (2000) determined that an earthquake of up to Mw 6.8 is possible on the Portland Hills Fault, which is mapped within about one half-mile of the project site. The largest known crustal earthquake in the Pacific Northwest is the 1872 North Cascades earthquake at approximate Mw 6.5 to 7.0. Other examples include the 1993 Mw 5.6 Scotts Mill earthquake and the 1993 Mw 6.0 Klamath Falls earthquake.

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3.3.2 Local Faults and Folds

Shallow crustal faults and folds throughout Oregon have been located and characterized by the United States Geological Survey (USGS). The USGS provides approximate fault locations and a detailed summary of available fault information in the USGS Quaternary Fault and Fold Database. The database defines four categories of faults, Class A through D, based on evidence of tectonic movement known or presumed to be associated with large earthquakes during Quaternary time (within the last 2.6 million years). For Class A faults, geologic evidence demonstrates that a tectonic fault exists and that it has likely been active within the Quaternary period. For Class B faults, there is equivocal geologic evidence of Quaternary tectonic deformation, or the fault may not extend deep enough to be considered a source of significant earthquakes. Class C and D faults lack convincing geologic evidence of Quaternary tectonic deformation or have been studied carefully enough to determine that they are not likely to generate significant earthquakes.

According to the USGS Quaternary Fault and Fold database (USGS, 2020), there are 12 Class A features within approximately 30 miles of the project site. Their names, general locations relative to the site, and the time since their most recent deformation are summarized in Exhibit 3-1. The CSZ itself is approximately 135 miles west of the project site, with an average slip rate of approximately 40 millimeters (1.5 inches) per year and the most recent deformation occurring about 300 years ago (Personius and Nelson, 2006).

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Exhibit 3-1: USGS Class A Faults Within an Approximate 30-mile Radius of the Project Site

Fault Name	USGS Fault Number	Approximate Length	Approximate Distance and Direction from Project Site[1]	Slip Rate Category[2]	Time Since Last Deformation[3]
Portland Hills Fault	877	30.4 miles	11.0 miles NE	< 0.2 mm/yr	<1.6 Ma
East Bank Fault	876	18.0 miles	15.8 miles NNE	< 0.2 mm/yr	< 750 ka
Oatfield Fault	875	18.0 miles	10.5 miles NE	< 0.2 mm/yr	< 1.6 Ma
Grant Butte Fault	878	6.2 miles	15.8 miles NE	< 0.2 mm/yr	< 750 ka
Damascus-Tickle Creek Fault	879	9.9 miles	12.8 miles NE	< 0.2 mm/yr	< 750 ka
Beaverton Fault Zone	715	9.3 miles	12.5 miles NNW	< 0.2 mm/yr	< 750 ka
Canby-Molalla Fault	716	31.1 miles	4.0 miles E	< 0.2 mm/yr	< 15 ka
Helvetia Fault	714	4.3 miles	17.6 miles NW	< 0.2 mm/yr	< 1.6 Ma
Lacamas Lake Fault	880	14.9 miles	27.0 miles NW	< 0.2 mm/yr	< 750 ka
Newberg Fault	717	3.1 miles	9.1 miles W	< 0.2 mm/yr	< 1.6 Ma
Gales Creek Fault Zone	718	45.4 miles	15.9 miles WNW	< 0.2 mm/yr	< 1.6 Ma
Mount Angel Fault	873	18.6 miles	13.3 miles S	< 0.2 mm/yr	< 15 ka

NOTES:

1 Approximate distance between project site and nearest extent of fault mapped at the ground surface.

2 mm = millimeters; yr = year.

3 Ma = "Mega-annum" or million years ago; ka = "Kilo-annum" or one thousand years ago.

4 FIELD EXPLORATIONS AND LABORATORY TESTING

4.1 Subsurface Explorations

Subsurface conditions at the site were explored with five geotechnical borings, designated B-1 through B-5. The locations and elevations of the borings have not been surveyed at the time of this report. Completed locations of borings were measured in the field with a geographic positioning system (GPS) and approximate boring locations are shown on Figure 2, Site and Exploration Plan. The geotechnical borings were drilled between April 6, 2020 and August 31, 2020 using three different truck-mounted CME-75 rotary drill rigs provided and operated by Western States Soil Conservation, Inc., out of Hubbard, Oregon. The borings were advanced to depths ranging from 61.5 to 76.5 feet below ground surface (bgs) using open-hole mud rotary drilling techniques. A Shannon & Wilson geology staff member was present throughout the exploration program to locate the borings, observe the drilling, collect soil samples, and log the materials encountered.

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Details of the subsurface explorations, including descriptions of the techniques used to advance and sample the borings, logs of the materials encountered, and borehole installation and abandonment procedures, are presented in Appendix A, Field Explorations.

Four additional geotechnical borings are proposed to be performed during final design, two for the MSE approach retaining walls and two for the bridge foundations as shown on Figure 2.

4.2 In-Situ Infiltration Testing

Two encased falling head infiltration tests, designated I-1 and I-2, are proposed within 10 feet of boring B-4 and proposed boring B-8 near the I-5 Pedestrian Bridge east approach, as shown on Figure 2, and will be performed during final design. The tests will be performed to support design of stormwater infiltration facilities within the project area and will be conducted in accordance with the 2015 City of Wilsonville Stormwater & Surface Water Design & Construction Standards.

4.3 Laboratory Testing

The samples we obtained during our subsurface explorations were transported to our laboratory for additional observations. We then selected some samples for laboratory testing. The laboratory testing program included moisture content tests, Atterberg limits tests, particle-size analyses, specific gravity, and corrosivity testing. Testing was performed by, GeoTesing Express of Acton, Massachusetts, and Shannon & Wilson. All tests were performed in accordance with applicable ASTM International (ASTM) standards. The results of the laboratory tests and brief descriptions of the test procedures are presented in Appendix B, Laboratory Test Results.

5 SUMMARY OF SUBSURFACE CONDITIONS

5.1 Geotechnical Soil Units

We grouped the materials encountered in our field explorations into three geotechnical units, as described below. Our interpretation of the subsurface conditions is based on our explorations and regional geologic information from published sources. Typical descriptions of the geotechnical units identified in the borings are as follows:

- **Fill**: includes pavement sections, surficial topsoil, and dense/hard, Silt with trace to some sand (ML); stiff to very stiff, Silty Clay with trace sand (CL); stiff to very stiff, Clay (CH); and very dense, Silty Gravel with some sand (GM);

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- **Missoula Flood Deposits - Coarse**: medium dense, Gravelly silty Sand (SM); medium dense to very dense, Silty Gravel with some sand (GM), Sandy silty Gravel (GM), Gravel with some sand and silt (GP-GM), Sandy clayey Gravel (GC), and Clayey Gravel with some sand (GC), with cobbles and possible boulders;
- **Pliocene / Pleistocene Sediments**: stiff to very stiff, Silt to Silt with some sand (ML); stiff, Clayey Silt (MH); stiff to hard, Silty Clay to Sandy silty Clay with trace gravel (CL); very stiff Clay with trace to some sand (CH); medium dense to dense, Silty Sand to Silty Sand with trace gravel (SM); medium dense, Clayey Sand to Clayey Sand with some gravel (SC), and very dense, Clayey Gravel with some sand (GC).

These geotechnical units were grouped based on their engineering properties, geologic origins, and their distribution in the subsurface. Our interpretation of their distribution in the subsurface is shown on the Interpretive Subsurface Profile A-A', Figure 3. The profile is interpretive, and variations in subsurface conditions may exist between the borings. Contacts between units may be more gradational than shown in the profiles and in the Drill Logs in Appendix A. The Standard Penetration Test (SPT) blow counts shown on the Drill Logs, profile, and discussed below, are in blows per foot (bpf) as counted in the field (uncorrected). The following sections describe the geotechnical unit characteristics in greater detail.

5.1.1 Fill

Fill was encountered in all borings from the ground surface to depths ranging from 1 to 9.5 feet bgs. This material was present at the surface in all borings and includes pavement sections consisting of approximately 6-inches of asphalt concrete underlain by approximately 6-inches of base aggregate, and where pavement sections were not encountered includes approximately 6-inches of surficial topsoil. The Fill material underlying the pavement sections and topsoil typically consisted of dense/hard, brown Silt with trace to some sand (ML); very stiff, red-brown to brown, Silty Clay with trace sand (CL); stiff to very stiff, brown Clay (CH); and very dense, brown to gray Silty Gravel with some sand (GM). The material is highly variable and ranges from clay- to gravel-size particles. Trace pockets of Silty Clay (CL), trace organics, and slight iron oxidation and staining were observed in some samples. Given the highly variable nature of the Fill material, cobbles and boulders may be possible within the unit. Although not encountered in the borings, based on other local explorations, roadway and construction debris may also be encountered within the Fill such as asphalt and concrete fragments, metal, glass and plastic debris, and wood and organics. Four out of eleven SPTs attempted in the unit met refusal with greater than 50 blows required to drive the sampler over a 6-inch interval. Non-refusal SPT N-values within the Fill ranged from 9 to 32 bpf and averaged 17 bpf. One

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Atterberg Limits test on a sample of Fill indicated a Plasticity Index of 25 and a USCS designation of CH.

5.1.2 Missoula Flood Deposits - Coarse

Missoula Flood Deposits - Coarse was encountered underlying the Fill in all borings and extended to depths ranging from 42.5 to 57.5 feet, with thicknesses of the unit ranging from 35.5-to 56-feet thick. The unit typically consisted of medium dense to very dense, gray to brown, Gravel with varying amounts of sand, silt and clay (GM, GP-GM, and GC) with minor interbeds of Gravelly silty Sand (SM). The gravel constituent was typically fine to coarse, and subangular to subrounded. Cobbles inferred from drill action were encountered in boring B-2 and based on other local explorations, boulders are often encountered within the Missoula Flood Deposits. There was considerable difficulty during drilling, and drilling mud circulation loss and drilling mud loss was observed within the Missoula Flood Deposits – Coarse material, often indicative of open-matrix gravels. Borehole instability and sloughing of the boreholes was also observed during drilling and in borings B-3 and B-4, casing was advanced to prevent sloughing and drilling mud loss. Six out of fifty-one SPTs attempted in the unit met refusal with greater than 50 blows over a 6-inch interval. Non-refusal SPT N-values ranged from 14 to 77 bpf and averaged 36 bpf. Grain size analysis on two composite samples of Missoula Flood Deposits – Coarse material indicated the percent passing the #200 sieve was 16 and 18 percent by dry weight.

5.1.3 Pliocene / Pleistocene Sediments

Pliocene / Pleistocene Sediments were encountered underlying the Missoula Flood Deposits in all borings at depths ranging from 42.5 feet in boring B-4 and B-5 to 57.5 feet in boring B-2. All five borings were terminated within this unit. Directly underlying the Missoula Flood Deposits in all borings, the upper section of the unit typically consisted of stiff to very stiff, gray and blue-gray, Silt with varying amounts of sand (ML). The Silt was typically low plasticity and micaceous, and occasional pockets of Silty Clay (CL) were observed within the material. Below the upper Silt section, the Pliocene / Pleistocene Sediments consisted of stiff to very stiff, gray, blue-gray and gray to brown, medium to high plasticity, Silty Clay, Clay and Clayey Silt with varying amounts of sand and gravel (CL, CH, MH), medium dense to very dense, brown, gray, dark gray, and blue-gray, Sand with varying amounts of gravel, silt and clay (SM, SC), and medium dense to very dense, blue-gray and gray to brown, Gravel with varying amounts of sand and clay (GC). The Pliocene / Pleistocene Sediments typically range from clay- to gravel-sized particles and occasional cobbles were inferred in boring B-2 within the unit. The material was described as micaceous, and slight to moderate iron oxidation and staining of samples was also often observed. Weak

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cementation was observed in boring B-1 and trace organics were observed in one sample in boring B-5.

Three out of twenty-six SPTs attempted in the unit met refusal with greater than 50 blows over a 6-inch interval. Non-refusal SPT N-values ranged from 9 to 75 bpf and averaged 26 bpf. Moisture content tests performed on samples of Pliocene / Pleistocene Sediments indicated moisture contents ranging from 22 to 43 percent and averaged 33 percent. Grain size analysis samples indicated the material passing the #200 sieve was 78 and 99 percent by dry weight. Atterberg Limits tests on three samples from 45, 50 and 60 feet (all predominantly fine-grained) indicated Plasticity Indexes of 10, 10 and 11, and a USCS designation of ML.

5.2 Groundwater

The borings were drilled using mud rotary techniques which make it difficult to discern the depth to groundwater if it is encountered. According to Well Logs retrieved from the Oregon Water Resources Department Well Report Mapping Tool (OWRD, 2020), monitoring wells installed in an approximate1,000-foot radius of the site indicate groundwater levels varying between 18 and 43 feet bgs. Groundwater levels should be expected to vary with changes in precipitation, time of year, topography, or other factors not observed during our subsurface explorations. Locally, groundwater highs typically occur in the late fall to spring and groundwater lows typically occur in the late summer and early fall. Based off the existing data, we used a groundwater elevation of 140 feet for design, including seismic hazard evaluation. One vibrating wire piezometer is currently proposed to be installed for final design in the proposed boring B-7, as shown on Figure 2.

5.3 Soil Corrosivity

Soil corrosivity potential at the I-5 Pedestrian Bridge was evaluated based on the soil pH, electrical resistivity, and chloride and sulfate concentrations, and guidelines in Section 10.7.5 of the AASHTO LRFD Bridge Design Specifications (AASHTO, 2020). Lab testing was completed by GeoTesting Express, Inc. of Acton, Massachusetts. The results of the corrosivity testing suite indicate that the risk for corrosion is low. Detailed analytical results are presented in Appendix B.

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6 SEISMIC GROUND MOTIONS AND HAZARD EVALUATIONS

6.1 Seismic Design Ground Motions

The ODOT Geotechnical Design Manual (GDM) (ODOT, 2018) requires that all bridges and highway retaining walls be designed for 1,000-year return period ground motions under "Life-Safety" criteria. Under this level of shaking, the bridge and approach structures, bridge foundation, approach slopes, and highway retaining walls must be able to withstand the forces and displacements without collapse of any portion of the structure.

ODOT also requires that all bridges and bridge retaining walls (i.e. retaining walls located within 100 feet of a bridge abutment) be designed to remain "Operational" after a full rupture Cascadia Subduction Zone Earthquake (CSZE). Under this level of shaking, the bridge, approach slopes, and bridge retaining walls are designed to remain in service shortly after the event to provide access for emergency vehicles. Guidance provided by the ODOT GDM (ODOT, 2018) states up to 1 foot of lateral displacement and 6 to 12 inches of vertical settlement is generally considered acceptable under the "Operational" design criteria.

The Seismic Site Class for the "Life-Safety" seismic design criteria was developed based on the recommended procedure, using SPT N-values from the explorations, in the AASHTO Load and Resistance Factor Design (LRFD) Bridge Design Specifications (AASHTO, 2020). Based on our interpretation of the borings performed in the exploration program for this project, the subsurface conditions at the proposed I-5 Pedestrian Bridge site are best characterized as Site Class D. Site Class D corresponds to soils with an average-weighted shear wave velocity between 600 and 1,200 feet per second (fps) or an average-weighted SPT N-value between 15 and 50 bpf in the upper 100 feet of soil.

While the Site Class is used in deriving the "Life-Safety" ground motion seismic parameters, the average-weighted shear wave velocity in the upper 30 meters of the soil profile (V_{s30}) is required to derive the "Operational" criteria response spectra. A V_{s30} of 250 meters per second (m/sec) (830 fps) was estimated based on the subsurface data available.

The ground motion seismic parameters for the "Life-Safety" criteria were derived using the ODOT Bridge Section's Excel application, ODOT_ARS.v.2014.16, which uses the three-point curve method with data from the 2014 USGS probabilistic seismic hazard maps for the 1,000-year return period. This Excel application is available through ODOT's web portal (ODOT, 2017).

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The deterministic response spectrum for the CSZE considered in the "Operational" seismic design criteria was generated by using the web-based application developed by Portland State University and available on the ODOT Bridge Section website (ODOT, 2017). Using the ODOT web-based application, a V_{s30} of 250 m/sec was input to generate the design response spectrum.

The recommended ground motion parameters are given in Exhibit 6-1, and the corresponding recommended design spectra are given on Figure 4.

Exhibit 6-1: Recommended Site Class D Acceleration Response Spectra for I-5 Wilsonville Pedestrian Bridge Project Site

Operational CSZE Full-Rupture Deterministic		Life-Safety 1,000-Year Return Period	
Period (s)	Sa (g)	Period (s)	Sa (g)
0	0.180	0.00	0.346
0.05	0.178	0.12	0.759
0.1	0.251	0.20	0.759
0.15	0.314	0.60	0.759
0.2	0.353	0.80	0.573
0.25	0.386	1.00	0.458
0.3	0.421	1.20	0.382
0.4	0.456	1.40	0.327
0.5	0.437	1.60	0.287
0.6	0.395	1.80	0.255
0.7	0.369	2.00	0.229
0.8	0.349	2.20	0.208
1	0.299	2.40	0.191
1.5	0.219	2.60	0.176
2	0.170	2.80	0.164
2.5	0.140	3.00	0.153
3	0.114	3.20	0.143
--	--	3.40	0.135
--	--	3.60	0.127
--	--	3.80	0.121
--	--	4.00	0.115

6.2 Seismic Hazard Evaluation

Seismic hazards generally include ground shaking, liquefaction and associated effects (e.g., flow failure, lateral spreading, and settlement), soil compaction, slope instability, ground

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surface fault rupture, and earthquake-induced flooding (i.e., tsunami and seiche). The primary seismic hazard at the project site is strong ground shaking. In our opinion, the potential for fault rupture is low given the distance between the project site and the nearest potentially active fault. The risk of seismically induced tsunami and seiche is also very low at the site. Based on our subsurface explorations, the relative density of the subsurface soils, and anticipated groundwater level, the on-site materials do not appear to be susceptible to liquefaction or related effects.

7 BRIDGE FOUNDATION DESIGN RECOMMENDATIONS

7.1 General

The proposed I-5 Pedestrian Bridge will connect Barber Street to the Wilsonville Town Center, crossing over Boones Ferry Road, I-5, and Town Center Loop West. As described in Section 2.2, we understand the current design consists of an eight-span structure with each interior bent supported on two drilled shafts and MSE abutments (spread footings constructed on back-to-back MSE walls for the bridge approaches).

Our design recommendations for the proposed bridge are based on the design information provided by DOWL. Geotechnical design recommendations are provided for the proposed bridge foundations, bridge abutments, and wing walls. Also, key construction considerations were developed associated with the geotechnical design recommendations for each project element. If project information changes, especially with regards to foundation types or design configurations after this report, Shannon & Wilson should be contacted so that we may reevaluate our recommendations and provide updates if necessary.

7.2 Bridge Foundation Alternatives

The selection of an appropriate foundation system for the proposed I-5 Pedestrian Bridge is dependent upon several factors, including foundation capacities, subsurface conditions, tolerance to total and differential settlement resulting from static loads, and construction considerations. Risk is involved with constructing spread footings directly upon the native gravels (Missoula Flood Deposits – Coarse) due to the variability in blow counts observed in our subsurface explorations and variable depth of overlying fill material. In addition, spread footings are also not feasible at Bents 4, 5, 6, and 7, due to limited available foundation footprint area. However, spread footings that are founded on the back-to-back MSE walls at the abutments are feasible due to the relatively modest design loads and assuming the MSE walls are founded on the native gravels. Driven pipe piles through the MSE wall were initially considered to support the abutments, however pile driving at the

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east end of the bridge may result in unacceptable vibrations at the nearby EyeHealth Northwest clinic. A discussion of potential construction vibration impacts is provided in Section 9.4. In our opinion, drilled shafts are the most economical and feasible foundation alternative at the proposed interior bent locations due to limited foundation footprint area for spread footings and pile driving vibration concerns at the east end of the bridge. A comparison of the foundation alternatives considered is presented in Table 1.

Based upon the comparisons summarized in Table 1, and through discussion with DOWL, we understand spread footings founded on the back-to-back MSE walls at the bridge approaches (MSE abutments) are the preferred foundation alternative at the abutments and dual drilled shafts are the preferred foundation alternative at the interior bents. Single drilled shaft support was considered at the interior bent locations, however some of the resulting shaft lengths based on axial capacity demands exceeded 100 feet.

The following sections present our geotechnical design recommendations for MSE abutments at Bents 1 and 9 and drilled shafts at Bents 2 through 8.

7.3 Bridge Abutment and Wingwall Design Recommendations

7.3.1 General

Based on conversation with DOWL, we understand the proposed bridge structure abutments will be founded on spread footings constructed on top of back-on-back MSE walls that retain up to 11 feet of fill (from top of wall to finished grade in front of wall).

Short abutment walls and wing walls will be constructed on top of the spread footings. This will impose additional loads on the spread footings supporting the bridge, although lateral loads due to earth pressures will be partially offset due to the wing wall on the opposite end of the footing. The following sections provide our recommendations for the bridge abutments and wing walls.

For design purposes, we have assumed that subdrainage systems will be installed to prevent hydrostatic pressure from developing behind all retaining walls. Also, we have assumed that the backfill behind the walls is flat.

7.3.2 Global Stability

Global stability was evaluated at the proposed bridge abutment locations considering the generalized subsurface conditions along the bridge centerline. The generalized subsurface conditions along the proposed bridge alignment are presented in Figure 3. Soil parameters for the analyses were determined from the results of field explorations, laboratory testing,

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standard ODOT recommended values for specified backfill materials, and engineering judgement.

We conducted global stability analyses for the proposed bridge abutments using the computer program SLOPE/W, Version 11 (Geo-Slope International, 2021). This program employs limit-equilibrium methods in accordance with the ODOT GDM (ODOT, 2018). The Morgenstern-Price slope stability analysis method was used for rotational and irregular surface failure mechanisms. The analyses were performed at the proposed bridge abutment locations, longitudinal to the bridge centerline, for static and seismic conditions.

An abutment footing bearing pressure of 4 ksf (recommended service limit state bearing resistance, see Section 7.5), applied over a 5-foot width, was assumed at the bridge abutments to model the proposed spread footing loading on the MSE wall. For the seismic condition, pseudo-static procedures described in the ODOT GDM (ODOT, 2018), Chapter 6 were followed. Horizontal acceleration coefficients equal to one-half of the site peak ground accelerations (0.5 x F_{pga} x PGA) were used. For our seismic slope stability analyses we used horizontal seismic coefficients, k_h, equal to 0.09 and 0.173 for the "Operational" and "Life Safety" criteria, respectively. Only seismic global stability analyses considering "Life Safety" criteria are shown in our results, which we determined were the controlling ground motions in our evaluation.

The ODOT GDM (ODOT, 2018) requires that slopes supporting bridge foundations be designed with a maximum resistance factor for global stability of 0.65, equivalent to a Factor of Safety (FS) of 1.5, for static conditions. For seismic analyses, a maximum resistance factor of 0.9, or an FS of 1.1, is required.

We modeled the approximate geometry of the abutments and grading displayed on the preliminary plans provided by DOWL. In accordance with the ODOT GDM (ODOT, 2018), the embedment for the MSE wall was assumed to be 2 feet at the face of the wall. In addition, we assumed the foundation for the MSE wall will be ODOT Stone Embankment Material which will be placed from the top of native Missoula Flood Deposits – Coarse to the bottom of the wall, in accordance with our recommendations in Section 8.2. MSE wall reinforcement length was assumed to be 70 percent of the total wall height (0.7H) as measured from the top of the leveling pad to roadway grade, or 8 feet, whichever was greater.

Based on our analyses, the proposed bridge abutments designed following the recommendations in this report will satisfy the minimum global stability FS requirements for all conditions assuming the minimum geometric requirements detailed above are met. A minimum 4-foot wide bench should be provided in front of the walls in accordance with

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ODOT GDM (ODOT, 2018) Section 15.3.7. The results of our global stability analyses for the bridge abutments are presented in Figures C1 to C4 in Appendix C, Global Stability Analysis Results, and summarized in Exhibit 7-1.

Exhibit 7-1: Global Stability Analysis Results for Proposed I-5 Wilsonville Pedestrian Bridge MSE Abutments with 0.7H Reinforcement Length

Location	Analysis Case	Factor of Safety	Minimum Factor of Safety Required by ODOT GDM
Bent 1 (West Abutment)	Static	1.5	1.5
	Seismic	1.2	1.1
Bent 9 (East Abutment)	Static	1.5	1.5
	Seismic	1.3	1.1

If the abutment configurations or grading in front of the abutments change, Shannon & Wilson should be notified to review and revise our recommendations as necessary.

7.3.3 Lateral Earth Pressures

The lateral earth pressures on the abutments and wing walls depend on the type of wall (i.e., yielding or non-yielding), the type and method of placement of backfill against the wall, the magnitude of surcharge weight on the ground surface adjacent to the wall, the slope of the backfill, and the design criteria. Based on the structural design information and the above assumptions, the lateral earth pressures on the walls were developed according to the ODOT GDM (ODOT, 2018) and AASHTO LRFD (AASHTO, 2020). The static lateral earth pressure acting on walls consists of two components: static earth pressure and static surcharge pressure. The seismic lateral earth pressure on walls consists of three components: static earth pressure, static surcharge pressure, and seismic earth pressure. A k_h equal to the site peak ground acceleration (F_{pga} x PGA), A_s, was used to determine the seismic earth pressure for non-yielding walls. A k_h equal to 1/2 of A_s was used to determine the seismic earth pressure for yielding walls, where 1 to 2 inches of lateral deformation is acceptable. The distributions of these lateral pressures are shown on Figure 5, Recommended Lateral Pressures for Bridge Abutments and Wing Walls.

7.3.4 Subdrainage

Suitable drainage for walls can be provided by granular backfill material and a wall base subdrain system consisting of a 6-inch-diameter perforated or slotted drain pipe. The perforated or slotted drain pipe should be wrapped in an envelope of filter material at least 12 inches thick and confined by a separation geotextile. The filter material is specified in Section 00430.11 of the ODOT Oregon Standard Specifications for Construction (OSSC)

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(ODOT, 2021). The subdrain should convey any collected seepage to the end of the wall and daylight at low spots below the wall elevation.

7.3.5 Backfill Material and Compaction

The wall backfill material should be in accordance with standard ODOT Granular Wall Backfill (Section 00510.12 of the OSSC) (ODOT, 2021). Heavy compaction equipment should not be allowed closer than 3 feet to the retaining wall to prevent high lateral earth pressures and/or wall yielding and/or damage. Required compaction of wall backfill within 3 feet of the walls shall be obtained using hand-operated compaction equipment, such as a vibrating plate compactor.

7.3.6 Lateral Resistances

We assume the lateral resistance for the abutment walls can be provided by the bridge foundations and lateral resistance for the wing walls will be generated through the structural connection with the abutment. If it is determined that bridge foundations designed without specific consideration for retaining wall loading cannot adequately support the abutments, specific foundation design recommendations will be provided upon request.

7.4 Drilled Shaft Design Recommendations

7.4.1 General

The following sections provide our recommendations for axial and lateral resistance of 5- and 6.5-foot-diameter drilled shafts for the proposed I-5 Pedestrian Bridge interior bent foundations (Bents 2 through 8). We understand that two 5- or 6.5-foot-diameter drilled shafts, spaced 20-feet on-center, will be used to support Bents 2, 3, 7, and 8, and that two 6.5-foot-diameter drilled shafts, spaced 26-feet on-center, will be used to support Bents 4, 5, and 6 where the loads are greater.

7.4.2 Drilled Shaft Axial Resistance

We performed axial resistance evaluation for drilled shafts in general accordance with the AASHTO LRFD Section 10.8 (AASHTO, 2020). We evaluated axial resistance for service, strength, and extreme event limit states. The analyses were based on the subsurface conditions encountered in the project borings and our experience with similar soil and project conditions. We estimated unit side and tip resistance values based on the average SPT N-values within each unit, laboratory test results, and our experience.

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Our axial resistance analysis results are presented in Figures 6 through 10 for drilled shafts at the interior bents. These results are presented as plots of nominal and factored axial resistance versus depth for service, strength, and extreme event limit states. Recommended resistance factors for each limit state are provided in the notes section of each figure. Estimated foundation length and tip elevation, based on the factored design loads provided in Exhibit 2-4, are summarized in Exhibit 7-2. The estimated foundation length and tip elevation provided are based on axial capacity requirements only and do not consider lateral capacity requirements, such as the depth required to develop lateral shaft fixity. We also considered a minimum shaft penetration of two shaft diameters (2D) into the Missoula Flood Deposits – Coarse bearing layer.

Exhibit 7-2: Estimated Drilled Shaft Length and Compressive Resistance

Bent	[1]Estimated Top of Shaft Elevation (feet)	Foundation Type	[2]Estimated Shaft Length (feet)	Estimated Shaft Tip Elevation (feet)	Factored Axial Compressive Resistance (kips)		
					Strength Limit	Service Limit	Extreme Event Limit
2	165	Two 5-foot dia. Drilled Shafts	34	131	431	484	828
		Two 6.5-foot dia. Drilled Shafts	21	144	436	328	852
3	165	Two 5-foot dia. Drilled Shafts	34	131	431	484	828
		Two 6.5-foot dia. Drilled Shafts	21	144	436	328	852
4	165	Two 6.5-foot dia. Drilled Shafts	34	131	674	633	1301
5	175	Two 6.5-foot dia. Drilled Shafts	28	147	955	672	1875
6	171	Two 6.5-foot dia. Drilled Shafts	22	149	881	542	1741
7	171	Two 5-foot dia. Drilled Shafts	20	151	506	375	999
		Two 6.5-foot dia. Drilled Shafts	20	151	860	504	1702
8	171	Two 5-foot dia. Drilled Shafts	31	140	615	573	1197
		Two 6.5-foot dia. Drilled Shafts	20	151	860	504	1702

NOTE:
1 Estimated top of shaft elevation based on approximate grade at proposed bent location, obtained from survey file, "I-5 Pedestrian Bridge Topo Basemap" provided by DOWL on September 9, 2020.
2 Estimated shaft length, taken as the distance between estimated top of shaft and estimated shaft tip elevation, is assumed to be +/-5 feet of the table value.

The estimated nominal axial resistance assumes the shafts are oriented in a single row and spaced at least three shaft diameters apart (3D), measured center-to-center. Based on our understanding that the shafts are oriented in a single row and spaced greater than three shaft diameters apart (3D), axial group effects are not considered.

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7.4.3 Drilled Shaft Lateral Resistance

The drilled shaft foundations will be subjected to lateral loads resulting from live and seismic loading. We understand that the laterally loaded shaft analyses will be performed with the aid of the LPILE computer program. Our recommended geotechnical input parameters for LPILE are provided in Table 2 for the static/seismic conditions at the interior bents (Bents 2 through 8). Ground slope effects should also be considered where applicable in LPILE analyses.

The estimated lateral resistance parameters presented in Table 2 are recommended for shafts with center-to-center spacing greater than five shaft diameters (5D) and in a single row, and therefore do not consider group effects. However, we understand the drilled shafts at Bents 2, 3, 7, and 8 will be spaced 20-feet on-center (4.1D for 5-foot shafts and 3.0D for 6.5-foot shafts), and the drilled shafts at Bents 4, 5, and 6 will be spaced at 26-feet on-center (4.0D). Based on this understanding, we recommend P-Multipliers be applied, as recommended by Reese and Van Impe (Reese, 2001). P-Multipliers for the shaft sections under consideration loaded in the longitudinal and transverse direction to the bridge centerline are provided in Exhibit 7-3. If the drilled shaft layout changes during design, or a bridge skew is implemented, Shannon & Wilson should be contacted to revise our recommended P-Multipliers.

Exhibit 7-3: Recommended P-Multipliers for Drilled Shafts Under Lateral Loading for I-5 Wilsonville Pedestrian Bridge.

Shaft Diameter (foot)	Spacing (feet)	[1]Loading Direction	[2]Row 1	Row 2
5	20	Longitudinal	1.0	1.0
		Transverse	1.0	0.82
6.5	20	Longitudinal	0.94	0.94
		Transverse	0.94	0.74
6.5	26	Longitudinal	1.0	1.0
		Transverse	1.0	0.81

NOTES:

1 Loading direction is in reference to the centerline of the bridge.

2 Shaft row numbering begins farthest from load application, i.e. Row 1 is the row of shafts farthest from where load is being applied.

7.4.4 Drilled Shaft Foundation Construction Considerations

7.4.4.1 General

The drilled shaft installation procedures should follow the OSSC, Section 00512 (ODOT, 2021), and its project special provisions. The selection of equipment and procedures for constructing drilled shafts should consider shaft diameter and length and subsurface

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conditions. The design and performance of drilled shafts can be significantly influenced by the equipment and construction procedures used to install the shafts.

Generally, drilled shafts are constructed by excavating a cylindrical bore to the prescribed embedment with an auger or other drilling tools. Temporary or permanent casing is often used, depending on site conditions. If the shaft to column rebar splice is located beneath the ground surface, temporary or permanent casing will be required for construction of the rebar splice. Typically, the casing should extend a minimum of 2 feet below the construction joint for the shaft to column rebar splice (i.e. 2 feet below top of shaft concrete). Upon completion of drilling, cleaning, and inspection of the shaft, a steel rebar cage is placed, and concrete is pumped into the hole to complete the drilled shaft. In our opinion, due to the possibility for instability in the gravels of the Coarse grained Missoula Flood Deposits, and to protect the adjacent Boones Ferry Road, I-5, and Town Center Loop West roadways, we recommend that the drilled shafts at the interior bents be constructed using temporary fully-cased excavations. A pilot hole should not be allowed unless approved by the geotechnical engineer.

The drilled shafts should be constructed in the wet, and the casing should be advanced ahead of the auger. We do not recommend use of a vibratory hammer to install the temporary casing due to vibration concerns. Therefore, the temporary casing should be installed using a casing rotator or oscillator. Due to the potential hydrostatic imbalances, drilling slurry may be required to avoid soil loss around the casing. Equipment used to remove the temporary casing should be powerful enough (i.e., have enough torque) to account for the behavior of the subsurface materials.

Drilled shaft contractors who participate on this project should be required to demonstrate that they have suitable equipment for this project and adequate experience in the construction of shafts with similar subsurface conditions.

7.4.4.2 Potential Obstructions

Based on our explorations and knowledge of the Fill and Missoula Flood Deposits – Coarse units, occasional cobbles and boulders may be encountered in these units at the site. A statement should be included in the contract specifications alerting the contractor to potential difficulties with cobbles and boulders when installing the drilled shafts.

7.4.4.3 Potential Concrete Loss

Loss of concrete into open-matrix gravels within the Missoula Flood Deposits - Coarse unit may occur during temporary casing removal. If the concrete level in the shaft excavation drops below the temporary casing tip during casing removal, caving of the excavation

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sidewall may occur and result in anomalies within the drilled shaft concrete. OSSC Section 00512.47(e) (ODOT, 2021) requires a minimum 5-foot head of concrete be maintained above the tip of the temporary casing during casing removal. However, we recommend this requirement be increased to 10 feet.

7.4.4.4 Shaft Quality Control

We recommend full-time observation of the drilled shafts by a qualified representative from our firm to observe the contractor's means, methods, and equipment; and to assist the Agencies' drilled-shaft inspector with an understanding of the critical issues for drilled shaft construction. In addition, the design geotechnical engineer and structural engineer should make periodic site visits. We recommend that Crosshole Sonic Logging (CSL) tubes be installed in every shaft and that testing be performed on the shafts in accordance with the OSSC and its project special provisions.

7.5 MSE Abutment Spread Footing Design Recommendations

We understand that each abutment will be supported by a spread footing constructed on top of the back-to-back MSE walls at the bridge approaches, otherwise referred to as an MSE abutment. The dimensions of the spread footings have not been determined. According to the Federal Highway Administration (FHWA) Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Slopes, Publication No. FHWA-NHI-10-024 (FHWA, 2009), steel strip reinforcements have historically been used at MSE walls supporting bridge foundations, however we understand that geogrid can also be used. If geogrid is used for reinforcement, the ODOT GDM (ODOT, 2018) requires that the spread footing widths be greater than 2 feet but no wider than 15 feet.

The ODOT GDM (ODOT, 2018) requires a minimum clear distance of at least 18 inches between the back of the MSE wall facing to the front edge of the bridge abutment spread footing when bridge abutment spread footings are placed on MSE walls with steel reinforcements. If the MSE walls use geogrid reinforcement, the required minimum spacing between the back of the MSE wall facing to the front edge of the bridge abutment spread footing should be 3 feet.

The bearing resistance of the MSE reinforced backfill zone supporting these spread footings should be taken as the following values, which are directly from the ODOT GDM (ODOT, 2018):

- For Service Limit State, bearing resistance = 4,000 psf
- For Strength Limit State, factored bearing resistance = 7,000 psf
- For Extreme Event Limit State, factored bearing resistance = 8,000 psf

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Resistance to lateral movement for a spread footing consists of sliding friction, which can develop on the base of the footing. We recommend that the sliding resistance evaluation follow the general requirements for a spread footing in the 9th Edition AASHTO LRFD Bridge Design Specifications, Section 10 (AASHTO, 2020). The nominal friction resistance may be expressed as the vertical load (at the base of footing) multiplied by a coefficient of friction equal to 0.67 for spread footings constructed on the MSE reinforced backfill. For LRFD design, resistance factors of 0.8 and 1.0 should be used in calculation of friction sliding resistance for the strength and extreme event limit states, respectively.

Additionally, the bridge abutment design should follow the guidance in the ODOT GDM (ODOT, 2018) Section 15.6.15, which gives dimensional criteria. The MSE walls are discussed in Section 8 below, and the recommendations presented in that section takes the guidance in the ODOT GDM into account. The internal stability design of all MSE walls will be by others. The MSE wall designer should be given the loading requirements and geometries of these spread footings.

8 MSE RETAINING WALL DESIGN RECOMMENDATIONS

8.1 General

Based on the preliminary plans provided by DOWL, approach fill heights of approximately 11 feet are anticipated at both abutments. The approach fills will be retained with back-to-back MSE walls tapering to the existing ground surface away from the abutment. Spread footings will be constructed on the MSE walls at the abutments to support the bridge. Specific recommendations for the MSE abutments provided herein, including reinforcement length, wall embedment, and other dimensional criteria presented in the ODOT GDM (ODOT, 2018), are applicable to the MSE walls starting at the bridge abutment and extending along the wall alignment to the point where a 1H:1V line projected down from the back of the abutment spread footing intersects the bottom of wall. This abutment footing influence zone is shown on Figure 11, MSE Abutment Typical Details.

In accordance with standard design procedures outlined in the ODOT GDM (ODOT, 2018), we have provided design recommendations for the proposed MSE walls, including lateral earth pressures, bearing resistance, estimated settlements, evaluation of wall global stability, and foundation subgrade preparation. We understand that selection of wall types and specific wall design items, including internal wall stability, external sliding and overturning, and final wall configuration, will be performed by the wall designer. Final design plans and specifications should be provided for our review.

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For back-to-back MSE walls, the MSE wall designer should consider the design requirements in Federal Highway Administration (FHWA) Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Slopes, Publication No. FHWA-NHI-10-024 (FHWA, 2009).

For MSE abutments, the MSE wall designer should consider the design requirements in the ODOT GDM (ODOT, 2018), the AASHTO LRFD (AASHTO, 2020), and in the FHWA Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Slopes, Publication No. FHWA-NHI-10-024 (FHWA, 2009), ordered by precedence in case of design conflict.

Four additional borings are currently proposed to further investigate the subsurface conditions at the bridge bents and MSE walls. Based on the results of our findings, our design recommendations are subject to change. The following sections present our preliminary design evaluations and recommendations.

8.2 MSE Wall Design Parameters

As recommended by the ODOT GDM (ODOT, 2018), Section 15.6.4, the minimum soil reinforcement length at the MSE walls should be 70 percent of the wall height (0.7H) as measured from the leveling pad, or 8 feet, whichever is greater. The reinforced material should meet the requirements provided in OSSC Section 00596A.11(b) - MSE Granular Wall Backfill (ODOT, 2021). Backfill should be placed and compacted in accordance with OSSC Section 00596A.47. Retained fill (borrow material) placed behind the reinforced material should meet the specifications provided in OSSC Section 00330.12 (ODOT, 2021). The estimated soil parameters for MSE wall design are presented in Exhibit 8-1.

Exhibit 8-1: MSE Wall Geotechnical Design Parameters

Soil Parameter	Material Type				
	[1]Reinforced Material (MSE Granular Wall Backfill)	Retained Fill (Borrow Material)	Foundation Soil		
			West Approach Walls	East Approach Walls	MSE Abutments
Unit Weight (pcf)	130	125	120	110	125
Internal Friction Angle (degrees)	34	32	33	26	36
Cohesion (psf)	0	0	0	0	0

NOTES:
1 Use the reinforced material soil parameters for retained fill when designing for back-to-back MSE walls.

When designing back-to-back MSE walls, the MSE Granular Wall Backfill design parameters provided in Exhibit 8-1 should be used to calculate lateral earth pressures on the MSE wall.

The MSE abutments should be founded on native Missoula Flood Deposits - Coarse. This will require the over-excavation of the undocumented fill deposits at the west and east abutments to anticipated depths of 5 and 8 feet, respectively, however could require deeper or shallower excavation due to the uncertainty in undocumented fill thickness. The over-excavation should extend from the front face of the wall, along the wall alignment to the point where a 1H:1V line projected down from the back of the abutment spread footing intersects the bottom of wall. Figure 11 presents a schematic to assist in visualizing the above recommendations, however, does not present all dimensional criteria required for MSE abutment design. Refer to guidance in the ODOT GDM (ODOT, 2018) Section 15.6.15.

The bottom of the MSE abutment wall should be constructed on the native Missoula Flood Deposits – Coarse, as shown on Figure 11. Alternatively, as shown on Figure 12, the excavation may be backfilled with Stone Embankment Material meeting ODOT OSSC, Section 00330.16 (ODOT, 2018) up to 2-feet below finish grade and then the MSE abutment wall constructed on top of the Stone Embankment material. The Stone Embankment material should also extend a minimum of 1-foot outside the face of the MSE wall, then down at a maximum 1H:1V slope to the bottom of over-excavation. A non-woven subgrade separation geotextile meeting the requirements of ODOT OSSC Section 02320 (ODOT, 2021) should be placed between the Stone Embankment material and MSE wall backfill.

The MSE approach walls (i.e. outside of the spread footing influence zone) should be embedded a minimum of 2-feet below the lowest adjacent final grade in front of the wall; see Section 8.4. A minimum 4-foot wide bench should also be provided in front of the MSE walls in accordance with ODOT GDM (ODOT, 2018), Section 15.6.4.

It is important to note that the ODOT GDM (ODOT, 2018) prohibits the use of full-height precast concrete facing panels for MSE abutments.

8.3 Lateral Earth Pressure

The active earth pressure was calculated for MSE walls using the soil parameters for MSE Granular Wall Backfill presented in Exhibit 8-1. The soil parameters and wall geometry yield an active earth pressure coefficient of 0.28. A resultant calculated from the distributed active earth pressure can be placed H/3 up from the base of the wall. The active earth pressure can be evaluated as an equivalent fluid unit weight of 37 pcf. Additionally, the static surcharge pressure is calculated using the active earth pressure coefficient. We present these earth pressures in Figure 13, Recommended Lateral Pressures for MSE Walls.

The seismic active pressure coefficient, K_{AE}, was calculated using the Mononobe-Okabe Method, which uses the horizontal (k_h) and vertical seismic acceleration coefficient (k_v) in conjunction with the geometry of the retaining wall. A k_h equal to 1/2 of A_s (F_{pga} x PGA) was used to determine the seismic earth pressure for yielding walls, where 1 to 2 inches of lateral deformation is acceptable. Guidance provided in the ODOT GDM (ODOT, 2018) allows the use of zero for the vertical component (k_v). The calculation uses the equation A11.3.1-1 in the AASHTO LRFD Bridge Design Specifications (AASHTO, 2017). This yields seismic active pressure coefficients of 0.39 and 0.33 for the 1,000-year Life-Safety and Operational ground motion levels, respectively. We present these earth pressure increments and shape of loading in Figure 13.

8.4 Global Stability Analysis

This section discusses our global stability analyses for the MSE approach walls. Global stability analyses were conducted at the MSE abutments and discussed in Section 7.3.2.

We conducted global stability analyses for the proposed MSE approach walls using the computer program SLOPE/W, Version 11 (Geo-Slope International, 2021). This program employs limit-equilibrium methods in accordance with the ODOT GDM (ODOT, 2018). The Morgenstern-Price slope stability analysis method was used for rotational and irregular surface failure mechanisms. The analyses were performed at a specific location along the west approach wall alignment for static and seismic loading conditions. We understand stormwater ponds will be constructed in front of the east approach MSE walls however the grading for the ponds has not yet been determined. Global stability analyses for the east approach MSE walls will be performed once stormwater pond locations and grading are developed.

For the seismic condition, pseudo-static procedures described in the ODOT GDM (ODOT, 2018), Chapter 6, were followed. Horizontal acceleration coefficients equal to one-half of the site peak ground accelerations (0.5 x F_{pga} x PGA) were used. For our seismic slope stability analyses, we used horizontal seismic coefficients, k_h, equal to 0.09 and 0.173 for the "Operational" and "Life Safety" criteria, respectively. Only seismic global stability analyses considering "Life Safety" criteria are shown in our results, which we determined were the controlling ground motions in our evaluation.

The ODOT GDM (ODOT, 2018) requires that highway and bridge retaining walls be designed with a maximum resistance factor for global stability of 0.65, equivalent to an FS of 1.5, for static conditions. For seismic analyses, a maximum resistance factor of 0.9, or an FS of 1.1, is required.

We developed a critical cross section for global stability analysis based on wall heights and existing surface topography. Generalized subsurface conditions and soil parameters were determined from the results of the field explorations and laboratory testing. In accordance with the ODOT GDM (ODOT, 2018), the wall embedment was assumed to be 2 feet at the

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face of the wall. MSE wall reinforcement length was assumed to be 70 percent of the total wall height (0.7H) as measured from the top of the leveling pad to roadway grade, or 8 feet, whichever was greater.

Based on our analyses, the proposed west approach MSE walls designed following the recommendations in this report will satisfy the minimum global stability FS requirements for all conditions assuming the minimum geometric requirements detailed above are met. A minimum 4-foot wide bench should be provided in front of the walls in accordance with ODOT GDM (ODOT, 2018) Section 15.3.7. The results of our global stability analyses for the west approach MSE walls are presented in Figures C5 and C6 in Appendix C, Global Stability Analysis Results, and summarized in Exhibit 8-2.

Exhibit 8-2: Global Stability Analysis Results for MSE Approach Walls with 0.7H Reinforcement Length

Location	Analysis Case	Factor of Safety[1]	Minimum Factor of Safety Required by ODOT GDM
West Approach	Static	2.2	1.5
	Seismic	1.6	1.1

NOTE:

1 Factor of Safety reflects minimum MSE Wall geometries as discussed in Section 10.4.2.

8.5 MSE Wall Lateral Resistance

Resistance to lateral movement for an MSE wall consists of sliding friction. Passive soil pressures are neglected when calculating lateral resistance, as required by guidelines in the ODOT GDM (ODOT, 2018). We recommend that the sliding resistance evaluation follow the general requirements for an MSE wall in the ODOT GDM (ODOT, 2018) and AASHTO LRFD Section 11 (AASHTO, 2020). The nominal friction resistance may be expressed as the vertical load (at the base of the wall) multiplied by the coefficient of friction. We calculated the frictional sliding resistance coefficient for the wall assuming the reinforced material is sliding on an approved native soil subgrade; the soil strength parameters are provided previously in Exhibit 8-1. We recommend using a coefficient of friction equal to 0.49 at the MSE approach walls bearing on Fill, to calculate nominal sliding resistance for MSE-reinforced soil mass on approved subgrade. We recommend using a coefficient of friction equal to 0.67 at the MSE abutments bearing on the native Missoula Flood Deposits - Coarse unit or Stone Embankment material, to calculate nominal sliding resistance for MSE-reinforced soil mass on approved subgrade. For LRFD design, a resistance factor of 1.0 should be used in calculation of friction sliding resistance for the strength and extreme event limit states.

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8.6 MSE Approach Wall Foundation Bearing Resistance and Settlement

For the MSE walls outside of the bridge abutment's footing influence zone, we expect that the majority of the proposed walls will be founded on Fill. We recommend that all MSE walls have a minimum embedment of 2 feet below lowest adjacent final grade in front of the wall. We performed bearing resistance analysis in accordance with the ODOT GDM (ODOT, 2018) and AASHTO LRFD (AASHTO, 2020). In LRFD design, the strength and extreme event limit state bearing resistances are obtained by selecting appropriate soil strength parameters and computing a nominal bearing pressure at which shear failure of the bearing soil would likely occur. The nominal bearing resistance multiplied by the appropriate resistance factor gives the factored bearing resistance. The factored bearing resistances for strength and extreme limit states, as a function of reinforcement length, are presented on Figures 14 and 15 for the west and east MSE approach walls, respectively. Resistance factors of 0.65 and 0.9 are used for the strength and extreme event limit states, respectively.

The service limit state was not evaluated because we have estimated the settlement induced by the placement of the proposed MSE wall fill directly. We estimate less than 1 inch of settlement at the west approach MSE walls and less than 2 inches of settlement at the east approach walls, based on the maximum height of the wall of 13 feet and a unit weight of the fill material of 130 pcf. We anticipate the settlement will occur during wall construction (i.e. fill placement).

8.7 MSE Abutment Foundation Bearing Resistance and Settlement

For the MSE abutments, we expect that the proposed walls will be founded on native Coarse-Grained Missoula Flood Deposits or Stone Embankment material. We recommend that all MSE walls have a minimum embedment of 2 feet below lowest adjacent final grade in front of the wall. However, additional wall embedment may be required (see Section 8.2). We performed bearing resistance analysis at the MSE abutments in accordance with the ODOT GDM (ODOT, 2018) and AASHTO LRFD (AASHTO, 2020). For the recommended spread footing bearing resistance, refer to Section 7.5. In LRFD design, the strength and extreme event limit state bearing resistances are obtained by selecting appropriate soil strength parameters and computing a nominal bearing pressure at which shear failure of the bearing soil would likely occur. The nominal bearing resistance multiplied by the appropriate resistance factor gives the factored bearing resistance. The factored bearing resistances for strength and extreme limit states, as a function of reinforcement length, are presented on Figure 16 for the MSE Abutments. Resistance factors of 0.65 and 0.9 are used for the strength and extreme event limit states, respectively.

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The service limit state was not evaluated because we have estimated the settlement induced by the placement of the proposed MSE wall fill directly. At the MSE abutments, we estimate less than 1 inch of settlement will occur during wall construction (i.e. placement of the fill).

8.8 MSE Wall Drainage

Proper drainage is necessary for long-term stability of the MSE walls. Backfill placed immediately behind the MSE wall reinforcement zone should be free-draining, granular material in accordance with ODOT OSSC, Section 00510.12 (ODOT, 2021). Specifically, MSE wall internal drainage design should be in accordance with ODOT GDM (ODOT, 2018), Section 15.6.8.

8.9 MSE Wall Construction Considerations

8.9.1 Excavation and Subgrade Preparation

Earthwork should be performed in accordance with ODOT OSSC, Section 00330 (ODOT, 2021). See Section 9 of this report for general geotechnical construction considerations, including excavation and subgrade preparation.

8.9.2 MSE Wall Leveling Pad

A leveling pad is an unreinforced concrete pad generally used to begin the facing construction if concrete fascia panels are used; this allows a uniform, level starting point to place the fascia panels and on which to build upward. The surface of the leveling pad should be smooth and horizontal, both side-to-side and front-to-back, to ensure the fascia panel courses are level.

9 GEOTECHNICAL CONSTRUCTION CONSIDERATIONS

9.1 Site Preparation and Excavation

Site preparation will include (1) clearing, grubbing, and roadside cleanup; (2) removal of existing structures and underground utilities; and (3) subgrade preparation and excavation. These construction activities should generally be accomplished in accordance with the ODOT OSSC (ODOT, 2021). If temporary shoring is needed, the design of such shoring is traditionally the responsibility of the contractor.

After site stripping and preparation activities are completed, the exposed subgrade to receive fill should be proof-rolled with a fully loaded 10- to 12-yard dump truck or similar

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heavy rubber-tired construction equipment to identify soft, loose, or unsuitable areas. The proof-roll should be conducted prior to fill placement.

The site stripping and proof-roll should be observed by a qualified geotechnical engineer or representative, who should determine stripping depth, evaluate the suitability of subgrade, and identify areas of yielding. If loose and/or wet, soft soil zones are identified during proof-rolling, the soils should be removed and replaced with compacted structural fill.

Disturbance of subgrade soil due to construction equipment and activities could affect support of the proposed walls and embankment. The contractor should take necessary steps to protect subgrade from becoming disturbed.

9.2 Temporary Cut-and-Fill Slopes

Temporary cut slopes are typically the responsibility of the Contractor and should comply with applicable local, state, and federal safety regulations, including the current Occupational Safety and Health Administration (OSHA) Excavation and Trench Safety Standards. For general guidance, we suggest that temporary construction slopes be made at 1H:1V or flatter. In areas of loose fills, very soft soil, or groundwater seepage, flatter slopes are likely to be required.

9.3 Temporary Shoring

Based upon the explored subsurface conditions, our opinion is that sheet pile walls are not a feasible alternative for temporary shoring at the I-5 Pedestrian Bridge project site due to potential obstructions on cobbles and boulders, as well as the dense surficial materials encountered in the subsurface explorations. However, driven or drilled-in soldier piles may be a feasible alternative for temporary shoring.

9.4 Potential Construction Vibration Impacts

We understand one commercial structure (Eye Health Northwest Clinic) is within 85 feet of the proposed bridge east abutment, and within 75 feet of the proposed MSE approach walls. Construction vibration on the nearby existing building may be a concern, especially since the building may hold vibration sensitive equipment.

For preliminary evaluation of pile driving vibration impacts, we estimate a peak particle velocity of 0.2 inches per second at the nearby potentially vibration sensitive commercial property (EyeHealth Northwest), which is 85 feet away from the nearest proposed pile driving location.

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For preliminary evaluation of the vibration impacts resulting from MSE wall construction, particularly during material compaction with a vibratory roller, we estimate a peak particle velocity of 0.15 inches per second at the nearby potentially vibration sensitive commercial property (EyeHealth Northwest), which is 75 feet away from the nearest proposed pile driving location.

Based on published studies (Caltrans, 2020; Woods, 1997), a typical threshold to prevent structural damage to fragile buildings is 0.2 inches per second for transient sources and 0.1 inches per second for continuous sources, such as vibratory compaction. For structures that are under normal conditions, and do not have structural deficiency, a typical "conservative" limit to prevent damage is 0.5 inches per second for transient sources and 0.25 inches per second for continuous sources, and the widely accepted threshold for damage is 2 inches per second. However, the above vibration criteria may not be acceptable for the Eye Health Northwest clinic where vibration sensitive equipment may be in use and eye surgeries are performed. We recommend the Agency contact Eye Health Northwest to discuss potential construction vibration impacts on their operations. Depending on the tolerable vibration level, some construction work at the east approach may need to be performed during the nighttime or outside of business hours, or the City may need to perform vibration monitoring.

We recommend the contractor, at a minimum, perform a pre-construction survey of the Eye Health Northwest clinic to document pre-construction conditions and evaluate any possible post-construction building and/or equipment damage.

10 LIMITATIONS

The analyses, conclusions, and recommendations contained in this report are based on site conditions as they presently exist, and further assume that the explorations are representative of the subsurface conditions throughout the site; that is, the subsurface conditions everywhere are not significantly different from those disclosed by the explorations. If subsurface conditions different from those encountered in the explorations are encountered or appear to be present during construction, we should be advised at once so that we can review these conditions and reconsider our recommendations, where necessary. If there is a substantial lapse of time between the submission of this report and the start of construction 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.

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Within the limitations of scope, schedule, and budget, the analyses, 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 conclusions and recommendations were based on our understanding of the project as described in this report and the site conditions as observed at the time of our explorations.

Unanticipated soil conditions are commonly encountered and cannot be fully determined by merely taking soil samples from test borings. Such unexpected conditions frequently require that additional expenditures be made to attain a properly constructed project. Therefore, some contingency fund is recommended to accommodate such potential extra costs.

This report was prepared for the exclusive use of DOWL, LLC, and their design team in the design and construction of the I-5 Pedestrian Bridge: Barber St. to Wilsonville Town Center project. This document is not suitable for use in final design and should not be provided to prospective contractors. Our report, conclusions, and interpretations should not be construed as a warranty of subsurface conditions included in this report.

The scope of our present work did not include environmental assessments or evaluations regarding the presence or absence of wetlands, or hazardous or toxic substances in the soil, surface water, 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.

Please read the Important Information Section at the back of this report to reduce your project risks.

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Table 1 - Comparison of I-5 Wilsonville Pedestrian Bridge Foundation Alternatives

Foundation	Description	Advantages	Disadvantages
Driven Pipe Piles at Abutments and Interior Bents	16- or 20-inch diameter open-ended pipe piles driven to bear in the Coarse-Grained Missoula Flood Deposits - Coarse or Pliocene/Pleistocene Sediments.	- Less risk of post-construction settlement compared to MSE abutments. - Does not require MSE walls to be founded on native gravels. - More economical alternative and faster construction compared to drilled shafts at interior bents.	- Risk of encountering early refusal before minimum pile tip elevation is reached due to potential boulders and cobbles within the Coarse-Grained Missoula Flood Deposits. - If additional piles or relocation of piles are required due to early refusal, re-design of pile cap and/or MSE wall may be needed. - Pile driving vibration and noise may impact and disturb nearby residences and businesses. - Pile cap may require larger foundation footprint at interior bents compared to drilled shafts.
Drilled Shafts at Interior Bents	Dual 5 or 6.5-foot diameter drilled shafts at the interior bents. Bearing in the Coarse-Grained Missoula Flood Deposits or Pliocene/Pleistocene Sediments.	- Reduced vibration and noise impacts compared to driven piles. - Higher level of control of construction variability compared to driven piles and spread footings. - Minimal risk of relocating foundation elements due to refusal on cobbles and boulders within Coarse Grained Missoula Flood Deposits. - Less foundation footprint required compared to driven piles.	- More expensive than driven piles. - Higher construction QA/QC requirements. - Relatively longer construction duration compared to driven piles. - Requires a specialty contractor.
Spread Footings at Interior Bents	Spread footings founded on Coarse-Grained Missoula Flood Deposits at Bents 2, 3, and 8.	- Reduced vibration and noise impacts compared to driven piles. - Conventional construction. - Potentially more economical than driven piles and drilled shafts.	- Higher excavation volume compared to other foundation alternatives. - Ground anchors drilled into Coarse Grained Missoula Flood Deposits may be required to provide additional resistance to footing rocking during seismic loading. - Variable blow counts in Coarse-Grained Missoula Flood Deposits presents risk of variable bearing resistance. - Requires larger foundation footprint compared to driven piles and drilled shafts. - Temporary shoring may be required for excavation to native gravel.
MSE Abutments	Continuous spread footing constructed on top of MSE walls at Bents 1 and 9.	- Reduced vibration and noise impacts compared to driven piles. - Conventional construction. - Construction upon MSE walls reduces risk of variable bearing resistance in Coarse-Grained Missoula Flood Deposits.	- Higher excavation volume compared to other foundation alternatives. - Overexcavation is required to found MSE walls on native gravel. - Variable fill thickness presents risk of increased overexcavation depth for MSE walls. - Risk of post-construction settlement if MSE walls are not properly constructed in accordance with ODOT standard specifications.

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Table 1 - Recommended LPILE Geotechnical Input Parameters for I-5 Wilsonville Pedestrian Bridge

Bent I.D.	Nearest Borehole	Approximate Top of Layer El. (NAVD88)[1]	Generalized Soil Unit	Recommended P-Y Curve	Effective Unit Weight (pcf)	Friction Angle (deg)	p-y modulus k (pci)	Undrained Cohesion (psf)	e50
Bent 5	B-3	175	Median Fill	Stiff Clay w/o Free Water (Reese)	110	--	--	1900	0.0065
		168	Roadway Fill	Sand (Reese)	120	33	150	--	--
		163	MD to VD Missoula Flood Deposits (Coarse)	Sand (Reese)	125	36	150	--	--
		140	MD to VD Missoula Flood Deposits (Coarse)	Sand (Reese)	63	36	90	--	--
		122.5	Upper Pliocene/Pleistocene Sediments	Sand (Reese)	53	28	50	--	--
		112.5	Pleistocene Alluvium (Cohesive)	Stiff Clay w/o Free Water (Reese)	58	--	--	3500	0.004
Bents 6, 7, 8 and 9	B-4 and B-5	171	Median Fill	Stiff Clay w/o Free Water (Reese)	110	--	--	1900	0.0065
		165	MD to VD Missoula Flood Deposits (Coarse)	Sand (Reese)	125	36	150	--	--
		140	MD to VD Missoula Flood Deposits (Coarse)	Sand (Reese)	63	36	90	--	--
		129	Upper Pliocene/Pleistocene Sediments	Sand (Reese)	53	28	50	--	--
		119	Pleistocene Alluvium (Cohesive)	Stiff Clay w/o Free Water (Reese)	58	--	--	3500	0.004
		98	Lower Pliocene/Pleistocene Sediments	Sand (Reese)	58	36	115	--	--

NOTES:

1 Top of Layer Elevation is based off the approximate ground surface elevation at each boring. If ground surface at bent location is greater than top of first layer elevation, extrapolate top of first layer to top of foundation elevation.

deg = degrees; pcf = pounds per cubic foot; pci = pounds per cubic inch

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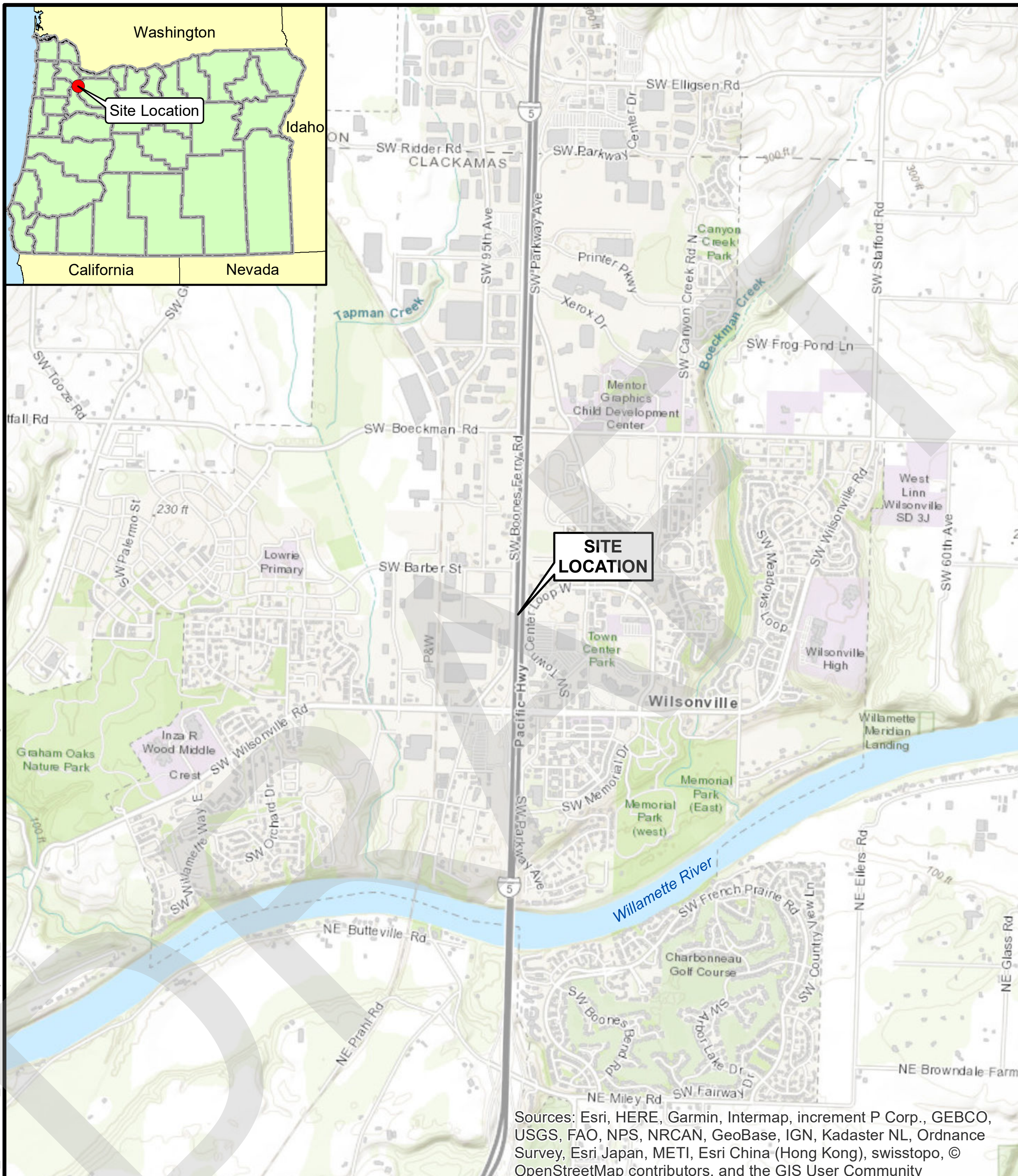
Table 2 - Recommended LPILE Geotechnical Input Parameters for I-5 Wilsonville Pedestrian Bridge

Bent I.D.	Nearest Borehole	Approximate Top of Layer El. (NAVD88)[1]	Generalized Soil Unit	Recommended P-Y Curve	Effective Unit Weight (pcf)	Friction Angle (deg)	p-y modulus k (pci)	Undrained Cohesion (psf)	e50
Bent 1	B-1 and B-2	165	Median Fill	Stiff Clay w/o Free Water (Reese)	110	--	--	1900	0.0065
		159	MD to VD Missoula Flood Deposits (Coarse)	Sand (Reese)	125	36	150	--	--
		148	MD Missoula Flood Deposits (Coarse)	Sand (Reese)	120	34	75	--	--
		140	MD Missoula Flood Deposits (Coarse)	Sand (Reese)	58	34	50	--	--
		132.5	MD to VD Missoula Flood Deposits (Coarse)	Sand (Reese)	63	36	90	--	--
		112.5	Upper Pliocene/Pleistocene Sediments	Sand (Reese)	53	28	50	--	--
		105	Lower Pliocene/Pleistocene Sediments	Sand (Reese)	58	36	115	--	--
Bents 2, 3, and 4	B-2	165	Median Fill	Stiff Clay w/o Free Water (Reese)	110	--	--	1900	0.0065
		160	MD to VD Missoula Flood Deposits (Coarse)	Sand (Reese)	125	36	150	--	--
		148	MD Missoula Flood Deposits (Coarse)	Sand (Reese)	120	34	75	--	--
		140	MD Missoula Flood Deposits (Coarse)	Sand (Reese)	58	34	50	--	--
		132.5	MD to VD Missoula Flood Deposits (Coarse)	Sand (Reese)	63	36	90	--	--
		107.5	Upper Pliocene/Pleistocene Sediments	Sand (Reese)	53	28	50	--	--
		97.5	Lower Pliocene/Pleistocene Sediments	Sand (Reese)	58	36	115	--	--



0 0.25 0.5 1

Scale in Miles

N

I-5 Pedestrian Bridge
Barber St. to Wilsonville Town Center
Wilsonville, Oregon

VICINITY MAP

November 2020 103953

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FIG. 1

Filename: T:\Projects\PDX\103000s\103953_Wilsonville_I-5\Avmxd\Vicinity Map_10.6.mxd Date: 11/16/2020 Login: AEH

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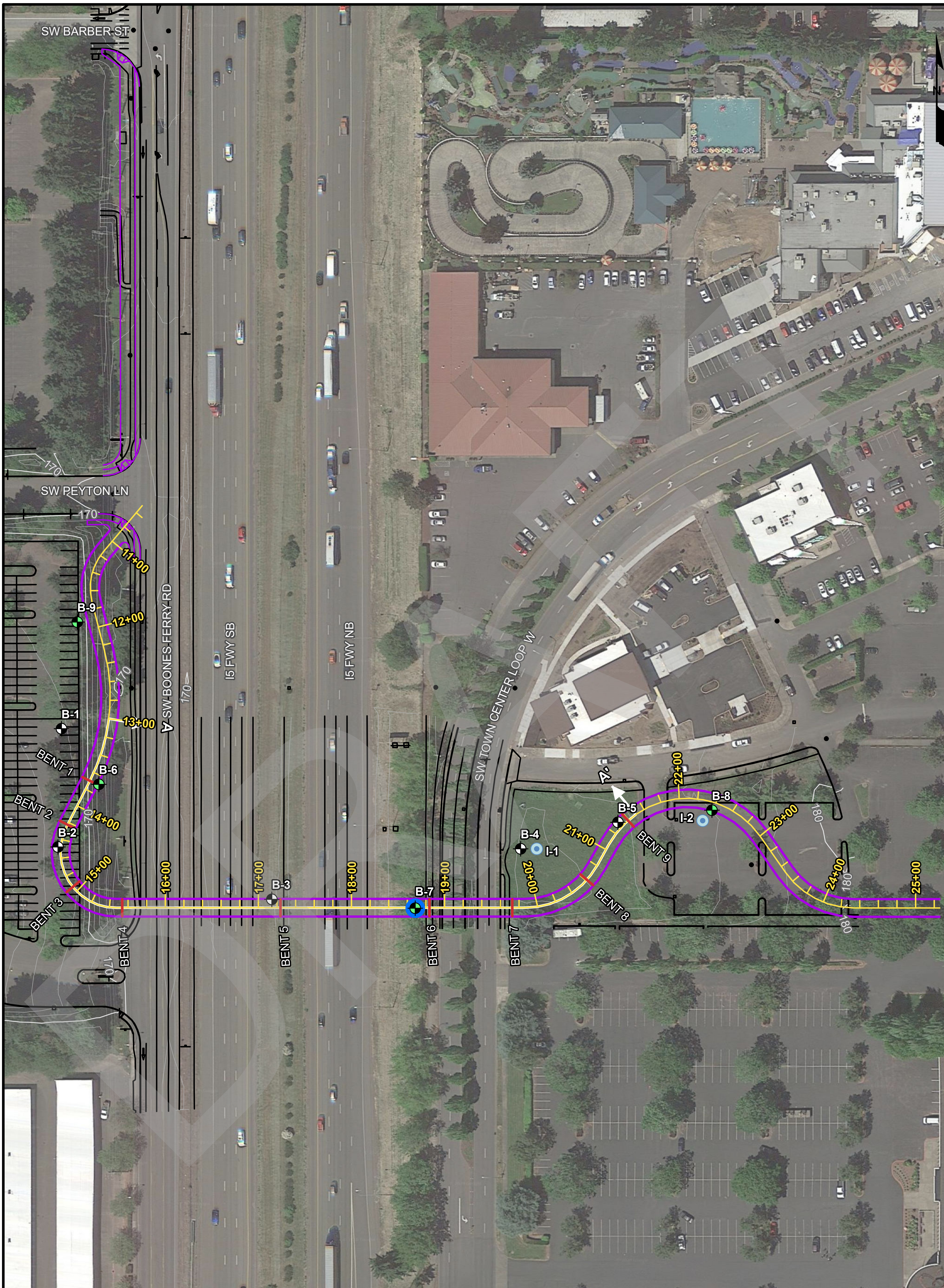
Filename: T:\Projects\PDX\103000s\103953_Wilsonville_I-5\Avmxd\Site Plan GeoPortrait_10.6.mxd Date: 11/30/2020 Login: AEH



LEGEND

- B-1 Designation and Approximate Location of Boring
- B-6 Designation and Location of Proposed Boring (20' Deep)
- B-7 Designation and Location of Proposed Boring with Piezometer (50' Deep)
- I-1 Designation and Location of Proposed Infiltration Test (7-10' Deep)
- Approximate Proposed Bent Location
- A A' Location and Designation of Interpretive Subsurface Profile

0 50 100 200

Scale in Feet

NOTES

1. Aerial imagery obtained through Google Maps Satellite.
2. Existing contours and features from file I-5 Pedestrian Bridge_Topo Basemap.dwg, provided by DOWL on September 9, 2020.
3. Proposed alignment and roadway features from file 02560042_dd01.dwg, provided by DOWL on November 4, 2020.

I-5 Pedestrian Bridge
Barber St. to Wilsonville Town Center
Wilsonville, Oregon

SITE AND EXPLORATION PLAN

November 2020 103953

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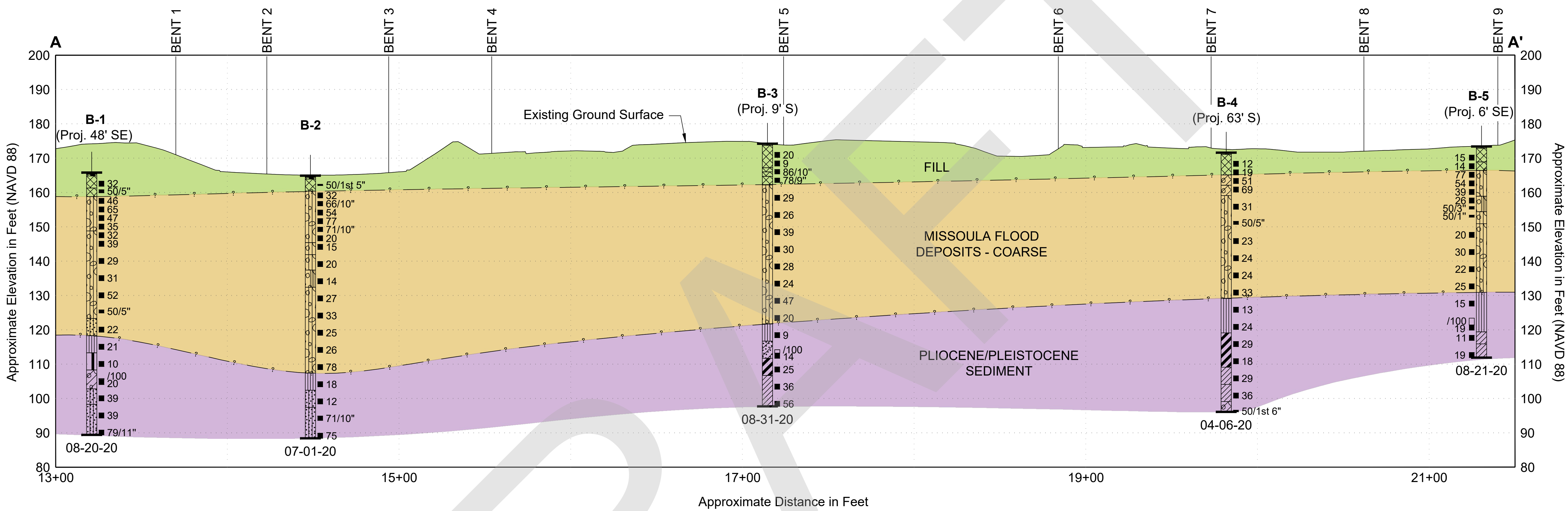
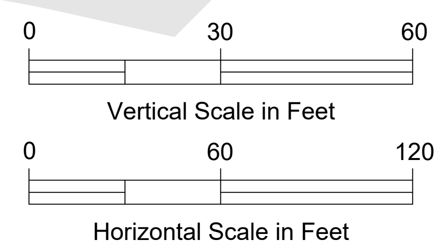
FIG. 2

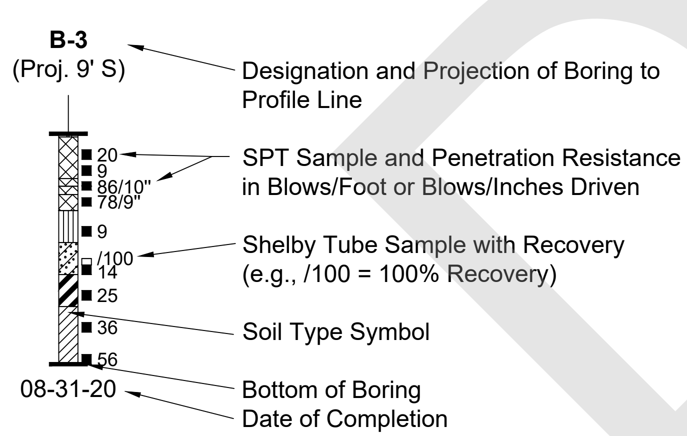
FIG. 2





Vertical Exaggeration = 2x

LEGEND



NOTES

1. Ground surface generated from surface in file I-5 Pedestrian Bridge_Topo Basemap.dwg, provided by DOWL on September 9, 2020.
2. Profile generalized from materials observed in borings. Variations may exist between profile and actual conditions. See Appendix A for complete boring logs and explanations of symbols.
3. See Figure 2 for profile location.
4. Boring locations and elevations are approxiamate.
5. Proposed bent locations based on email from DOWL on November 4, 2020.

I-5 Pedestrian Bridge
Barber St. to Wilsonville Town Center
Wilsonville, Oregon

INTERPRETIVE SUBSURFACE PROFILE A-A'

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FIG. 3

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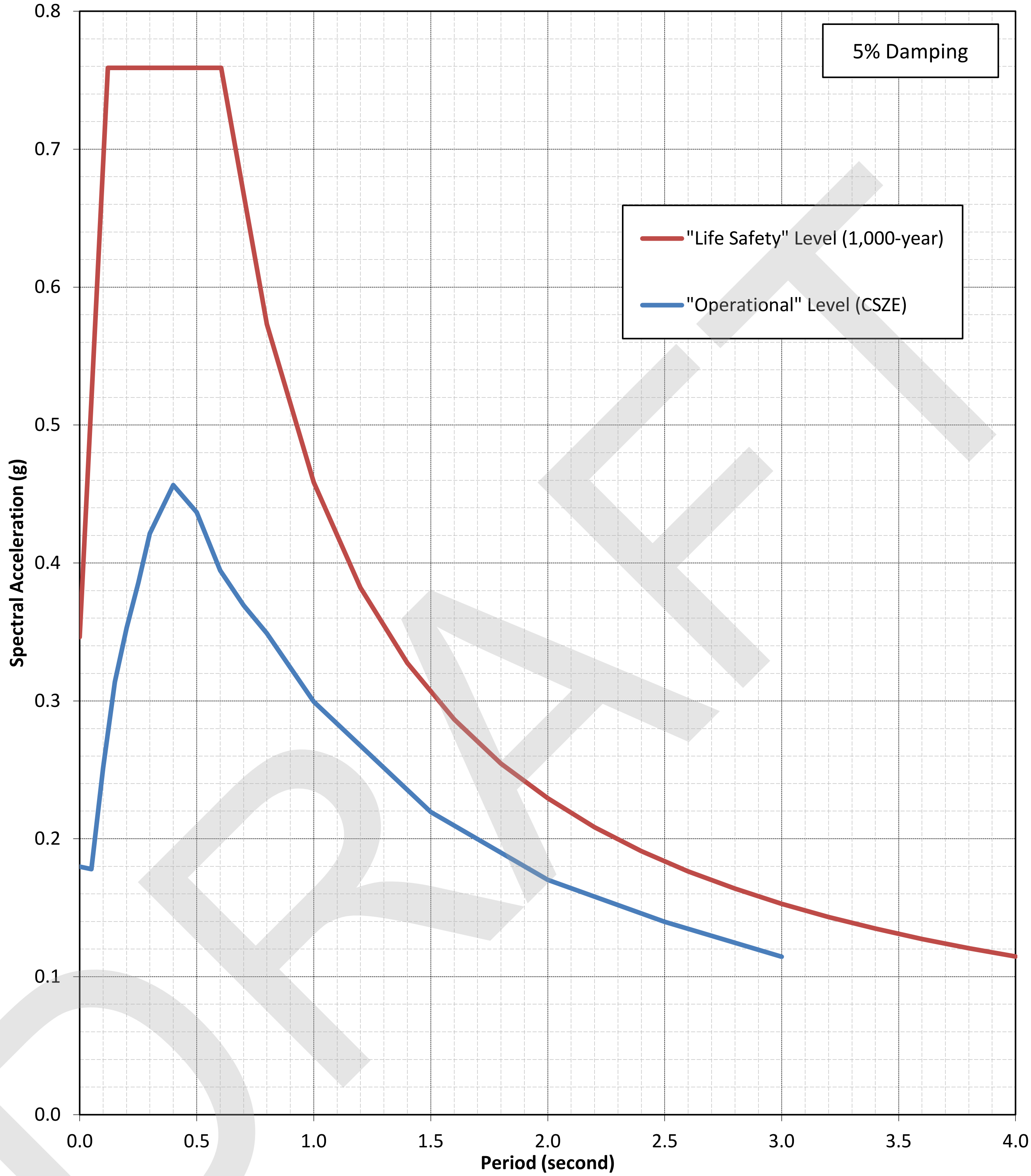
File: I:\EF\PDX\103000s\103953 Wilsonville I-5\DRAFTING\Profile.dwg Date: 11-17-2020 Author: AEH



NOTES

1. "Life Safety" 1,000-year and "Operational" CSZE spectra were calculated using the ODOT Excel Application [v2014.16] and Portland State University CSZE Web Tool, respectively.

2 g = acceleration due to gravity;

I-5 Pedestrian Bridge
Barber St. to Wilsonville Town Center
Wilsonville, Oregon

RECOMMENDED ACCELERATION RESPONSE SPECTRA SITE CLASS D

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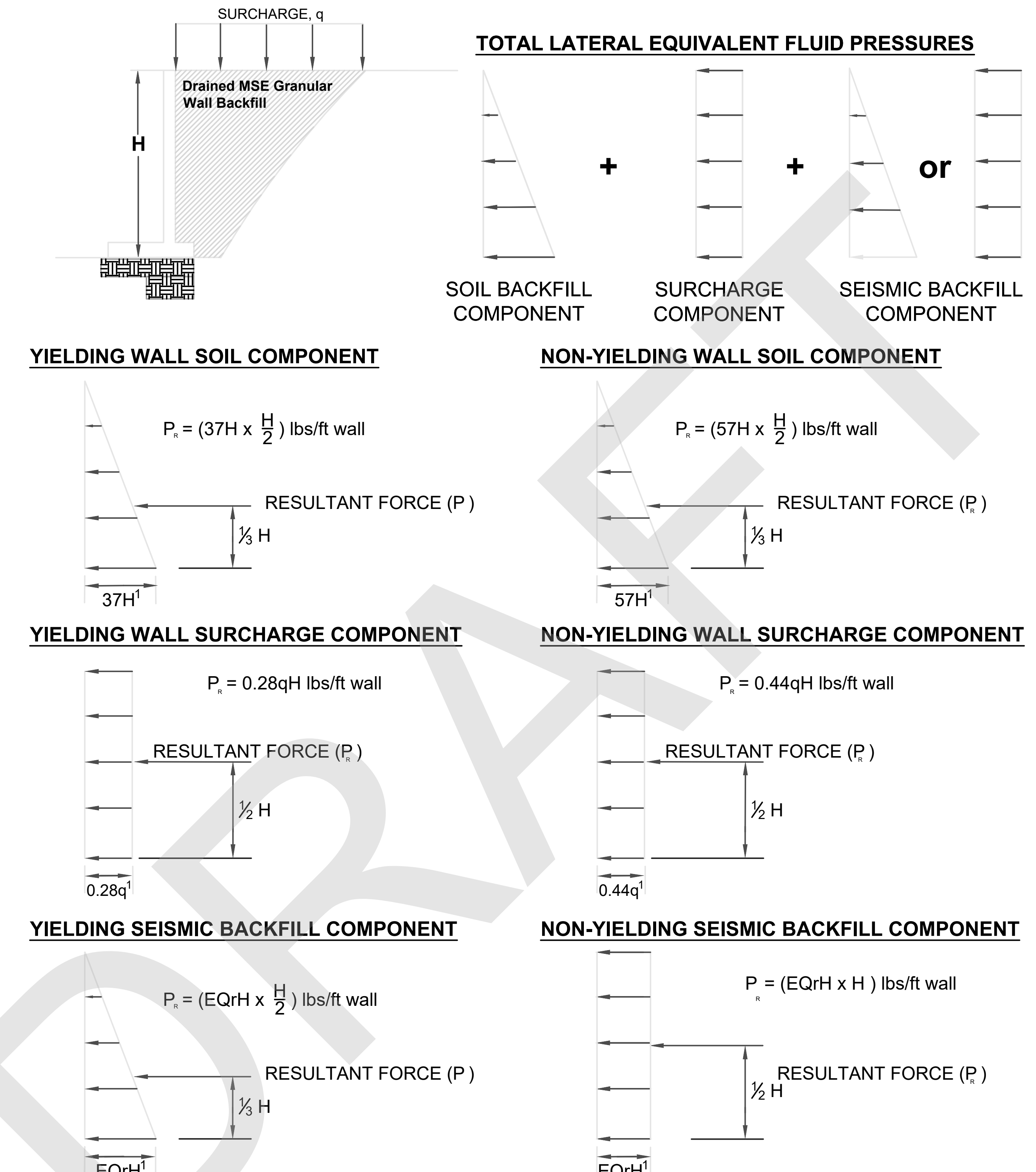
FIG. 4



NOTES
1. Units are pounds per square foot (psf).
2. Backfill unit weight of 130 pcf.
3. Backfill friction angle is 34 deg.
4. Retained wall backfill is assumed to be drained MSE granular wall backfill material.
5. Seismic pressures provided for peak ground accelerations associated with a 1,000-year earthquake ("Life Safety" criteria) and the CSZE ("Operational" critera). See Table 1 for values.

TABLE 1

EQ LEVEL	YIELDING EQr (pcf)	NON YIELDING EQr (pcf)
CSZE	7	8
1000	14	17

I-5 Pedestrian Bridge
Barber St. to Wilsonville Town Center
Wilsonville, Oregon

RECOMMENDED LATERAL PRESSURES FOR ABUTMENTS AND WING WALLS

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FIG. 5

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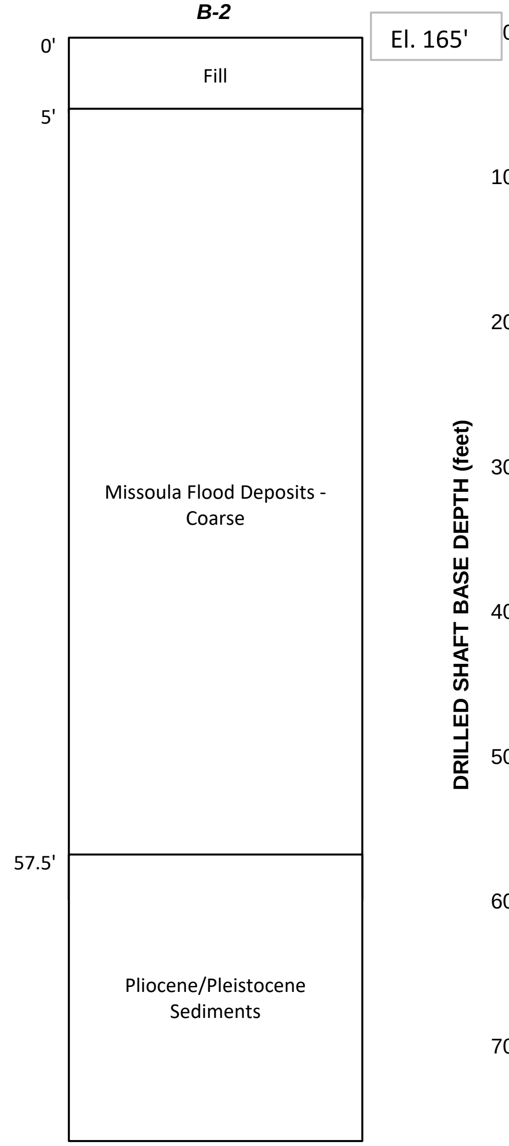
File: I:\EF\PDX\103000s\103953 Wilsonville I-5\DRAFTING\LEP Diagrams.dwg Date: 11-30-2020 Author: DBP

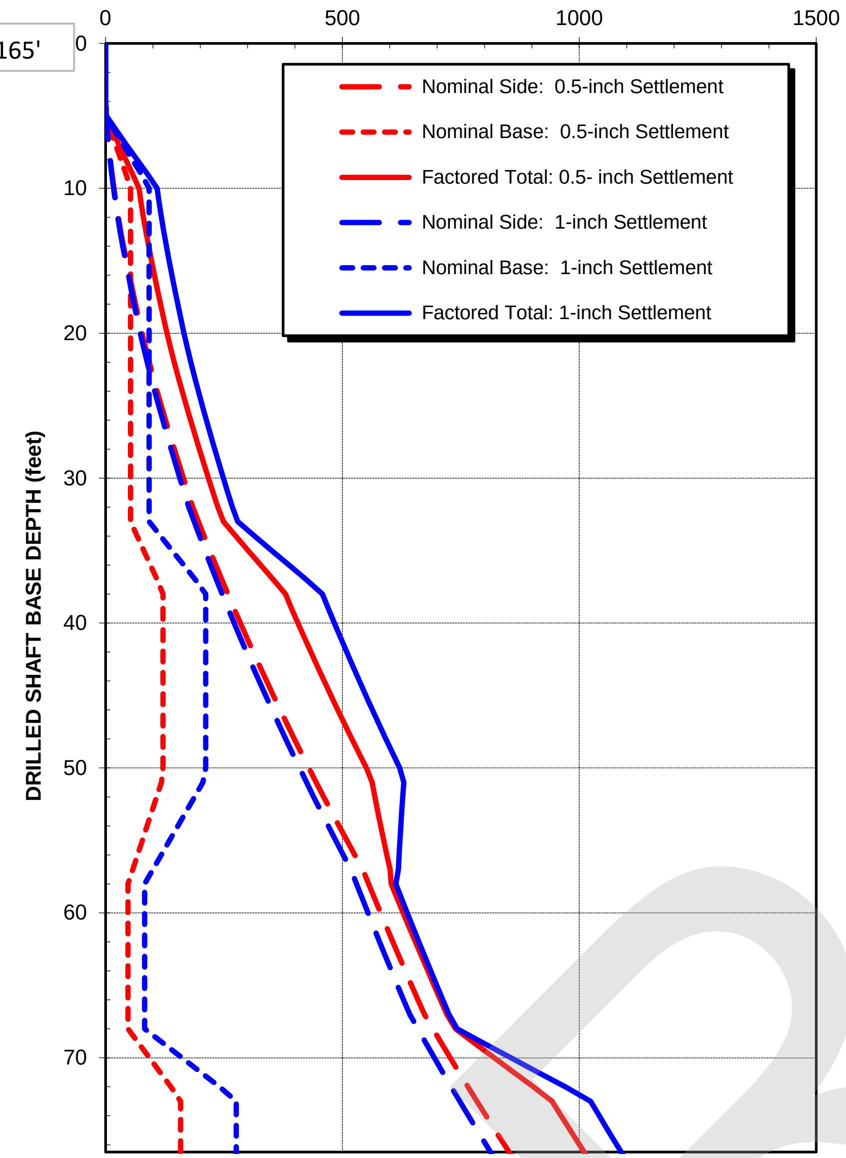
ASSUMED SUBSURFACE PROFILE

Based on Nearby Explorations:
B-2



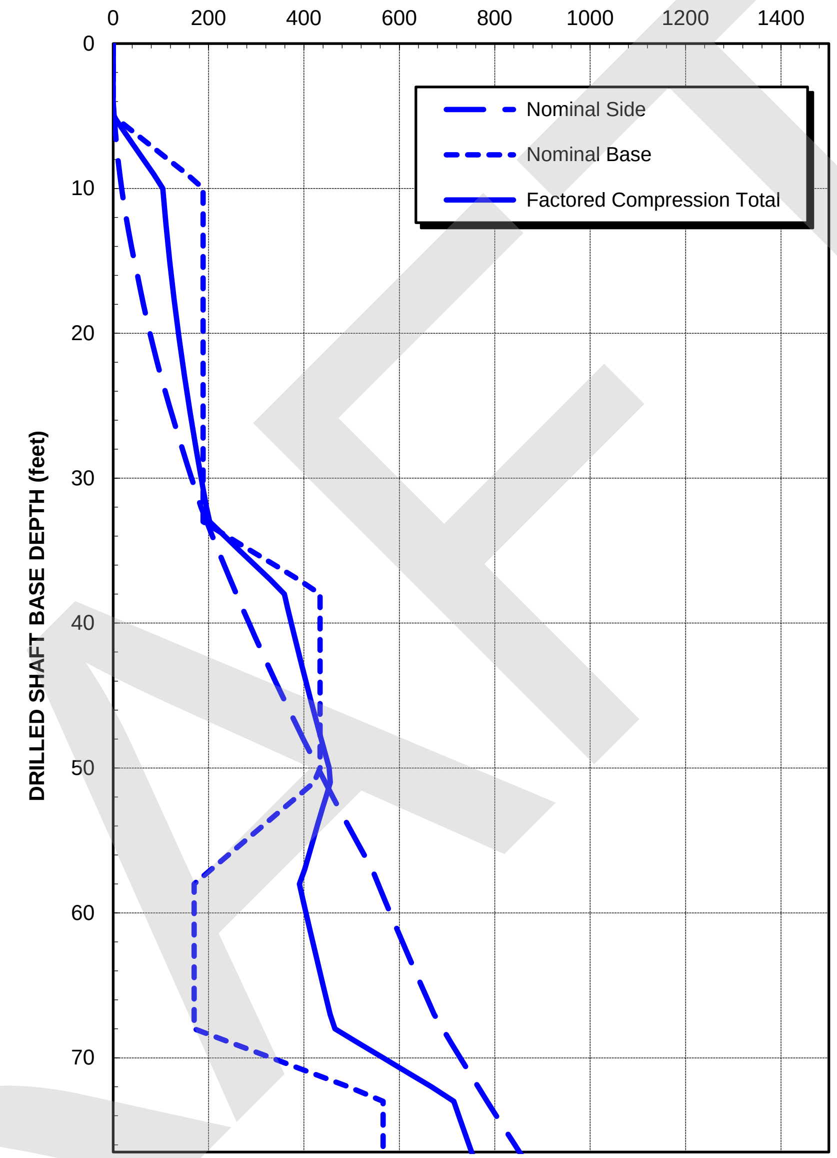
SERVICE LIMIT

NOMINAL RESISTANCE (tons)



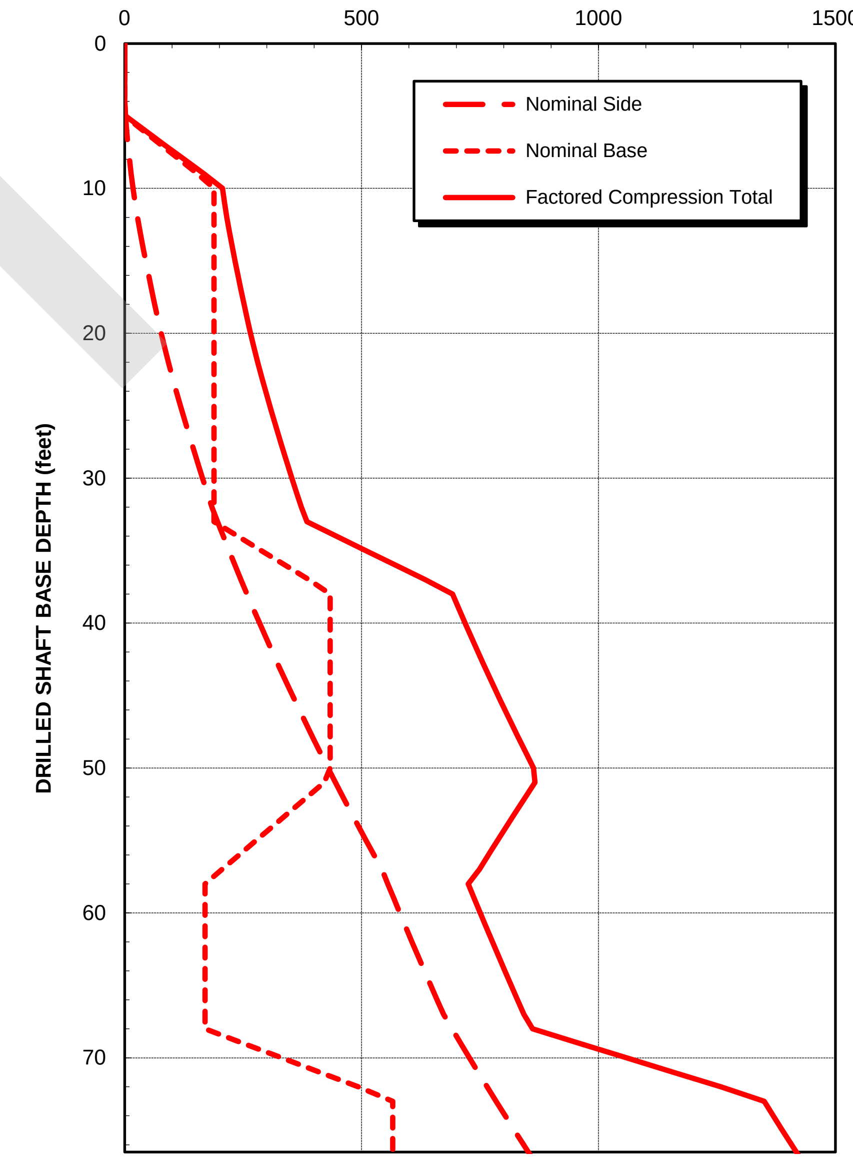
STRENGTH LIMIT

NOMINAL RESISTANCE (tons)



EXTREME EVENT LIMIT

NOMINAL RESISTANCE (tons)



SERVICE LIMIT NOTES:

1. Recommended resistance factors per ODOT GDM are 1.0 for both side and base resistance.
2. Settlement is based on a single shaft. No group action is considered.

STRENGTH LIMIT NOTES:

1. Recommended compression resistance factors per ODOT GDM are 0.55 and 0.5 for side and base resistance, respectively.
2. Shaft uplift resistance can be estimated by using the nominal side resistance shown above and a recommended resistance factor of 0.45 (per ODOT GDM).

EXTREME EVENT LIMIT NOTES:

1. Recommended resistance factors per ODOT GDM for both side and base resistance are 1.0 for compression and 0.8 for uplift.

GENERAL NOTES

1. The analyses were performed based on guidelines included in the AASHTO LRFD Bridge Design Specifications (AASHTO LRFD) and local experience. The analyses are based on a single shaft and do not consider group action of closely spaced shafts (closer than 4 diameters, center to center).
2. Factored total shaft resistance shown on plots is determined by adding its nominal side and base resistances multiplied by the appropriate resistance factors as noted above.
3. Estimated shaft resistance assumes that if casing is used, it will be removed after the shaft installation. If, however, the casing is left in place, grouting should be used to fill all potential voids around the casing and the estimated resistance given above should be re-evaluated.

I-5 Pedestrian Bridge
Barber St. to Wilsonville Town Center
Wilsonville, Oregon

ESTIMATED AXIAL SHAFT RESISTANCE
5-FOOT DIAMETER DRILLED SHAFT
BENTS 2 & 3

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FIG. 6

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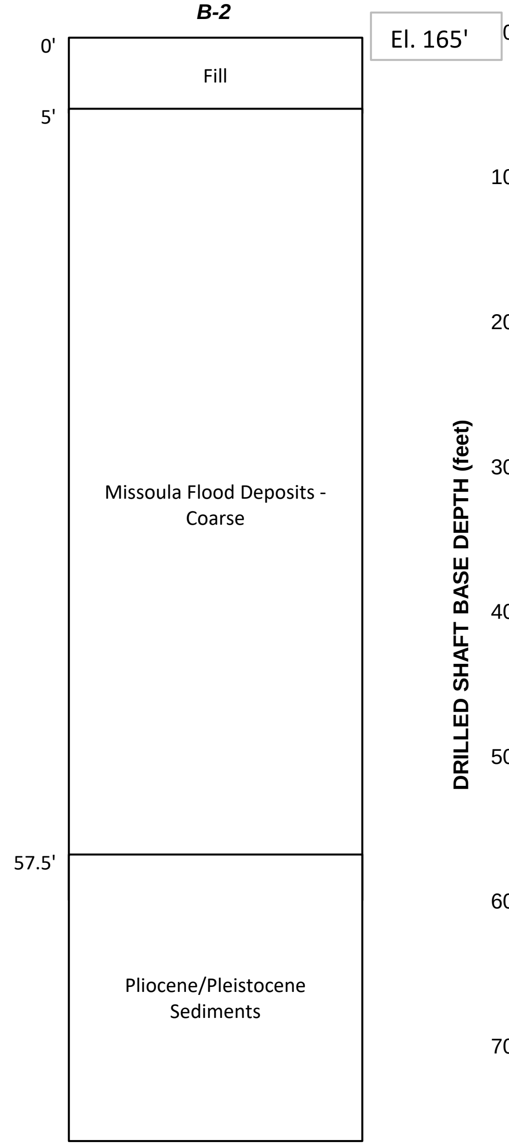
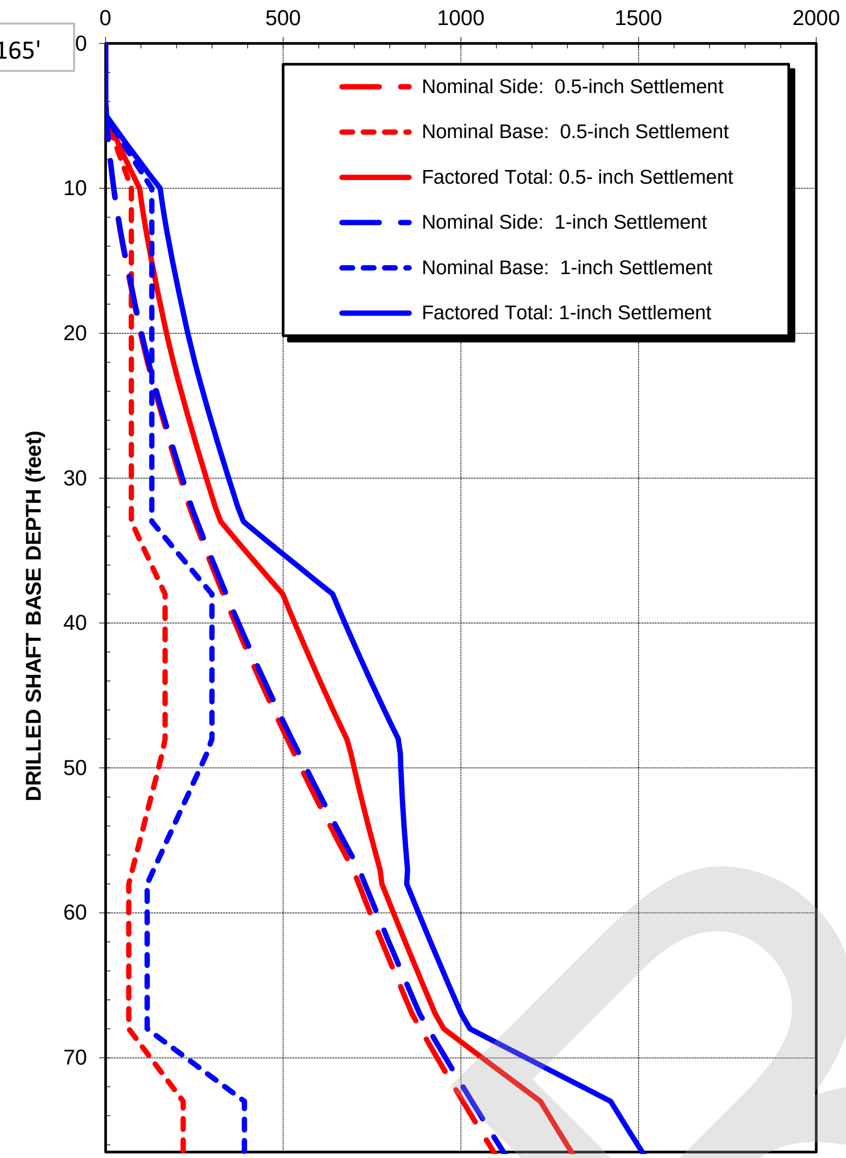
ASSUMED SUBSURFACE PROFILE

Based on Nearby Explorations:
B-2



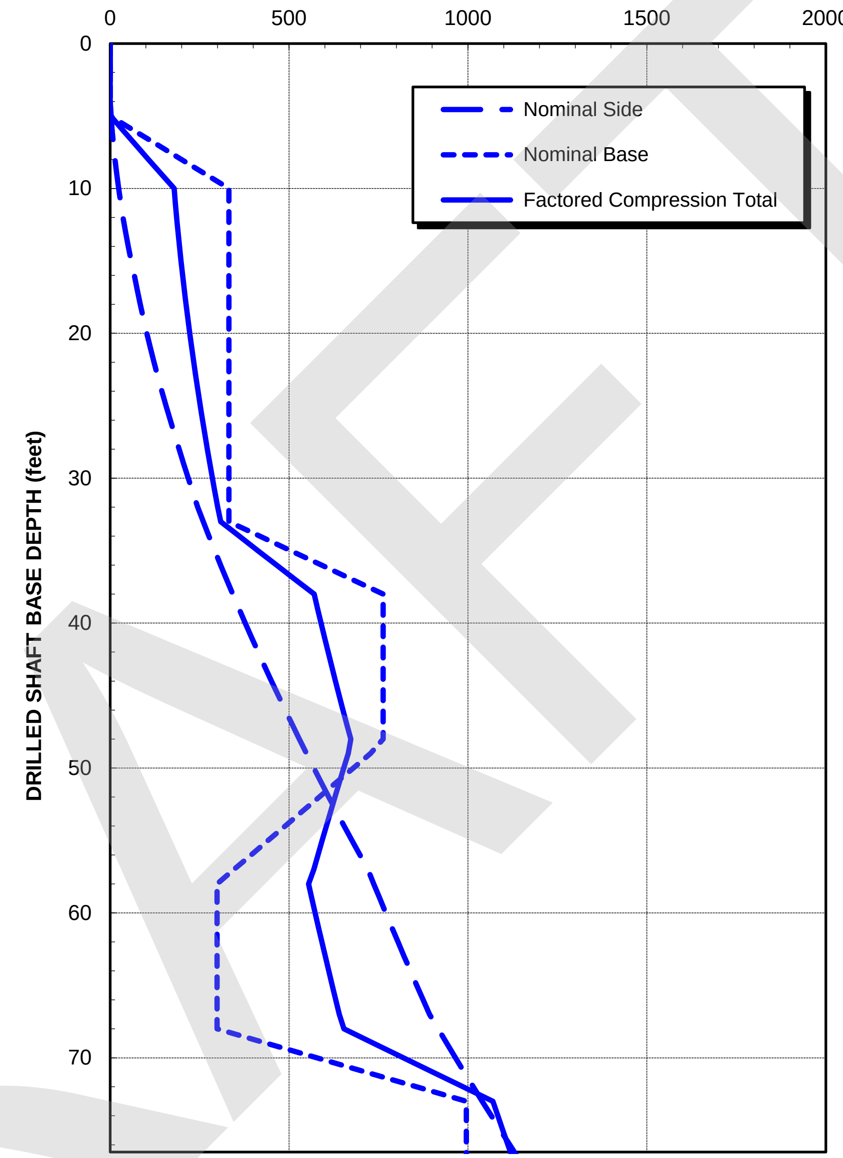
SERVICE LIMIT

NOMINAL RESISTANCE (tons)



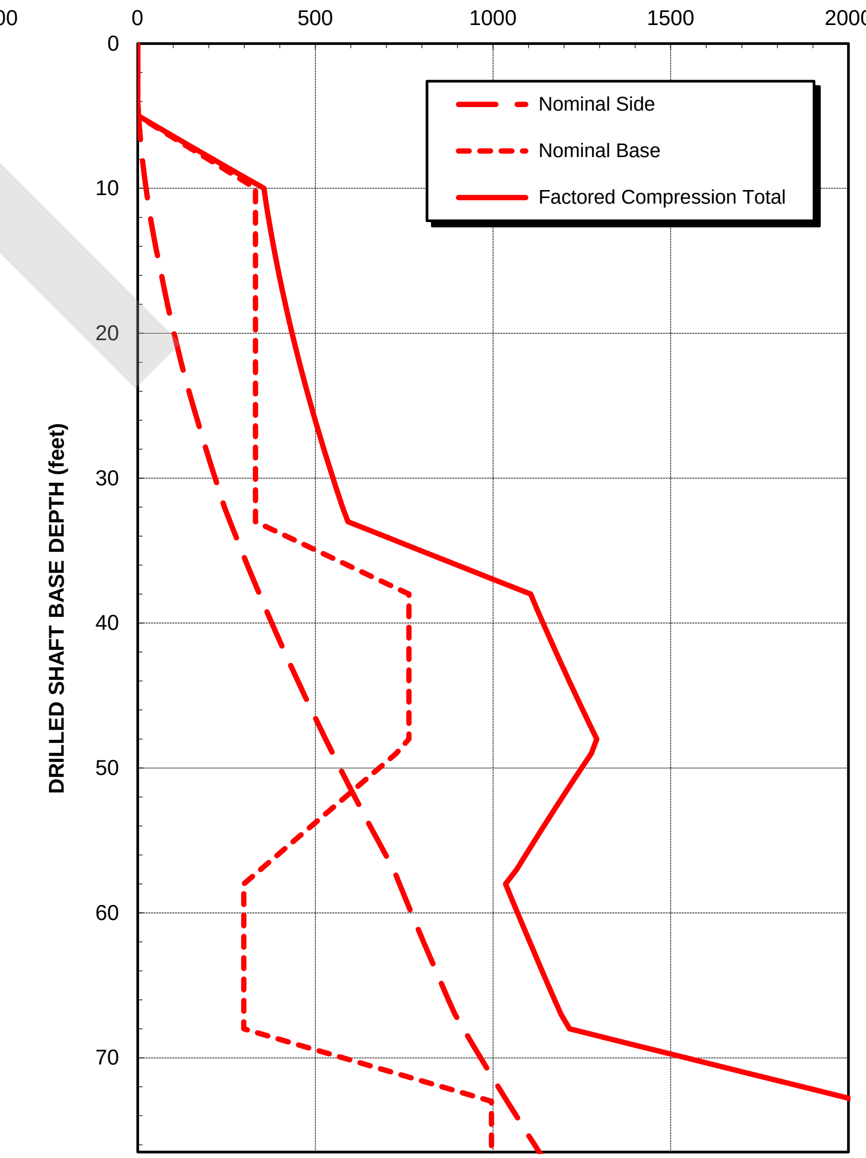
STRENGTH LIMIT

NOMINAL RESISTANCE (tons)



EXTREME EVENT LIMIT

NOMINAL RESISTANCE (tons)



SERVICE LIMIT NOTES:

1. Recommended resistance factors per ODOT GDM are 1.0 for both side and base resistance.
2. Settlement is based on a single shaft. No group action is considered.

STRENGTH LIMIT NOTES:

1. Recommended compression resistance factors per ODOT GDM are 0.55 and 0.5 for side and base resistance, respectively.
2. Shaft uplift resistance can be estimated by using the nominal side resistance shown above and a recommended resistance factor of 0.45 (per ODOT GDM).

EXTREME EVENT LIMIT NOTES:

1. Recommended resistance factors per ODOT GDM for both side and base resistance are 1.0 for compression and 0.8 for uplift.

GENERAL NOTES

1. The analyses were performed based on guidelines included in the AASHTO LRFD Bridge Design Specifications (AASHTO LRFD) and local experience. The analyses are based on a single shaft and do not consider group action of closely spaced shafts (closer than 4 diameters, center to center).
2. Factored total shaft resistance shown on plots is determined by adding its nominal side and base resistances multiplied by the appropriate resistance factors as noted above.
3. Estimated shaft resistance assumes that if casing is used, it will be removed after the shaft installation. If, however, the casing is left in place, grouting should be used to fill all potential voids around the casing and the estimated resistance given above should be re-evaluated.

I-5 Pedestrian Bridge
Barber St. to Wilsonville Town Center
Wilsonville, Oregon

ESTIMATED AXIAL SHAFT RESISTANCE
6.5-FOOT DIAMETER DRILLED SHAFT
BENTS 2, 3, & 4

November 2020 103953

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FIG. 7

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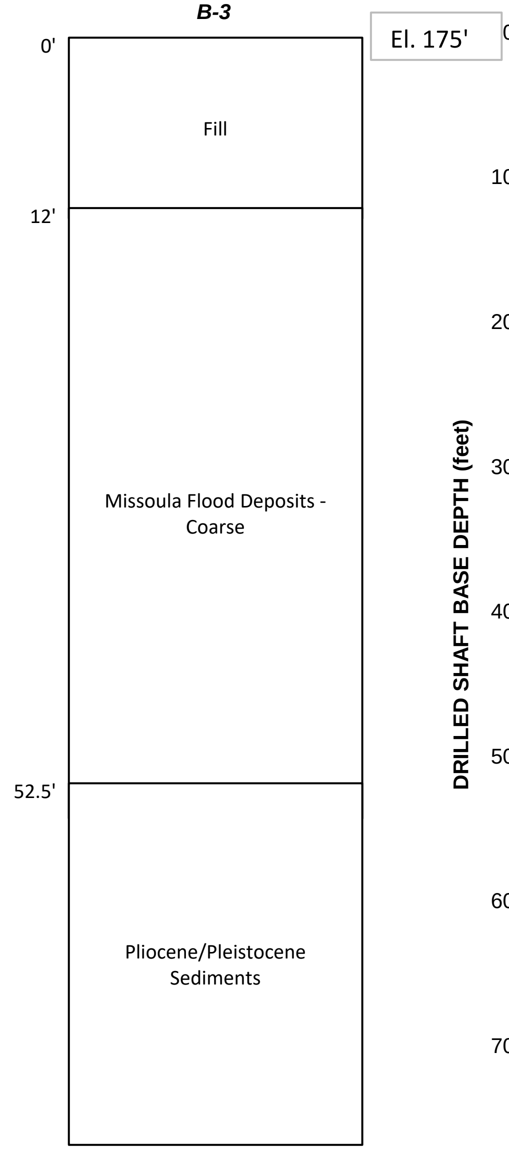
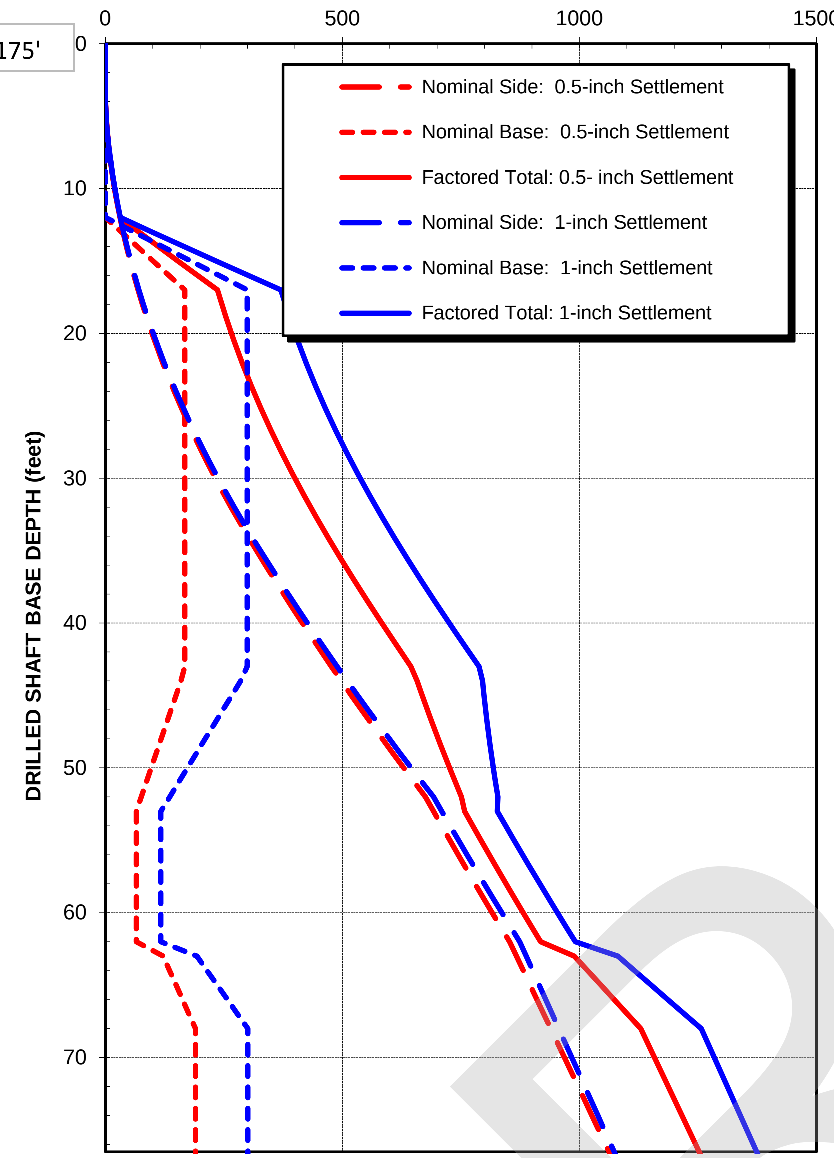
ASSUMED SUBSURFACE PROFILE

Based on Nearby Explorations:
B-3



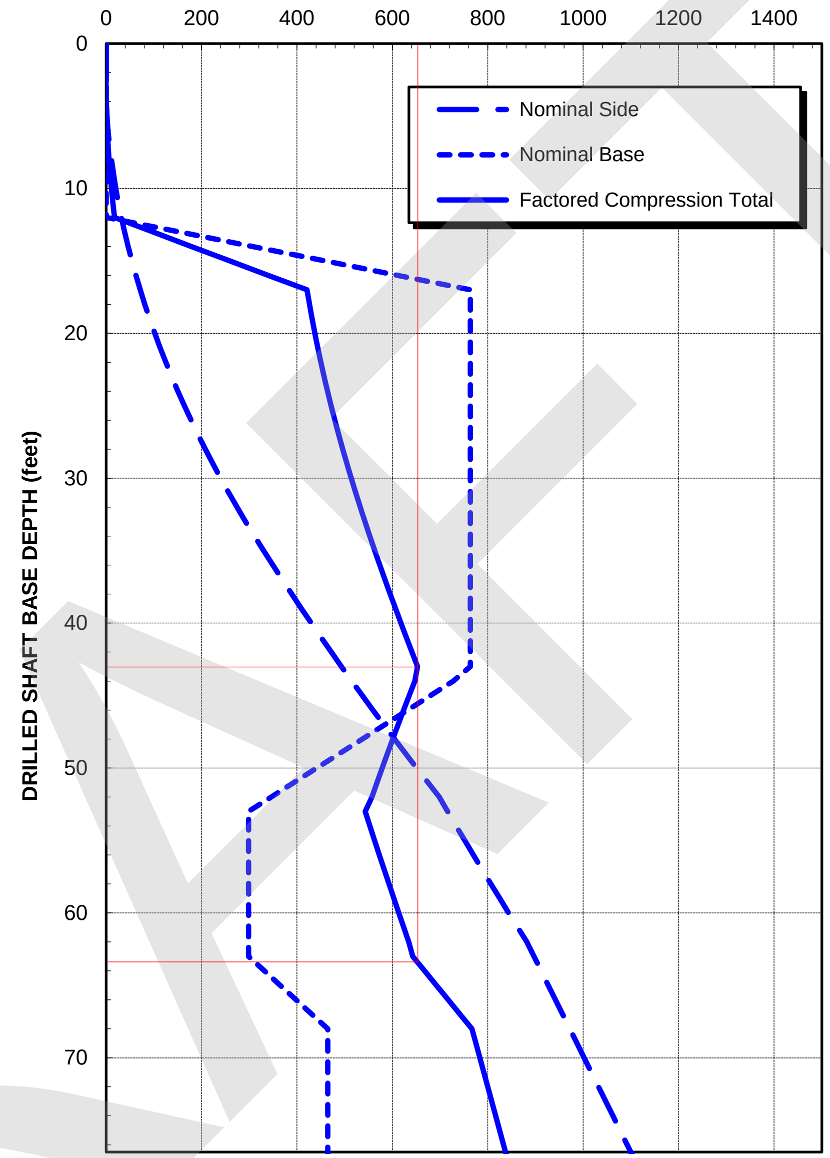
SERVICE LIMIT

NOMINAL RESISTANCE (tons)



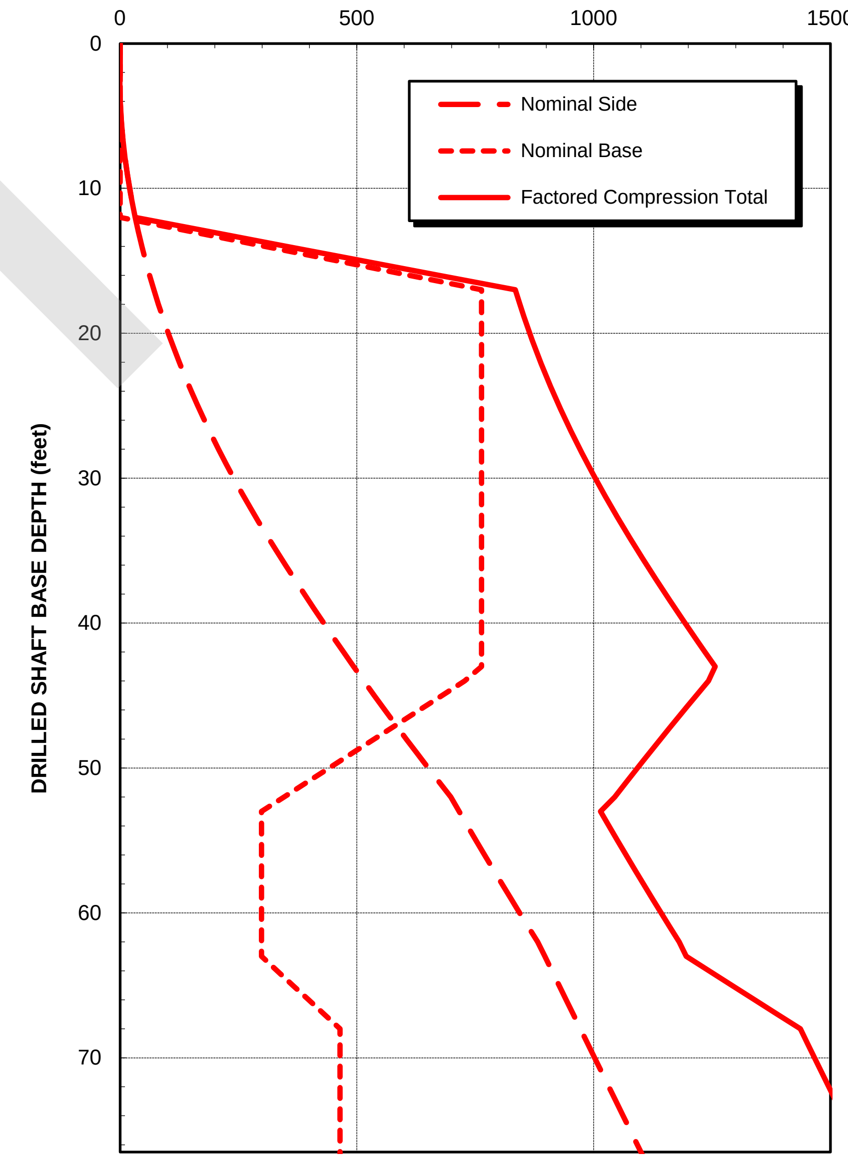
STRENGTH LIMIT

NOMINAL RESISTANCE (tons)



EXTREME EVENT LIMIT

NOMINAL RESISTANCE (tons)



SERVICE LIMIT NOTES:

1. Recommended resistance factors per ODOT GDM are 1.0 for both side and base resistance.
2. Settlement is based on a single shaft. No group action is considered.

STRENGTH LIMIT NOTES:

1. Recommended compression resistance factors per ODOT GDM are 0.55 and 0.5 for side and base resistance, respectively.
2. Shaft uplift resistance can be estimated by using the nominal side resistance shown above and a recommended resistance factor of 0.45 (per ODOT GDM).

EXTREME EVENT LIMIT NOTES:

1. Recommended resistance factors per ODOT GDM for both side and base resistance are 1.0 for compression and 0.8 for uplift.

GENERAL NOTES

1. The analyses were performed based on guidelines included in the AASHTO LRFD Bridge Design Specifications (AASHTO LRFD) and local experience. The analyses are based on a single shaft and do not consider group action of closely spaced shafts (closer than 4 diameters, center to center).
2. Factored total shaft resistance shown on plots is determined by adding its nominal side and base resistances multiplied by the appropriate resistance factors as noted above.
3. Estimated shaft resistance assumes that if casing is used, it will be removed after the shaft installation. If, however, the casing is left in place, grouting should be used to fill all potential voids around the casing and the estimated resistance given above should be re-evaluated.

I-5 Pedestrian Bridge
Barber St. to Wilsonville Town Center
Wilsonville, Oregon

ESTIMATED AXIAL SHAFT RESISTANCE
6.5-FOOT DIAMETER DRILLED SHAFT
BENT 5

November 2020 | 103953

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FIG. 8

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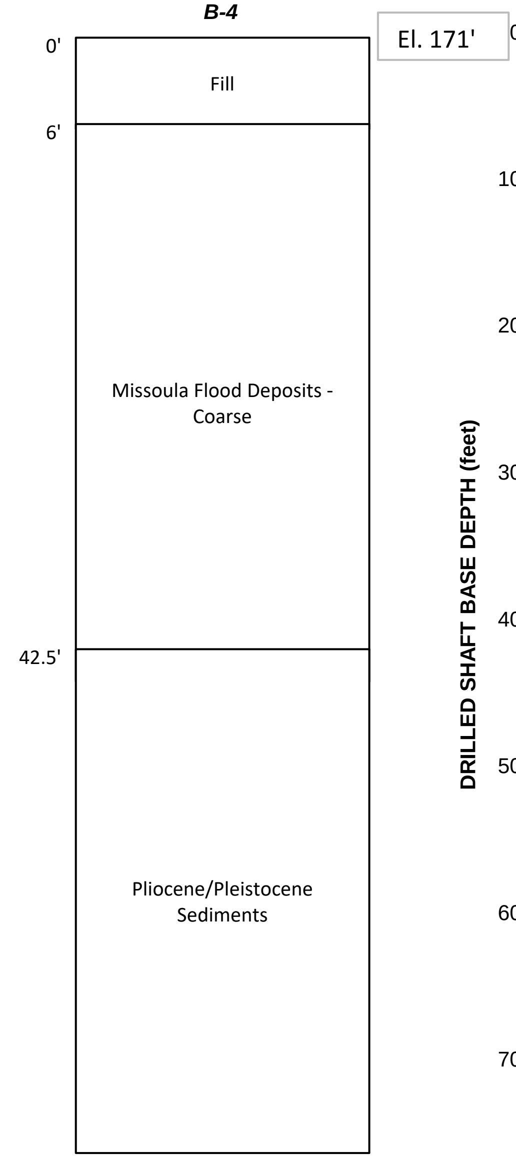
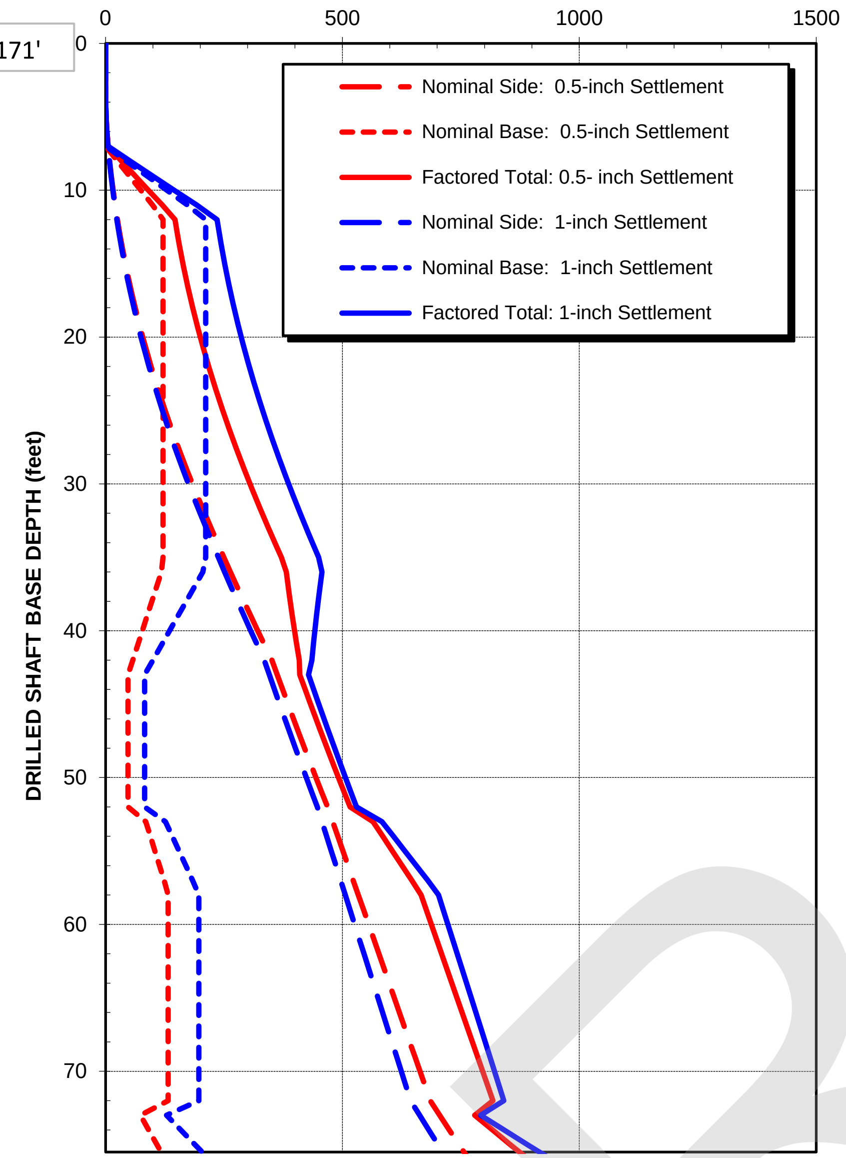
ASSUMED SUBSURFACE PROFILE

Based on Nearby Explorations:
B-4



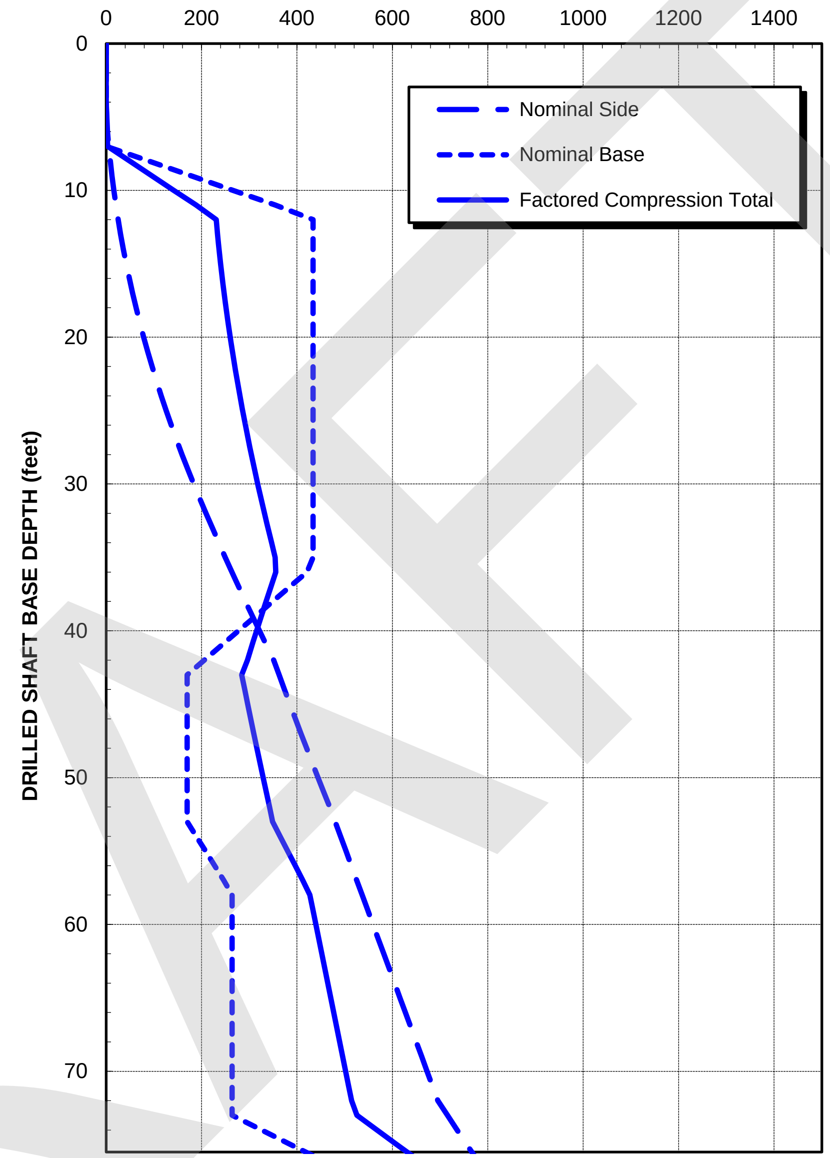
SERVICE LIMIT

NOMINAL RESISTANCE (tons)



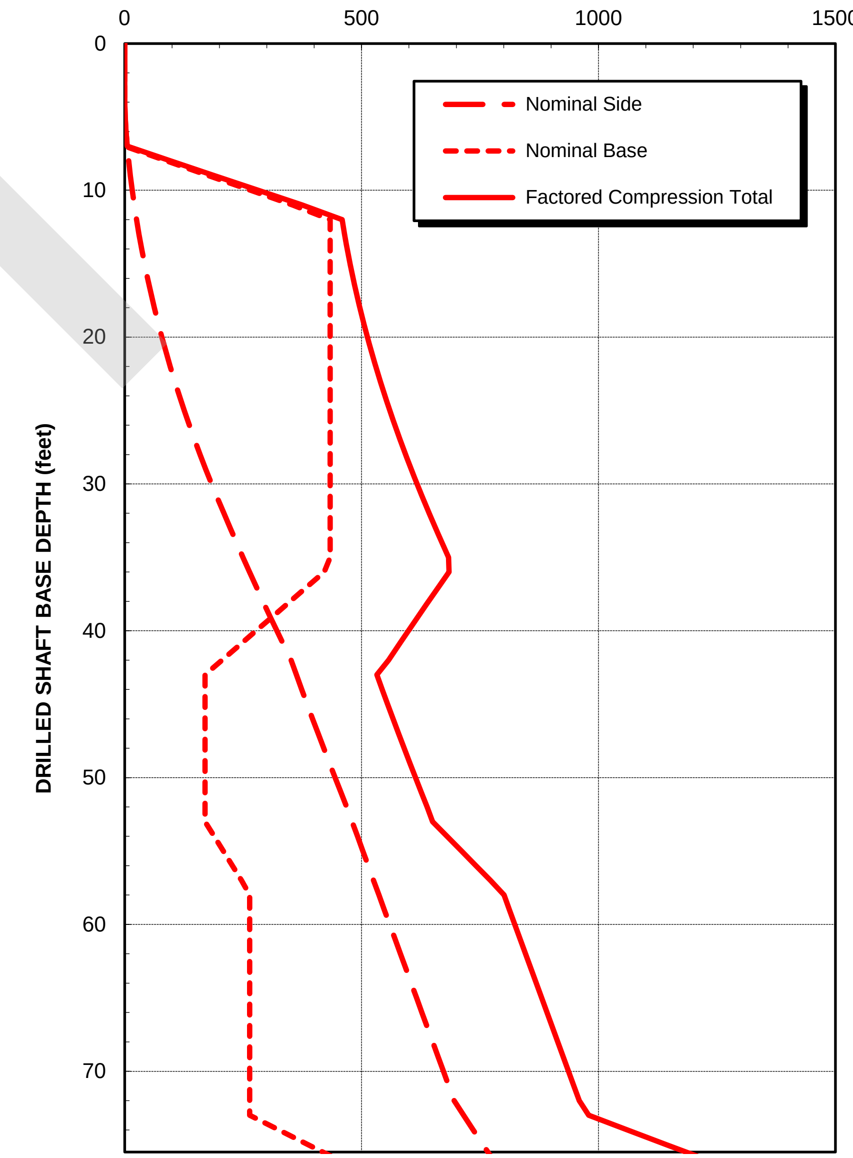
STRENGTH LIMIT

NOMINAL RESISTANCE (tons)



EXTREME EVENT LIMIT

NOMINAL RESISTANCE (tons)



SERVICE LIMIT NOTES:

1. Recommended resistance factors per ODOT GDM are 1.0 for both side and base resistance.
2. Settlement is based on a single shaft. No group action is considered.

STRENGTH LIMIT NOTES:

1. Recommended compression resistance factors per ODOT GDM are 0.55 and 0.5 for side and base resistance, respectively.
2. Shaft uplift resistance can be estimated by using the nominal side resistance shown above and a recommended resistance factor of 0.45 (per ODOT GDM).

EXTREME EVENT LIMIT NOTES:

1. Recommended resistance factors per ODOT GDM for both side and base resistance are 1.0 for compression and 0.8 for uplift.

GENERAL NOTES

1. The analyses were performed based on guidelines included in the AASHTO LRFD Bridge Design Specifications (AASHTO LRFD) and local experience. The analyses are based on a single shaft and do not consider group action of closely spaced shafts (closer than 4 diameters, center to center).
2. Factored total shaft resistance shown on plots is determined by adding its nominal side and base resistances multiplied by the appropriate resistance factors as noted above.
3. Estimated shaft resistance assumes that if casing is used, it will be removed after the shaft installation. If, however, the casing is left in place, grouting should be used to fill all potential voids around the casing and the estimated resistance given above should be re-evaluated.

I-5 Pedestrian Bridge
Barber St. to Wilsonville Town Center
Wilsonville, Oregon

ESTIMATED AXIAL SHAFT RESISTANCE
5-FOOT DIAMETER DRILLED SHAFT
BENTS 7 & 8

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FIG. 9

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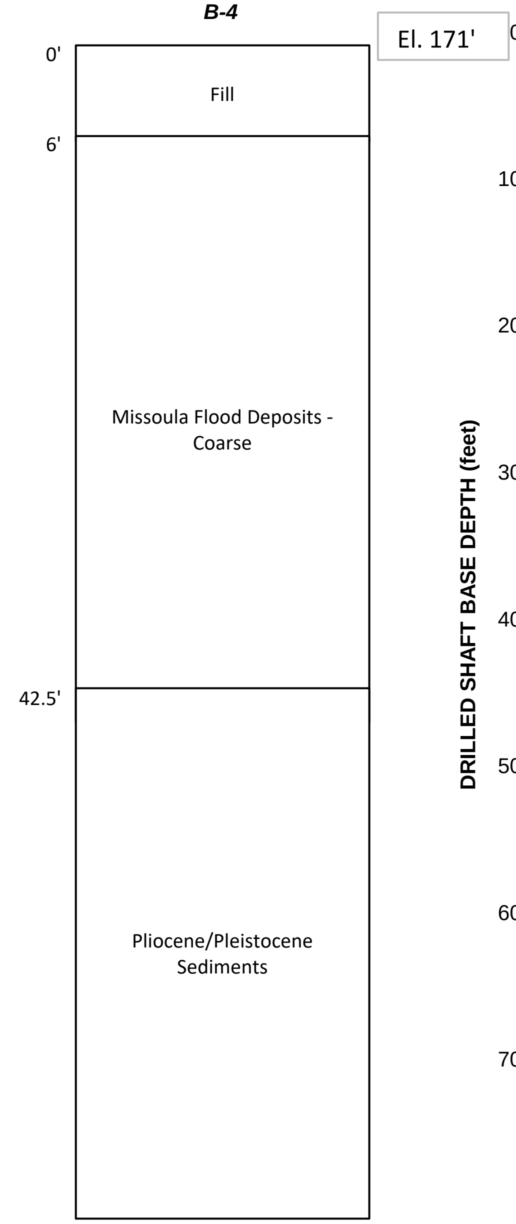
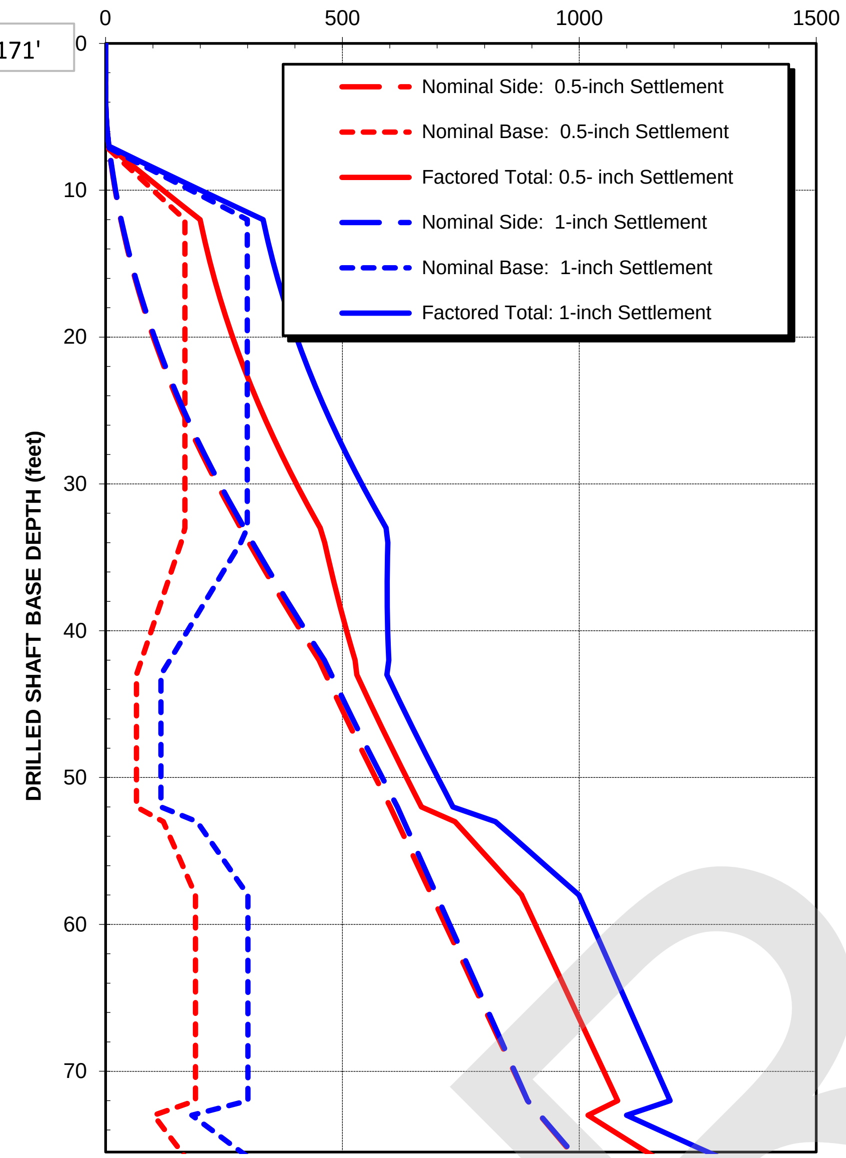
ASSUMED SUBSURFACE PROFILE

Based on Nearby Explorations:
B-4



SERVICE LIMIT

NOMINAL RESISTANCE (tons)

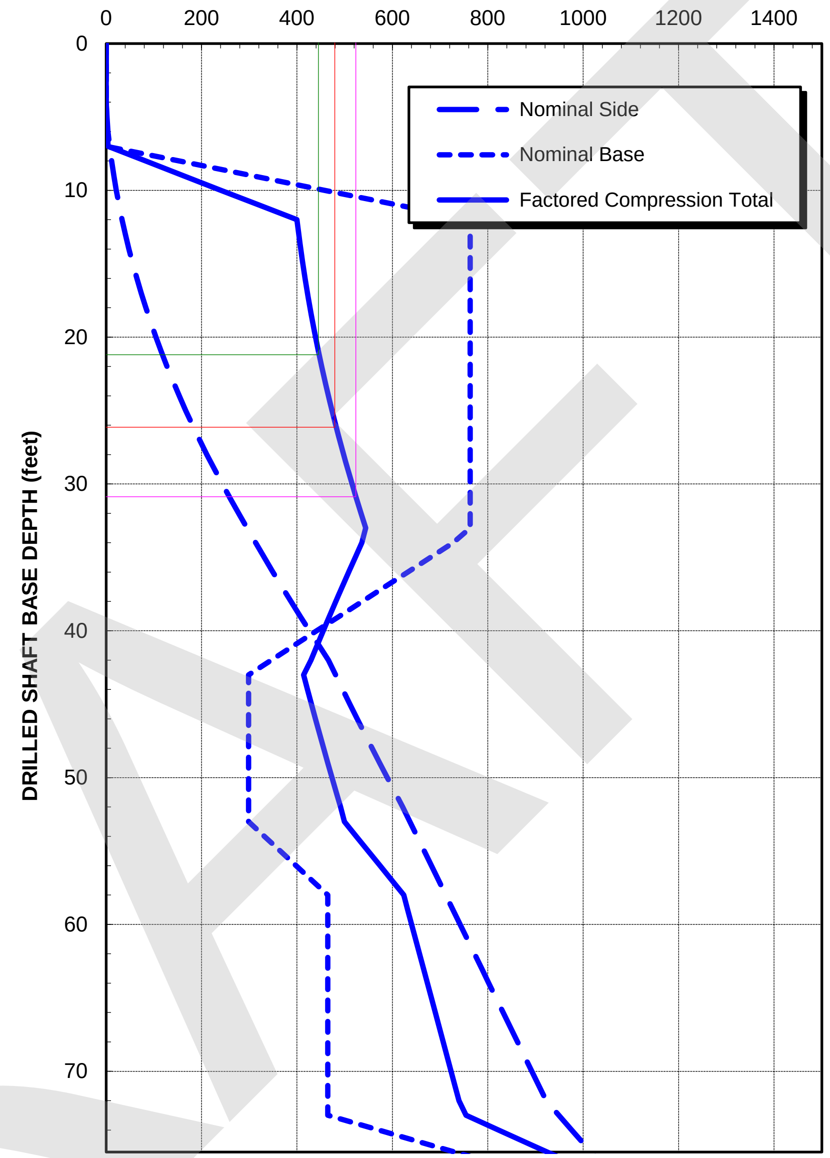


SERVICE LIMIT NOTES:

1. Recommended resistance factors per ODOT GDM are 1.0 for both side and base resistance.
2. Settlement is based on a single shaft. No group action is considered.

STRENGTH LIMIT

NOMINAL RESISTANCE (tons)

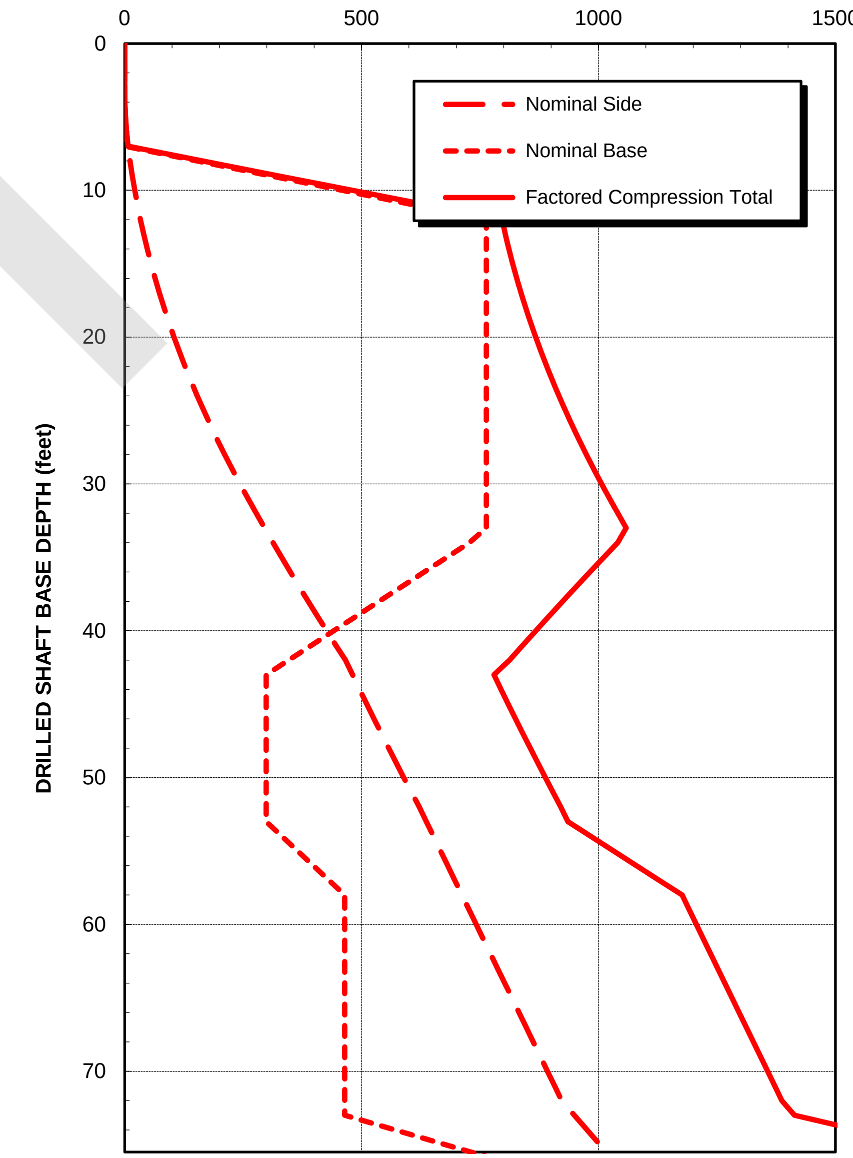


STRENGTH LIMIT NOTES:

1. Recommended compression resistance factors per ODOT GDM are 0.55 and 0.5 for side and base resistance, respectively.
2. Shaft uplift resistance can be estimated by using the nominal side resistance shown above and a recommended resistance factor of 0.45 (per ODOT GDM).

EXTREME EVENT LIMIT

NOMINAL RESISTANCE (tons)



EXTREME EVENT LIMIT NOTES:

1. Recommended resistance factors per ODOT GDM for both side and base resistance are 1.0 for compression and 0.8 for uplift.

GENERAL NOTES

1. The analyses were performed based on guidelines included in the AASHTO LRFD Bridge Design Specifications (AASHTO LRFD) and local experience. The analyses are based on a single shaft and do not consider group action of closely spaced shafts (closer than 4 diameters, center to center).
2. Factored total shaft resistance shown on plots is determined by adding its nominal side and base resistances multiplied by the appropriate resistance factors as noted above.
3. Estimated shaft resistance assumes that if casing is used, it will be removed after the shaft installation. If, however, the casing is left in place, grouting should be used to fill all potential voids around the casing and the estimated resistance given above should be re-evaluated.

I-5 Pedestrian Bridge
Barber St. to Wilsonville Town Center
Wilsonville, Oregon

ESTIMATED AXIAL SHAFT RESISTANCE
6.5-FOOT DIAMETER DRILLED SHAFT
BENTS 6, 7 & 8

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FIG. 10

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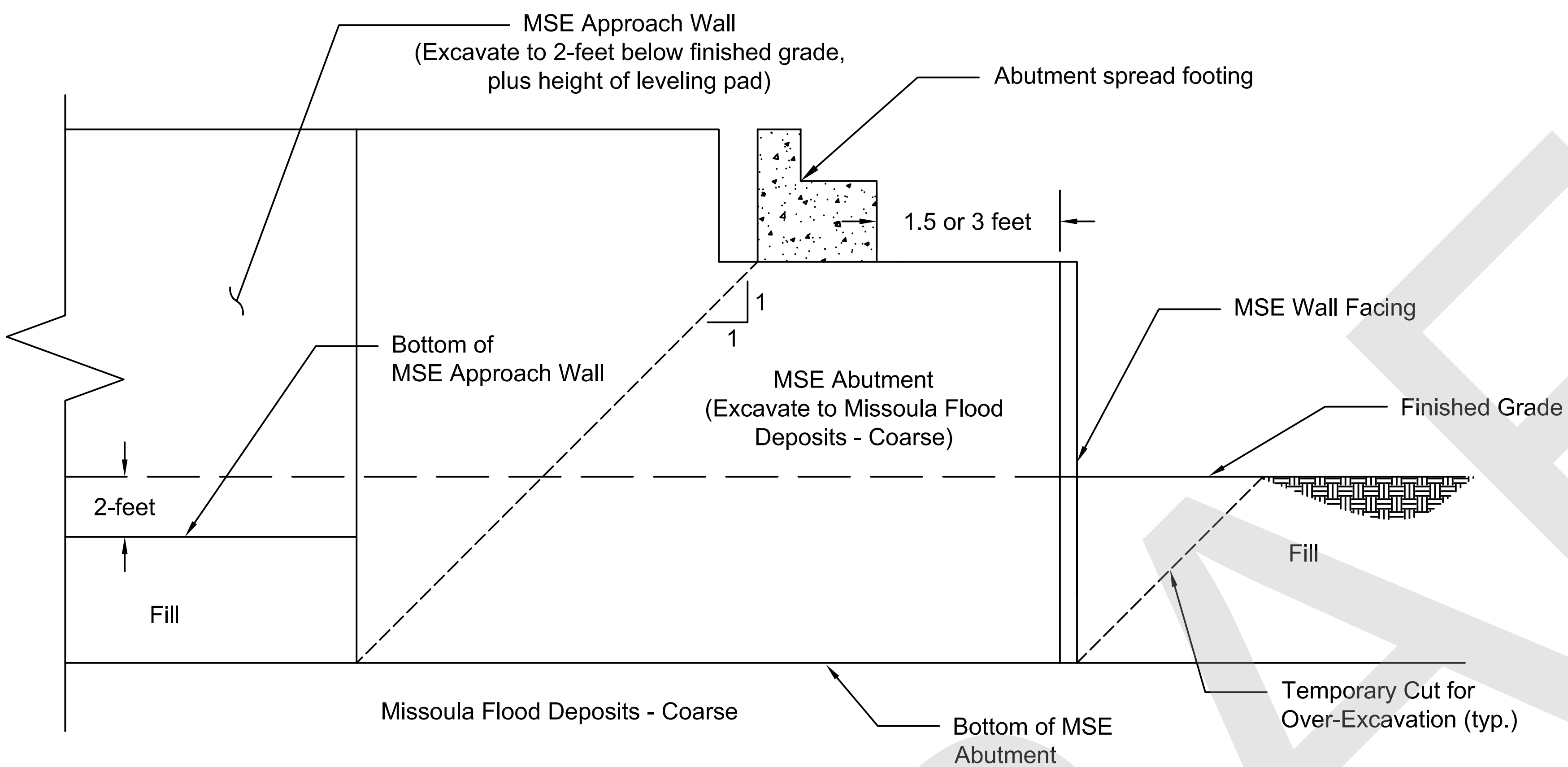
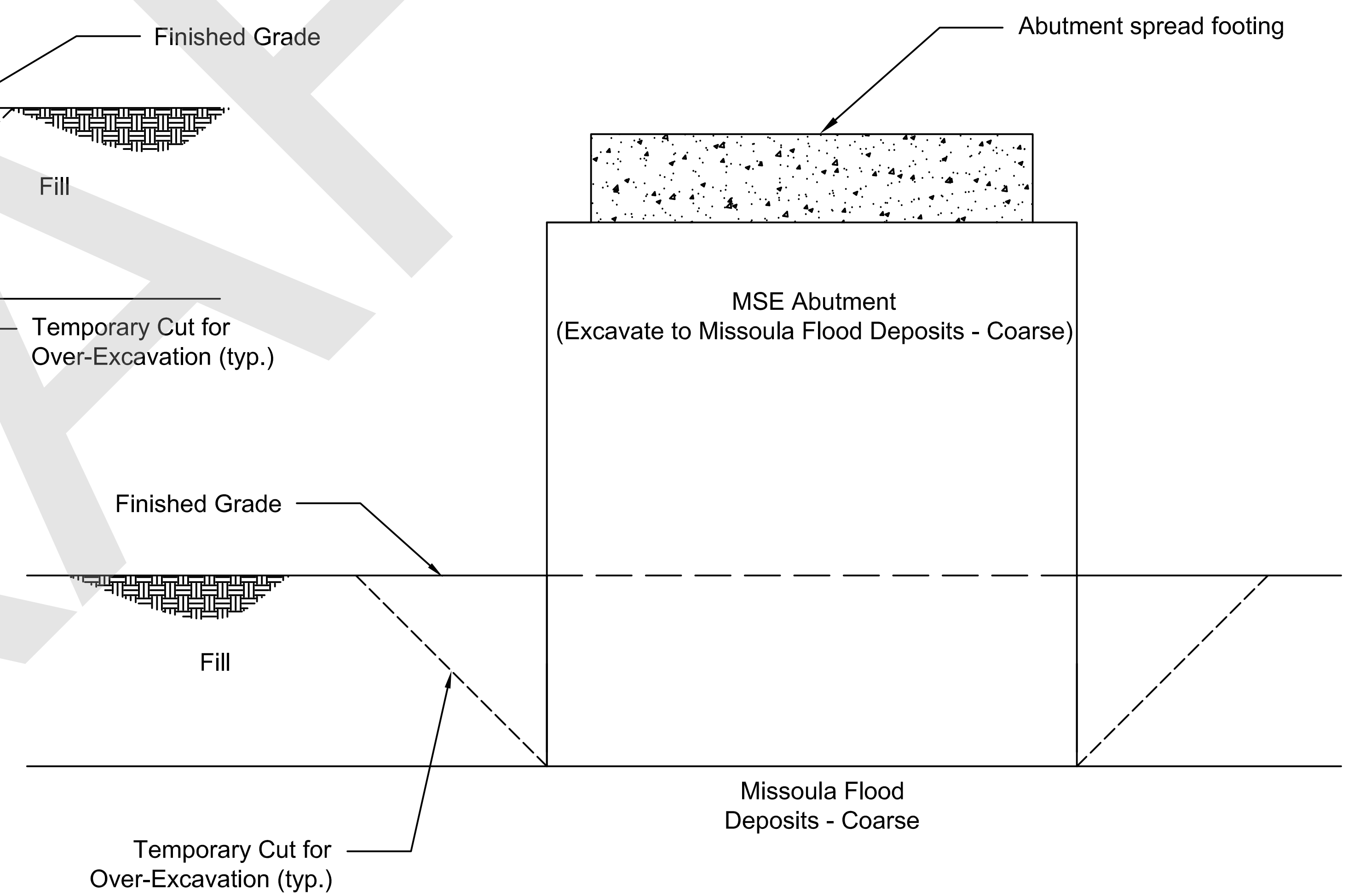
ELEVATION VIEW
NOT TO SCALE



CROSS-SECTIONAL VIEW
NOT TO SCALE



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Note:
1) Overexcavate to top of native Missoula Flood Deposits - Coarse, and begin MSE Abutment Construction.
2) Footing setback depends on type of MSE reinforcement. Per ODOT GDM, use a minimum of 18-inches (1.5-feet) with steel reinforcement, and 3-feet with geogrid reinforcement.

I-5 Pedestrian Bridge
Barber St. to Wilsonville Town Center
Wilsonville, Oregon

MSE ABUTMENT TYPICAL DETAILS

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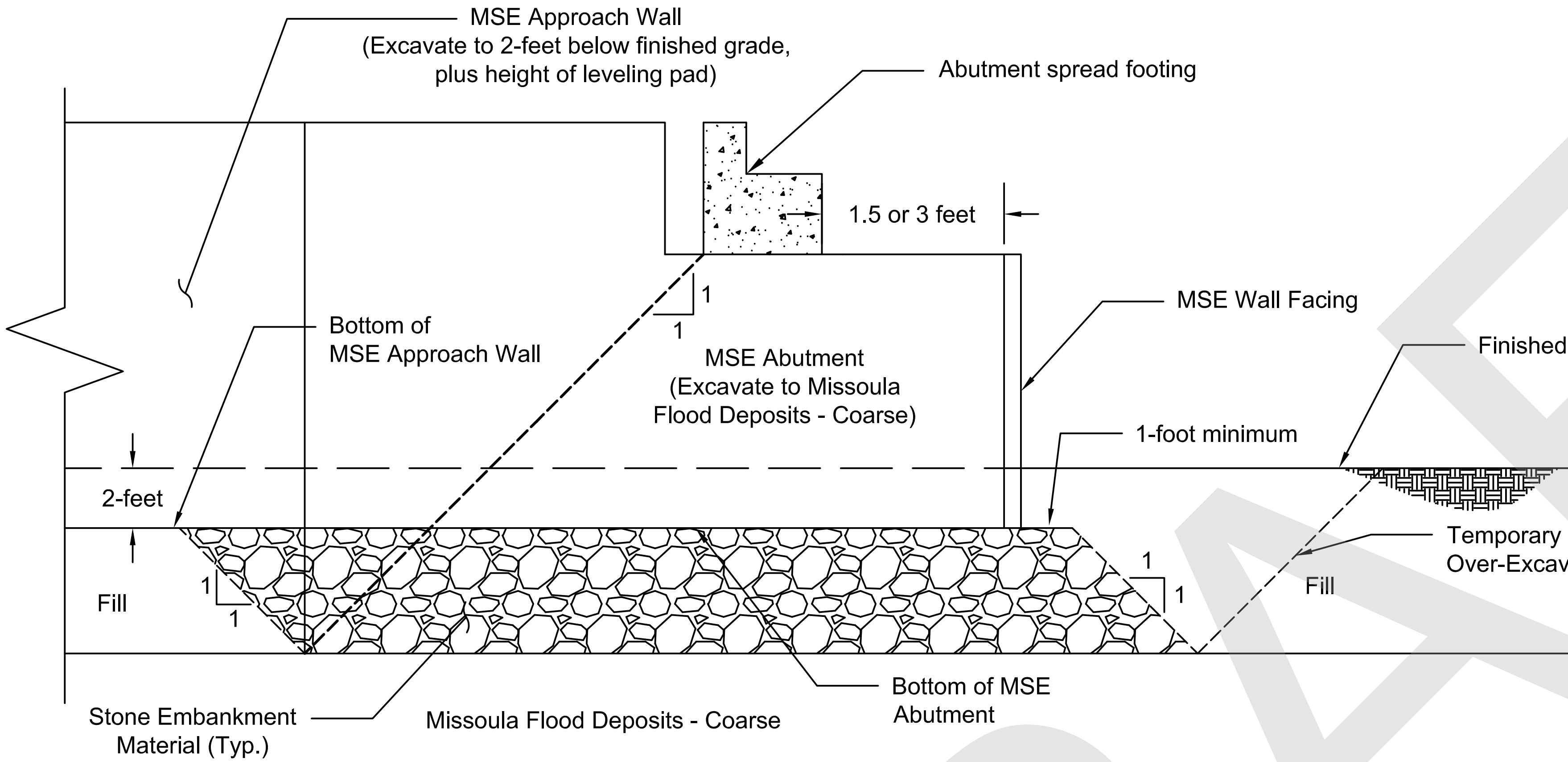
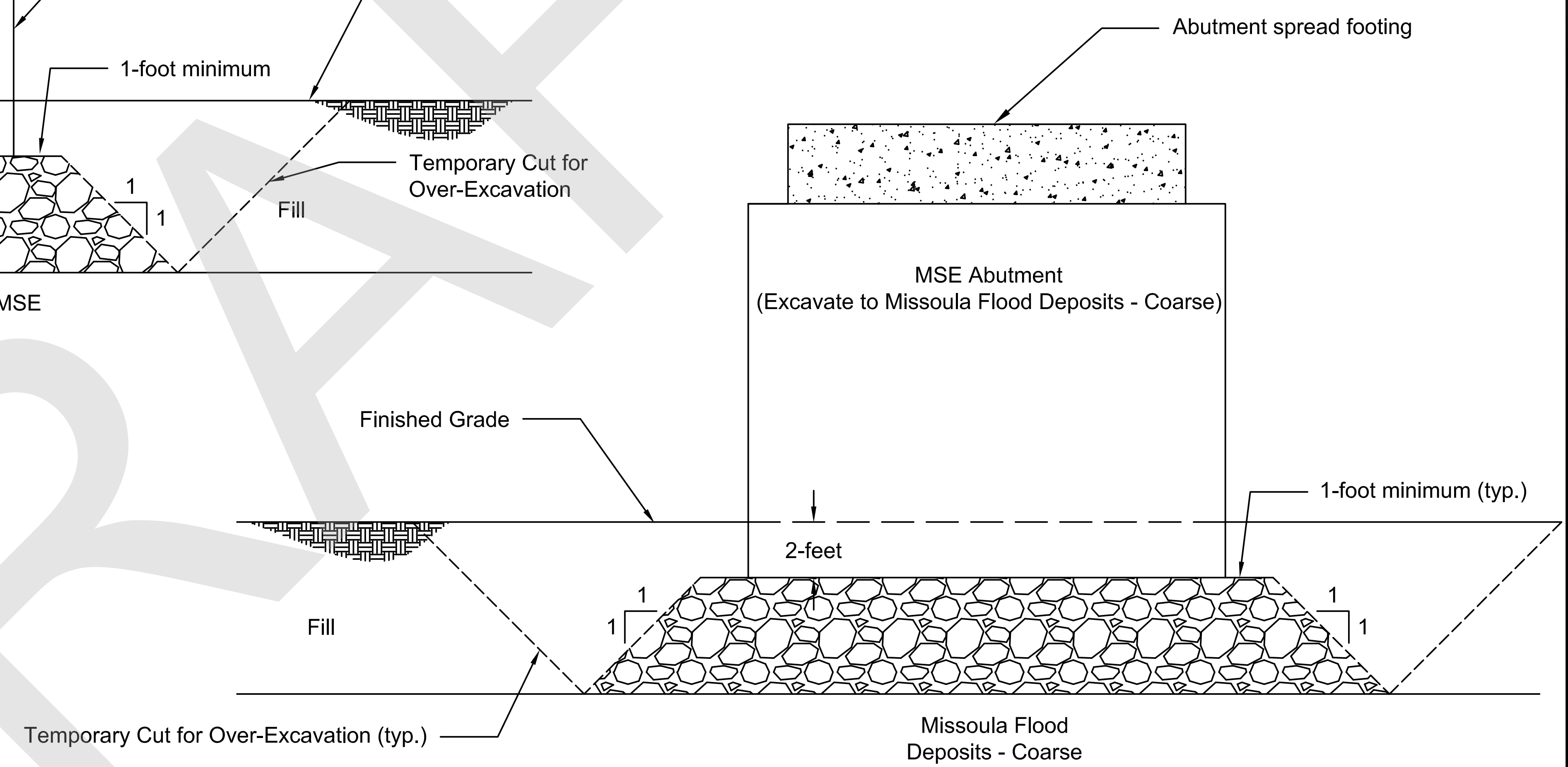
FIG. 11

Filename: I:\EF\PDX\103000s\103953 Wilsonville I-5\DRAFTING\MSE Abutment Figures.dwg Layout: MSE Abutment Excavation 1 Date: 11-30-2020 Login: DBP

ELEVATION VIEW
NOT TO SCALE



CROSS-SECTIONAL VIEW
NOT TO SCALE



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Note:

1) Overexcavate to top of native Missoula Flood Deposits - Coarse and backfill with stone embankment material to 2-feet below finish grade. Install a non-woven subgrade seperation geotextile, and begin MSE Abutment Construction above.
2) Footing setback depends on type of MSE reinforcement. Per ODOT GDM, use a minimum of 18-inches (1.5-feet) with steel reinforcement, and 3-feet with geogrid reinforcement.

I-5 Pedestrian Bridge
Barber St. to Wilsonville Town Center
Wilsonville, Oregon

**MSE ABUTMENT TYPICAL DETAILS:
MSE ABUTMENT FOUNDED ON STONE
EMBANKMENT MATERIAL**

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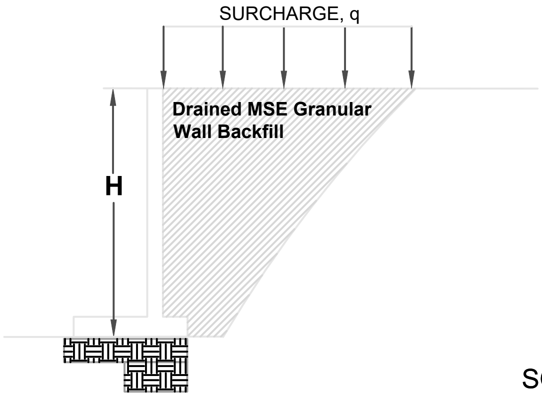
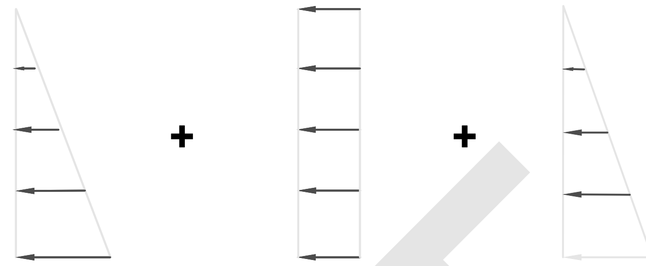
FIG. 12

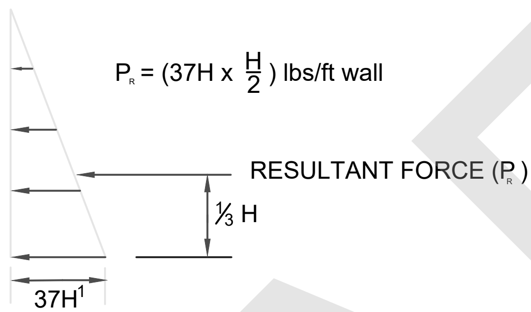
Filename: I:\EF\PDX\103000s\103953 Wilsonville I-5\DRAFTING\MSE Abutment Figures.dwg Layout: MSE Abutment Excavation 2 Date: 11-30-2020 Login: DBP



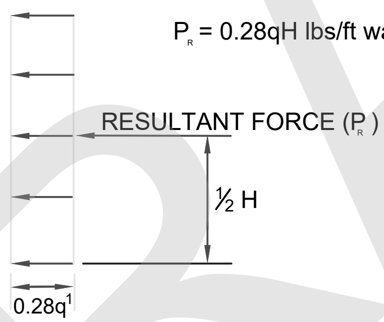
TOTAL LATERAL EQUIVALENT FLUID PRESSURES



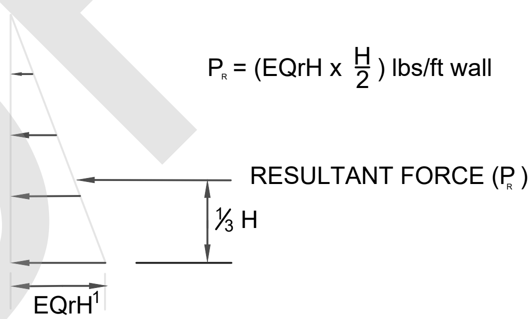
YIELDING WALL SOIL COMPONENT



YIELDING WALL SURCHARGE COMPONENT



YIELDING SEISMIC BACKFILL COMPONENT



NOTES

1. Units are pounds per square foot (psf).
2. Backfill unit weight of 130 pcf.
3. Backfill friction angle is 34 deg.
4. Retained wall backfill is assumed to be drained MSE granular wall backfill material.
5. Seismic pressures provided for peak ground accelerations associated with a 1,000-year earthquake ("Life Safety" criteria) and the CSZE ("Operational" critera). See Table 1 for values.

TABLE 1

EQ LEVEL	YIELDING EQr (pcf)
CSZE	6.5
1000	14.5

I-5 Pedestrian Bridge
Barber St. to Wilsonville Town Center
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RECOMMENDED LATERAL PRESSURES FOR MSE WALLS

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FIG. 13

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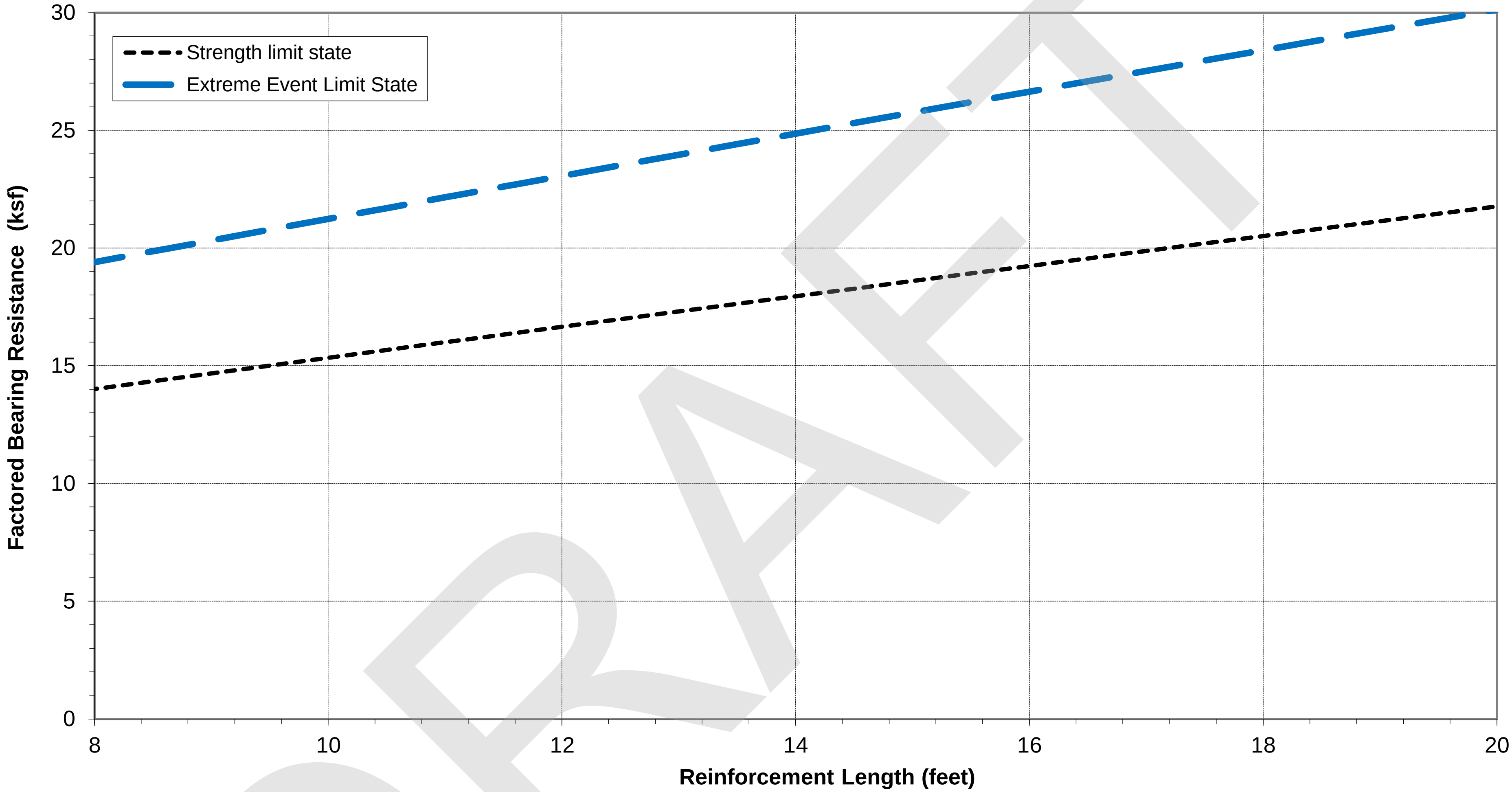

NOTES

1. We recommend using the following resistance factors for footing LRFD design; the plotted bearing capacities use the bearing capacity resistance factors.

Limit State	Sliding Shear	Passive Press.	Bearing Capacity
Service	N/A	N/A	1.0
Strength	0.8	0.5	0.65
Extreme Event	1.0	1.0	0.9

2. The factored bearing capacities are based on a soil friction angle of 33 degrees, a soil cohesion of 0 psf, and a total unit weight of 120 pcf. We assumed that the bottom of the footing was 2 feet below the ground surface.

3. **psf** - pounds per square foot; **pcf** - pounds per cubic foot; **ksf** - kips per square foot (1 kip = 1000 pounds)

I-5 Pedestrian Bridge
Barbur St. to Wilsonville Town Center
Wilsonville, Oregon

FACTORED BEARING RESISTANCE VS REINFORCEMENT LENGTH FOR WEST APPROACH MSE WALL

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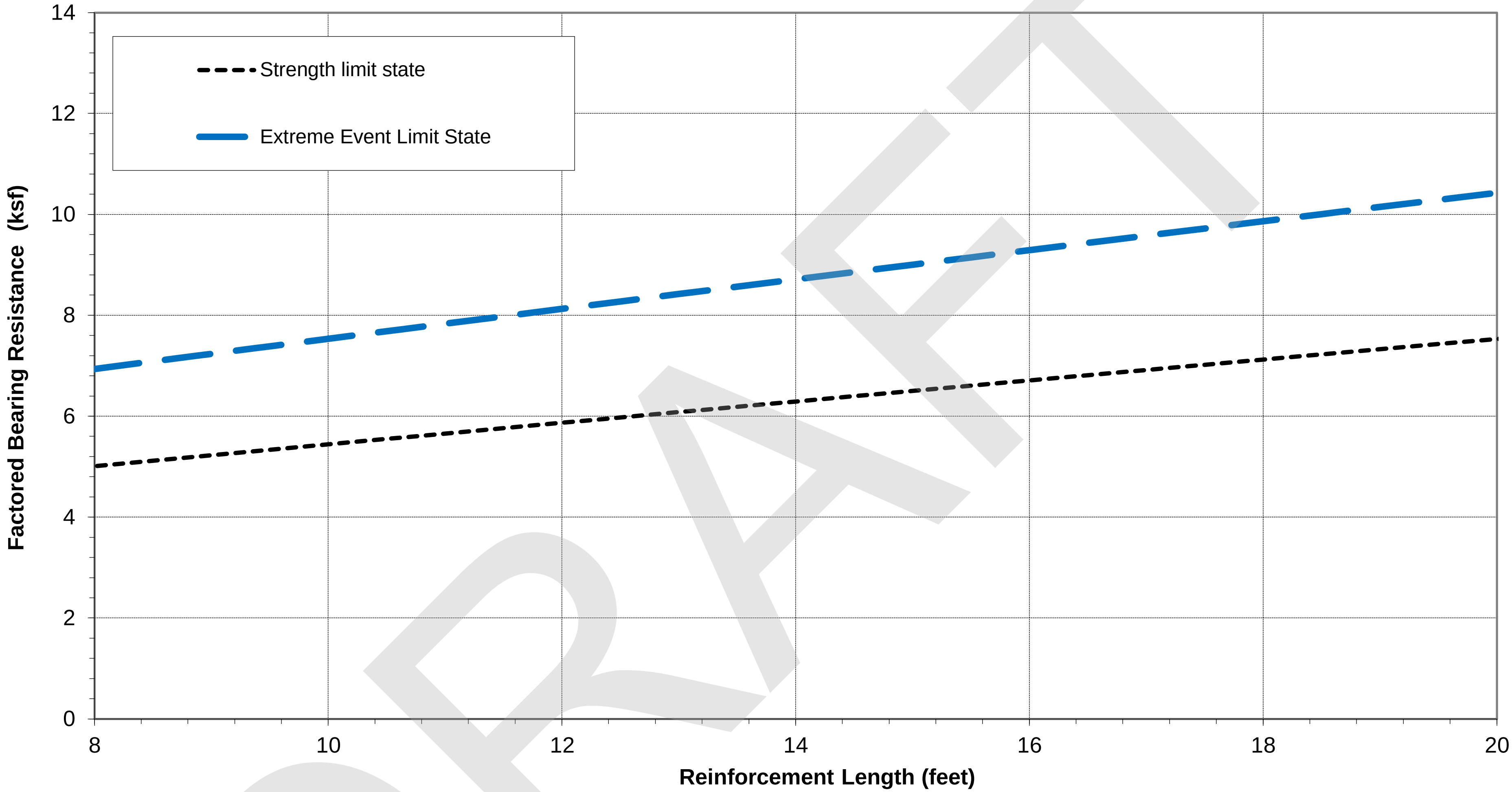
FIG. 14

FIG. 14



NOTES

1. We recommend using the following resistance factors for footing LRFD design; the plotted bearing capacities use the bearing capacity resistance factors.

Limit State	Sliding Shear	Passive Press.	Bearing Capacity
Service	N/A	N/A	1.0
Strength	0.8	0.5	0.65
Extreme Event	1.0	1.0	0.9

2. The factored bearing capacities are based on a soil friction angle of 26 degrees, a soil cohesion of 0 psf, and a total unit weight of 110 pcf. We assumed that the bottom of the footing was 2 feet below the ground surface.

3. **psf** - pounds per square foot; **pcf** - pounds per cubic foot; **ksf** - kips per square foot (1 kip = 1000 pounds)

I-5 Pedestrian Bridge
Barbur St. to Wilsonville Town Center
Wilsonville, Oregon

FACTORED BEARING RESISTANCE VS REINFORCEMENT LENGTH FOR EAST APPROACH MSE WALL

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FIG. 15

FIG. 15

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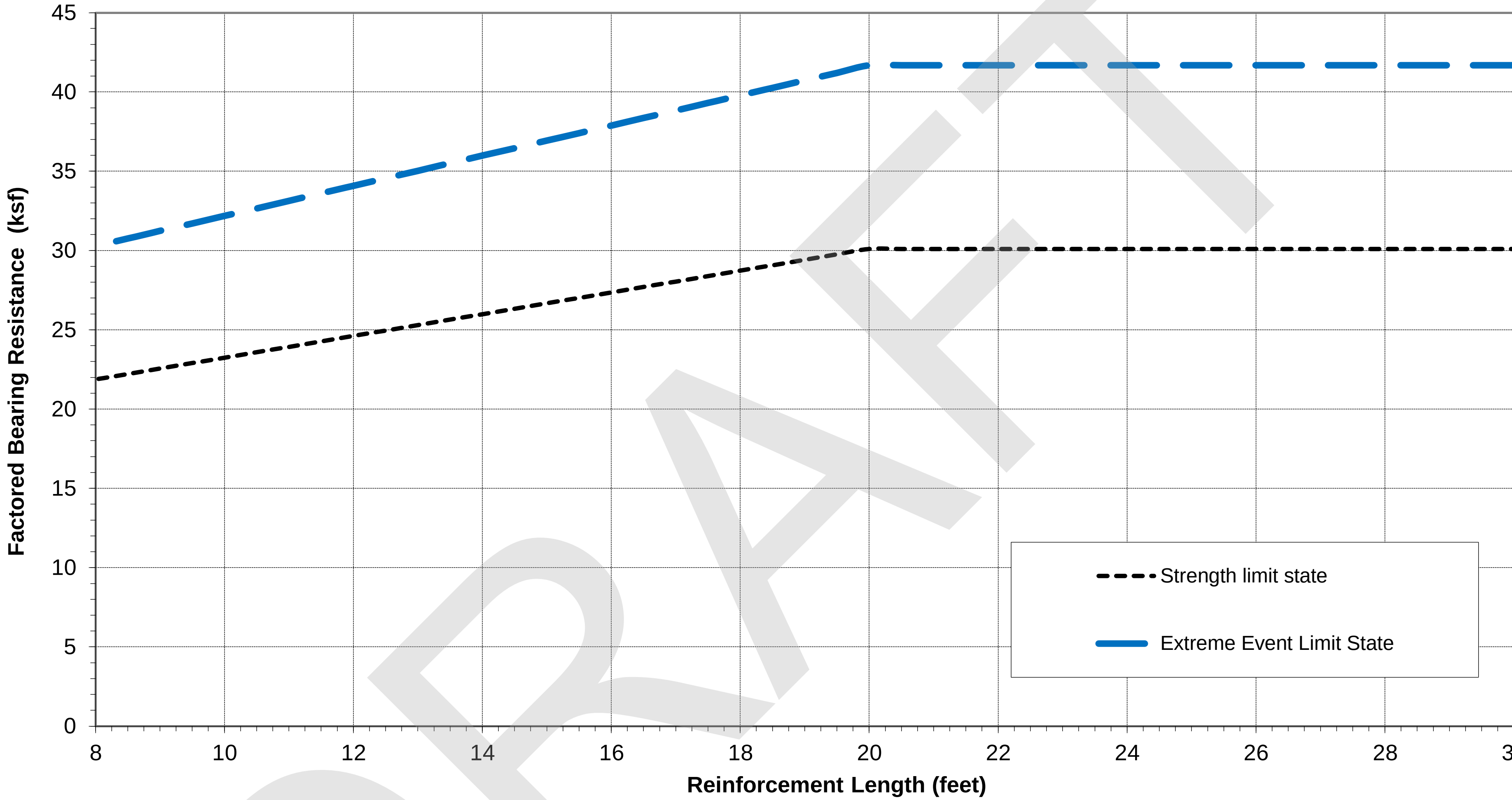

NOTES

1. We recommend using the following resistance factors for footing LRFD design; the plotted bearing capacities use the bearing capacity resistance factors.

Limit State	Sliding Shear	Passive Press.	Bearing Capacity
Service	N/A	N/A	1.0
Strength	0.8	0.5	0.65
Extreme Event	1.0	1.0	0.9

2. The factored bearing capacities are based on a soil friction angle of 36 degrees, a soil cohesion of 0 psf, and a total unit weight of 125 pcf. We assumed that the bottom of the footing was 2 feet below the ground surface, and bearing on either the Missoula Flood Deposits - Coarse unit or stone embankment material.
3. **psf** - pounds per square foot; **pcf** - pounds per cubic foot; **ksf** - kips per square foot (1 kip = 1000 pounds)

I-5 Pedestrian Bridge
Barber St. to Wilsonville Town Center
Wilsonville, Oregon

FACTORED BEARING RESISTANCE VS REINFORCEMENT LENGTH FOR MSE ABUTMENTS

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FIG. 16

FIG. 16

APPENDIX A: FIELD EXPLORATIONS

Appendix A

Field Explorations

CONTENTS

A.1 General A-1
A.2 Drilling A-1
A.2.1 Disturbed Sampling A-2
A.2.2 Relatively Undisturbed Sampling A-2
A.3 Material Descriptions A-3
A.4 Drill Logs A-3
A.5 Borehole Abandonment A-3

Tables

Table A-1: Summary of Geotechnical Borehole Information

Figures

Figure A1: Drill Log, B-1
Figure A2: Drill Log, B-2
Figure A3: Drill Log, B-3
Figure A4: Drill Log, B-4
Figure A5: Drill Log, B-5

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A.1 GENERAL

Shannon & Wilson, Inc., explored subsurface conditions at the project site with five geotechnical borings, designated B-1 through B-5. Completed boring locations were surveyed by Shannon & Wilson, Inc. using a handheld GPS system, and we understand the completed boring locations will be surveyed in at a later date. Borehole coordinates and elevations are presented on the Drill Logs and are reported in NAD 83 Oregon State Plane South (US Feet) and NAVD88, respectively. Approximate boring locations are shown on the Site and Exploration Plan, Figure 2. Shannon & Wilson geologists were present during the drilling to locate the borings, check for underground utilities, log the materials encountered, and collect soil samples for laboratory testing. Table A-1 provides a summary of borehole information, including boring designation, total depth, drill rig and hammer efficiencies, and drilling technique.

This appendix describes the techniques used to advance and sample the borings and presents logs of the materials encountered.

Table A-1: Summary of Geotechnical Borehole Information.

Borehole Designation	Total Depth (feet)	Drill Rig Type	Hammer Efficiency[1]	Drilling Techniques[2]
B-1	76.4	CME-75 Truck Rig	69.2%	MR
B-2	76.5	CME-75 Truck Rig	80.8%	MR, CA
B-3	76.5	CME-75 Truck Rig	69.2%	MR, CA
B-4	75.5	CME-75 Truck Rig	78.4%	MR, CA
B-5	61.5	CME-75 Truck Rig	69.2%	MR

Notes:
1 Energy Transfer Efficiency (measured hammer energy divided by the rated hammer energy)
2 MR – Open-hole Mud Rotary; CA – Casing Advancer

A.2 DRILLING

The geotechnical borings were performed with three different mobilizations between April 6, 2020 and August 31, 2020. The borings were drilled by Western States Soil Conservation Inc. of Hubbard, Oregon using three different truck-mounted CME-75 drill rigs. The five geotechnical borings were advanced to depths ranging from 61.5 to 76.5 feet bgs. The

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borings were advanced using open-hole mud rotary and casing advancer drilling techniques.

A.2.1 Disturbed Sampling

Disturbed samples were typically collected in the borings, 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 at the time of sampling, sealed in a labeled plastic jar or bag to retain moisture, and returned to our laboratory for additional examination and testing.

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. Three different truck-mounted CME-75 drill rigs were used, and based on information from Western States, the average energy transfer efficiency of the automatic hammer used on Rig #1 averaged 78.4 percent, Rig #4 averaged 69.2 percent, and Rig #5 averaged 80.8 percent. The efficiencies of the hammers used for this project are also presented in Table A-1. 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.

An SPT was considered to have met refusal where more than 50 blows were required to drive the sampler six inches. If refusal was encountered for the first 6-inch interval (for example, 50 for 1.5”), the count is reported as 50/1st 1.5”. If refusal was encountered in the second 6-inch interval (for example, 48, 50 for 1.5”), the count is reported as 50/1.5”. If refusal was encountered in the last 6-inch interval (for example, 39, 48, 50 for 1.5”), the count is reported as 98/7.5”. Sample recovery is identified as a percentage of material retained for the length the sampler was driven.

A.2.2 Relatively Undisturbed Sampling

Relatively undisturbed samples were collected in 3-inch O.D. thin-wall Shelby tubes which were hydraulically pushed into the undisturbed soil at the bottoms of boreholes. The soils exposed at the ends of the tubes were examined and described in the field. After examination, the ends of the tubes were sealed to preserve the natural moisture of the

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samples. The sealed tubes were stored in the upright position and care was taken to avoid shock and vibration during their transport and storage in our laboratory.

A.3 MATERIAL DESCRIPTIONS

In the field, samples were described and identified visually in accordance with the ODOT Soil and Rock Classification Manual (1987). The ASTM International (ASTM) D2488 Visual-Manual method was also used as a guide in determining the key diagnostic properties of soils. Consistency, color, relative moisture, degree of plasticity, peculiar odors, and other distinguishing characteristics of the samples were noted. Once returned to our laboratory, the samples were reexamined, various laboratory tests were conducted, and the field descriptions and identifications were modified where necessary. Please refer to the ODOT Soil and Rock Classification Manual (1987) for definitions of descriptive terminology used in the Drill Logs.

A.4 DRILL LOGS

Summary logs of the borings are presented in the Drill Logs, Figures A1 through A5. 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 drill logs, and the right-hand portions provides a graphic log, miscellaneous comments, and a graphic depicting hole installation and backfill details.

A.5 BOREHOLE ABANDONMENT

Borings were backfilled with bentonite cement grout or bentonite chips in accordance with Oregon Water Resource Department regulations. Boreholes in roadways or parking lots were finished with sections of ODOT approved asphalt cold patch consistent with the existing pavement section thickness or 6 inches, whichever was greater, and nominally compacted gravel extending to a minimum depth of 2 feet.

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DRILL LOG
OREGON DEPARTMENT OF TRANSPORTATION

		Hole No. **B-1**
Project **I-5 Pedestrian Bridge Barber St. to Wilsonville Town Center**	Purpose **Bridge Foundation**	E.A. No. **N/A**
Highway **Interstate 5**	County **Clackamas**	Key No. **N/A**
Hole Location Northing: **~ 94,092** Easting: **~ 322,878**		Start Card No. **N/A**
Equipment **CME 75 Truck Rig (Hammer Efficiency = 69.2%**	Driller **Western States**	Bridge No. **N/A**
Project Geologist **Seth C. Sonnier, RG**	Recorder **David Jacobson**	Ground Elev. **~ 166 ft.**
Start Date **August 20, 2020** End Date **August 20, 2020**	Total Depth **76.40 ft**	Tube Height **N/A**

Test Type
"A" - Auger Core
"GP" - GeoProbe®
"X" - Auger
"C" - Core, Barrel Type
"N" - Standard Penetration
"U" - Undisturbed Sample
"T" - Test Pit

Rock Abbreviations

Discontinuity	Shape	Surface Roughness
J - Joint	Pl - Planar	P - Polished
F - Fault	C - Curved	Sl - Slickensided
B - Bedding	U - Undulating	Sm - Smooth
Fo - Foliation	St - Stepped	R - Rough
S - Shear	Ir - Irregular	VR - Very Rough

Typical Drilling Abbreviations

Drilling Methods	Drilling Remarks
WL - Wire Line	LW - Lost Water
HS - Hollow Stem Auger	WR - Water Return
DF - Drill Fluid	WC - Water Color
SA - Solid Auger	DP - Down Pressure
CA - Casing Advancer	DR - Drill Rate
HA - Hand Auger	DA - Drill Action

Material Description — SOIL: Soil Name, USCS, Color, Plasticity, Moisture, Consistency/Relative Density, Texture, Cementation, Structure, Origin. ROCK: Rock Name, Color, Weathering, Hardness, Discontinuity Spacing, Joint Filling, Core Recovery, Formation Name.

Depth (ft)	Test Type, No.	Percent Recovery	Soil: Driving Resistance	Rock: Discontinuity Data / Or RQD%	Percent Natural Moisture	Material Description	Unit Description	Graphic Log	Drilling Methods, Size and Remarks	Water Level/ Date	Backfill/ Instrumentation
0							**0.00 - 0.50** Asphalt Concrete; (Fill)				
							0.50 - 1.00 Base Aggregate; (Fill)				
							1.00 - 4.50 SILT with trace sand; ML; Brown; Nonplastic; Moist; Dense; Fine sand; Micaceous; Slight iron oxidation and staining; Trace organics; Pockets of Silty CLAY (CL); (Fill)				
	N1	78	8-11-21		28	N- 1 (2.50-4.00) SILT with trace sand; ML; Brown; Nonplastic; Moist; Dense; Fine sand; Micaceous; Slight iron oxidation and staining; Trace organics; Pockets of Silty CLAY (CL); (Fill)					
5	N2	91	31-50/5"			N- 2 (5.00-5.90) Silty GRAVEL with some sand; GM; Brown to gray; Nonplastic to low plasticity fines; Moist; Very dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse sand; Micaceous; Slight iron oxidation and staining; (Fill)	**4.50 - 7.00** Silty GRAVEL with some sand, with possible cobbles and boulders; GM; Brown to gray; Nonplastic to low plasticity fines; Moist; Very dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse sand; Micaceous; Slight iron oxidation and staining; (Fill)				
	N3	100	16-27-19		24	N- 3 (7.50-9.00) Sandy silty GRAVEL; GM; Brown to gray; Low plasticity fines; Moist; Dense; Fine to coarse, subangular to subrounded gravel; Fine sand; Micaceous; Slight iron oxidation and staining; (Missoula Flood Deposits - Coarse)	**7.00 - 17.00** Sandy silty GRAVEL, with possible cobbles and boulders; GM; Brown to gray; Low plasticity fines; Moist; Dense to very dense; Fine to coarse, subangular to subrounded gravel; Fine sand; Micaceous; Slight iron oxidation and staining; (Missoula Flood Deposits - Coarse)		Lost DF circulation at 7.5 ft.		
10	N4	89	17-24-41			N- 4 (10.00-11.50) Sandy silty GRAVEL; GM; Brown to gray; Low plasticity fines; Moist; Very dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse sand; Micaceous; (Missoula Flood Deposits - Coarse)					
	N5	39	31-26-21			N- 5 (12.50-14.00) Sandy silty GRAVEL; GM; Brown to gray; Low plasticity fines; Moist; Dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse sand; Micaceous; Slight iron oxidation and staining; Basalt clasts; (Missoula Flood Deposits - Coarse)					
15											

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Depth (ft)	Test Type, No.	Percent Recovery	Soil: Driving Resistance / Rock: Discontinuity Data Or RQD%	Percent Natural Moisture	Material Description SOIL: Soil Name, USCS, Color, Plasticity, Moisture, Consistency/Relative Density, Texture, Cementation, Structure, Origin. ROCK: Rock Name, Color, Weathering, Hardness, Discontinuity Spacing, Joint Filling, Core Recovery, Formation Name.	Unit Description	Graphic Log	Drilling Methods, Size and Remarks	Water Level/ Date	Backfill/ Instrumentation
15	N6	78	20-17-18		N- 6 (15.00-16.50) Sandy silty GRAVEL; GM; Brown to gray; Low plasticity fines; Moist; Dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse sand; Micaceous; Slight iron oxidation and staining; Basalt clasts; (Missoula Flood Deposits - Coarse)					
						17.00 - 42.50 Sandy silty GRAVEL to Silty GRAVEL with some sand, with possible cobbles and boulders; GM; Brown to gray; Nonplastic to low plasticity fines; Moist; Dense to very dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse sand Micaceous; Slight iron oxidation and staining; Basalt clasts; (Missoula Flood Deposits - Coarse)				
	N7	39	10-16-16		N- 7 (17.50-19.00) Sandy silty GRAVEL; GM; Brown to gray; Nonplastic to low plasticity fines; Moist; Dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse sand; Micaceous; Slight iron oxidation and staining; Basalt clasts; (Missoula Flood Deposits - Coarse)					
20	N8	56	19-30-9		N- 8 (20.00-21.50) Silty GRAVEL with some sand; GM; Brown to gray to orange; Nonplastic to low plasticity fines; Moist; Dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse sand; Micaceous; Slight iron oxidation and staining; Basalt clasts; (Missoula Flood Deposits - Coarse)					
								Regained DF circulation at 23 ft.		
25	N9	72	18-14-15		N- 9 (25.00-26.50) Sandy silty GRAVEL; GM; Brown to gray to orange; Nonplastic to low plasticity fines; Moist; Medium dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse sand; Micaceous; Slight iron oxidation and staining; Basalt clasts; (Missoula Flood Deposits - Coarse)					
								Lost DF circulation at 29 ft.		
30	N10	78	17-16-15		N- 10 (30.00-31.50) Silty GRAVEL with some sand; GM; Brown to gray; Nonplastic fines; Moist to wet; Dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse sand; (Missoula Flood Deposits - Coarse)					
35	N11	78	16-34-18		N- 11 (35.00-36.50) Silty GRAVEL with some sand; GM; Brown to gray; Low plasticity fines; Wet; Very dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse sand; (Missoula Flood Deposits - Coarse)					
38										

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Depth (ft)	Test Type, No.	Percent Recovery	Soil: Driving Resistance / Rock: Discontinuity Data Or RQD%	Percent Natural Moisture	Material Description (SOIL: Soil Name, USCS, Color, Plasticity, Moisture, Consistency/Relative Density, Texture, Cementation, Structure, Origin. ROCK: Rock Name, Color, Weathering, Hardness, Discontinuity Spacing, Joint Filling, Core Recovery, Formation Name.)	Unit Description	Graphic Log	Drilling Methods, Size and Remarks	Water Level/Date	Backfill/Instrumentation
38										
40	N12	73	11-50/5"		N- 12 (40.00-40.90) Sandy silty GRAVEL; GM; Gray; Low plasticity fines; Moist to wet; Very dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse sand; (Missoula Flood Deposits - Coarse)					
						42.50 - 47.50 Gravelly silty SAND, with possible cobbles and boulders; SM; Gray to brown; Nonplastic to low plasticity fines; Moist to wet; Medium dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse sand; Heavy iron oxidation and staining; (Missoula Flood Deposits - Coarse)				
45	N13	89	7-14-8		N- 13 (45.00-46.50) Gravelly silty SAND; SM; Gray to brown; Nonplastic to low plasticity fines; Moist to wet; Medium dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse sand; Heavy iron oxidation and staining; (Missoula Flood Deposits - Coarse)					
						47.50 - 52.50 SILT with trace sand; ML; Gray; Low plasticity; Moist; Very stiff; Fine to medium sand; Micaceous; Pockets of Silty CLAY with some sand (CL); (Pliocene / Pleistocene Sediments)				
50	N14	100	5-9-12	31	N- 14 (50.00-51.50) SILT with trace sand; ML; Gray; Low plasticity; Moist; Very stiff; Fine to medium sand; Micaceous; Pockets of Silty CLAY with some sand (CL); (Pliocene / Pleistocene Sediments)			Atterberg Limits N14: LL=34, PL=24, PI=10.		
						52.50 - 57.50 Clayey SILT; MH; Gray; Medium to high plasticity; Moist; Stiff; Micaceous; (Pliocene / Pleistocene Sediments)		Regained DF circulation at 54 ft.		
55	N15	100	5-5-5	43	N- 15 (55.00-56.50) Clayey SILT; MH; Gray; Medium to high plasticity; Moist; Stiff; Micaceous; (Pliocene / Pleistocene Sediments)					
						57.50 - 63.00 Clayey GRAVEL with some sand; GC; Gray to brown; Low to medium plasticity fines; Moist; Fine subangular to subrounded gravel;				
60										

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Depth (ft)	Test Type, No.	Percent Recovery	Soil: Driving Resistance / Rock: Discontinuity Data Or RQD%	Percent Natural Moisture	Material Description (SOIL: Soil Name, USCS, Color, Plasticity, Moisture, Consistency/Relative Density, Texture, Cementation, Structure, Origin. ROCK: Rock Name, Color, Weathering, Hardness, Discontinuity Spacing, Joint Filling, Core Recovery, Formation Name.)	Unit Description	Graphic Log	Drilling Methods, Size and Remarks / Water Level/ Date	Backfill/ Instrumentation
60	U1	100			U- 1 (60.00-60.25) Clayey GRAVEL with some sand; GC; Gray to brown; Low to medium plasticity fines; Moist; Fine subangular to subrounded gravel; Fine to coarse sand; (Pliocene / Pleistocene Sediments)	**Fine to coarse sand; (Pliocene / Pleistocene Sediments)**		Lost circulation of DF at 61 ft.	
	N16	100	14-10-10	35	N- 16 (60.25-61.75) Clayey GRAVEL with some sand; GC; Gray to brown; Low to medium plasticity fines; Moist; Medium dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse sand; Iron oxidation and staining; (Pliocene / Pleistocene Sediments)				
65	N17	100	11-16-23		N- 17 (65.00-66.50) Silty SAND; SM; Brown to blue-gray; Nonplastic to low plasticity fines; Moist; Dense; Fine to medium sand; Stratified with interbeds of Sandy SILT (ML); Moderate Iron oxide staining; (Pliocene / Pleistocene Sediments)	**63.00 - 67.50 Silty SAND; SM; Brown to blue-gray; Nonplastic to low plasticity fines; Moist; Dense; Fine to medium sand; Stratified with interbeds of Sandy SILT (ML); Moderate Iron oxide staining; (Pliocene / Pleistocene Sediments)**			
70	N18	100	14-17-22	22	N- 18 (70.00-71.50) Silty SAND with trace to some gravel; SM; Gray; Nonplastic fines; Moist; Dense; Fine to coarse, subangular to rounded gravel; Fine to coarse sand; Weakly cemented; (Pliocene / Pleistocene Sediments)	**67.50 - 76.40 Silty SAND to Silty SAND with trace to some gravel; SM; Gray; Nonplastic fines; Moist; Dense to very dense; Fine to coarse, subangular to rounded gravel; Fine to coarse sand; Weakly cemented; (Pliocene / Pleistocene Sediments)**			
75	N19	100	9-29-50/5"		N- 19 (75.00-76.40) Silty SAND; SM; Gray; Nonplastic fines; Moist; Very dense; Fine subangular to subrounded gravel; Fine to coarse sand; Weakly cemented; (Pliocene / Pleistocene Sediments)			Total DF loss of 850 gallons in boring B-1.	
						76.40 End of hole			
80									
83									

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DRILL LOG
OREGON DEPARTMENT OF TRANSPORTATION

		Hole No. **B-2**
Project **I-5 Pedestrian Bridge Barber St. to Wilsonville Town Center**	Purpose **Bridge Foundation**	E.A. No. **N/A**
Highway **Interstate 5**	County **Clackamas**	Key No. **N/A**
Hole Location Northing: **~ 93,965** Easting: **~ 322,874**		Start Card No. **N/A**
Equipment **CME 75 Truck Rig (Hammer Efficiency = 80.8%)**	Driller **Western States**	Bridge No. **N/A**
Project Geologist **Seth C. Sonnier, RG**	Recorder **Kevin Cowell**	Ground Elev. **~ 165 ft.**
Start Date **July 1, 2020** End Date **July 1, 2020**	Total Depth **76.50 ft**	Tube Height **N/A**

Test Type	Rock Abbreviations			Typical Drilling Abbreviations	
	Discontinuity	Shape	Surface Roughness	Drilling Methods	Drilling Remarks
"A" - Auger Core; "GP" - GeoProbe®	J - Joint	Pl - Planar	P - Polished	WL - Wire Line	LW - Lost Water
"X" - Auger	F - Fault	C - Curved	Sl - Slickensided	HS - Hollow Stem Auger	WR - Water Return
"C" - Core, Barrel Type	B - Bedding	U - Undulating	Sm - Smooth	DF - Drill Fluid	WC - Water Color
"N" - Standard Penetration	Fo - Foliation	St - Stepped	R - Rough	SA - Solid Auger	DP - Down Pressure
"U" - Undisturbed Sample	S - Shear	Ir - Irregular	VR - Very Rough	CA - Casing Advancer	DR - Drill Rate
"T" - Test Pit				HA - Hand Auger	DA - Drill Action

Material Description — SOIL: Soil Name, USCS, Color, Plasticity, Moisture, Consistency/Relative Density, Texture, Cementation, Structure, Origin. ROCK: Rock Name, Color, Weathering, Hardness, Discontinuity Spacing, Joint Filling, Core Recovery, Formation Name.

Depth (ft)	Test Type, No.	Percent Recovery	Soil: Driving Resistance / Rock: Discontinuity Data Or RQD%	Percent Natural Moisture	Material Description	Unit Description	Drilling Methods, Size and Remarks	Water Level/ Date
0						**0.00 - 0.50 Asphalt Concrete; (Fill)**		
						0.50 - 1.00 Base Aggregate; (Fill)		
						1.00 - 4.50 Sandy silty GRAVEL; GM; Brown to gray; Low plasticity fines; Moist to wet; Very dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse, subangular to subrounded sand; Micaceous; (Fill)		
	N1	80	50/1st 5"		N- 1 (2.50-2.90) Sandy silty GRAVEL; GM; Brown to gray; Low plasticity fines; Moist to wet; Very dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse, subangular to subrounded sand; Micaceous; (Fill)			
5	N2	67	13-20-12	17	N- 2 (5.00-6.50) Sandy silty GRAVEL; GM; Brown to gray; Low plasticity fines; Moist to wet; Dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse, subangular to subrounded sand; Micaceous; (Missoula Flood Deposits - Coarse)	**4.50 - 19.50 Sandy silty GRAVEL, with cobbles and possible boulders; GM; Brown to gray; Nonplastic to low plasticity fines; Moist to wet; Medium dense to very dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse, subangular to subrounded sand; Micaceous; (Missoula Flood Deposits - Coarse)**	Composite N2, N3, N4: 48% gravel, 34% sand, 18% fines.	
	N3	100	11-16-50/4"		N- 3 (7.50-8.80) Sandy silty GRAVEL, with cobbles; GM; Brown to gray; Low plasticity fines; Moist to wet; Very dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse, subangular to subrounded sand; Micaceous; (Missoula Flood Deposits - Coarse)			
10	N4	100	13-35-19		N- 4 (10.00-11.50) Sandy silty GRAVEL, with cobbles; GM; Brown to gray; Nonplastic to low plasticity fines; Moist to wet; Very dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse, subangular to subrounded sand; Micaceous; (Missoula Flood Deposits - Coarse)		Cobbles inferred from drill action from 5 to 15 ft.	
	N5	100	21-35-42		N- 5 (12.50-14.00) Sandy silty GRAVEL, with cobbles; GM; Brown to gray; Nonplastic fines; Moist to wet; Very dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse, subangular to subrounded sand; Micaceous; (Missoula Flood Deposits - Coarse)			
15								

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Project Name **I-5 Pedestrian Bridge** Hole No. **B-2**

Depth (ft)	Test Type, No.	Percent Recovery	Soil: Driving Resistance / Rock: Discontinuity Data Or RQD%	Percent Natural Moisture	Material Description SOIL: Soil Name, USCS, Color, Plasticity, Moisture, Consistency/Relative Density, Texture, Cementation, Structure, Origin. ROCK: Rock Name, Color, Weathering, Hardness, Discontinuity Spacing, Joint Filling, Core Recovery, Formation Name.	Unit Description	Graphic Log	Drilling Methods, Size and Remarks	Water Level/ Date	Backfill/ Instrumentation
15	N6	54	15-21-50/4"		N- 6 (15.00-16.30) Sandy silty GRAVEL, with cobbles; GM; Brown to gray; Nonplastic fines; Moist to wet; Very dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse, subangular to subrounded sand; Micaceous; (Missoula Flood Deposits - Coarse)			5 ft. of steel casing abandoned in borehole at approx. 15 ft. Cobbles inferred from drill action from 15 to 20 ft.		
	N7	40	10-11-9		N- 7 (17.50-19.00) Sandy silty GRAVEL; GM; Brown to gray; Nonplastic fines; Moist to wet; Medium dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse, subangular to subrounded sand; Micaceous; (Missoula Flood Deposits - Coarse)					
20	N8	47	15-9-6		N- 8 (20.00-21.50) Silty GRAVEL with some sand; GM; Brown to gray; Nonplastic fines; Moist to wet; Medium dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse, subangular to subrounded sand; Micaceous; (Missoula Flood Deposits - Coarse)	**19.50 - 23.00 Silty GRAVEL with some sand, with cobbles and possible boulders; GM; Brown to gray; Nonplastic fines; Moist to wet; Medium dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse, subangular to subrounded sand; Micaceous; (Missoula Flood Deposits - Coarse)**				
25	N9	73	9-10-10		N- 9 (25.00-26.50) Sandy silty GRAVEL; GM; Brown to gray; Nonlplastic fines; Moist to wet; Medium dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse, subangular to subrounded sand; Micaceous; (Missoula Flood Deposits - Coarse)	**23.00 - 27.50 Sandy silty GRAVEL, with cobbles and possible boulders; GM; Brown to gray; Nonlplastic fines; Moist to wet; Medium dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse, subangular to subrounded sand; Micaceous; (Missoula Flood Deposits - Coarse)**		Lost DF circulation from 25 ft. to bottom of borehole. Lost approx. 1000 gallons DF total.		
30	N10	20	6-7-7		N- 10 (30.00-31.50) GRAVEL with some sand and silt; GP-GM; Brown to gray; Nonplastic fines; Moist to wet; Medium dense; Fine to coarse, subangular to subrounded gravel; Fine sand; Micaceous; (Missoula Flood Deposits - Coarse)	**27.50 - 32.50 GRAVEL with some sand and silt, with cobbles and possible boulders; GP-GM; Brown to gray; Nonplastic fines; Moist to wet; Medium dense; Fine to coarse, subangular to subrounded gravel; Fine sand; Micaceous; (Missoula Flood Deposits - Coarse)**				
35	N11	100	18-12-15	19	N- 11 (35.00-36.50) Sandy silty GRAVEL; GM; Brown to gray; Nonplastic to low plasticity fines; Moist to wet; Dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse, subangular to subrounded sand; Micaceous; (Missoula Flood Deposits - Coarse)	**32.50 - 57.50 Sandy silty GRAVEL, with cobbles and possible boulders; GM; Brown to gray; Nonplastic to low plasticity fines; Moist to wet; Dense to very dense; Fine to coarse, subangular to subrounded**		Composite N11, N12, N13: 43% gravel, 41% sand, 16% fines.		
38										

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Depth (ft)	Test Type, No.	Percent Recovery	Soil: Driving Resistance / Rock: Discontinuity Data Or RQD%	Percent Natural Moisture	Material Description (SOIL: Soil Name, USCS, Color, Plasticity, Moisture, Consistency/Relative Density, Texture, Cementation, Structure, Origin. ROCK: Rock Name, Color, Weathering, Hardness, Discontinuity Spacing, Joint Filling, Core Recovery, Formation Name.)	Unit Description	Graphic Log	Drilling Methods, Size and Remarks	Water Level/ Date	Backfill/ Instrumentation
38						**gravel; Fine to coarse, subangular to subrounded sand; Micaceous; (Missoula Flood Deposits - Coarse)**				
40	N12	100	17-17-16		N- 12 (40.00-41.50) Sandy silty GRAVEL; GM; Brown to gray; Nonplastic fines; Moist to wet; Very dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse, subangular to subrounded sand; Micaceous; (Missoula Flood Deposits - Coarse)					
45	N13	100	18-14-11		N- 13 (45.00-46.50) Sandy silty GRAVEL; GM; Brown to gray; Nonplastic fines; Moist to wet; Dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse, subangular to subrounded sand; Micaceous; (Missoula Flood Deposits - Coarse)					
50	N14	87	12-11-15		N- 14 (50.00-51.50) Sandy silty GRAVEL; GM; Brown to gray; Nonplastic to low plasticity fines; Moist to wet; Dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse, subangular to subrounded sand; Micaceous; (Missoula Flood Deposits - Coarse)					
55	N15	60	11-30-48		N- 15 (55.00-56.50) Sandy silty GRAVEL; GM; Brown to gray; Low plasticity fines; Moist to wet; Very dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse, subangular to subrounded sand; Micaceous; (Missoula Flood Deposits - Coarse)					
60						**57.50 - 62.50 SILT with some sand; ML; Gray; Low plasticity; Moist to wet; Very stiff; Fine to medium sand; Micaceous; (Pliocene / Pleistocene**				

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Depth (ft)	Test Type, No.	Percent Recovery	Soil: Driving Resistance / Rock: Discontinuity Data Or RQD%	Percent Natural Moisture	Material Description (SOIL: Soil Name, USCS, Color, Plasticity, Moisture, Consistency/Relative Density, Texture, Cementation, Structure, Origin. ROCK: Rock Name, Color, Weathering, Hardness, Discontinuity Spacing, Joint Filling, Core Recovery, Formation Name.)	Unit Description	Graphic Log	Drilling Methods, Size and Remarks	Water Level/ Date	Backfill/ Instrumentation
60	N16	100	5-7-11	33	N- 16 (60.00-61.50) SILT with some sand; ML; Gray; Low plasticity; Moist to wet; Very stiff; Fine to medium sand; Micaceous; (Pliocene / Pleistocene Sediments)	**Sediments)**		Atterberg Limits N16: LL=40, PL=29, PI=11. N16: 78% fines		
						62.50 - 67.50 Silty SAND; SM; Brown to gray with blue pockets; Medium plasticity; Moist to wet; Medium dense; Fine to medium, subangular to subrounded sand; Interbedded with sandy SILT; (Pliocene / Pleistocene Sediments)				
65	N17	100	6-6-6	35	N- 17 (65.00-66.50) Silty SAND; SM; Brown to gray with blue pockets; Medium plasticity; Moist to wet; Medium dense; Fine to medium, subangular to subrounded sand; Interbedded with sandy SILT; (Pliocene / Pleistocene Sediments)					
						67.50 - 76.50 Silty SAND with trace to some gravel, with cobbles; SM; Dark gray; Nonplastic; Moist to wet; Very dense; Fine to coarse, rounded to subrounded gravel; Fine to medium, subangular to subrounded sand; (Pliocene / Pleistocene Sediments)				
70	N18	100	12-21-50/4"		N- 18 (70.00-71.30) Silty SAND with some gravel, with cobbles; SM; Dark gray; Nonplastic to low plasticity fines; Moist to wet; Very dense; Medium to coarse, subangular gravel; Fine to medium, subangular to rounded sand; (Pliocene / Pleistocene Sediments)					
75	N19	87	32-33-42		N- 19 (75.00-76.50) Silty SAND with trace to some gravel, with cobbles; SM; Dark gray; Nonplastic; Moist to wet; Very dense; Fine to medium, rounded to subrounded gravel; Fine to medium, subangular to subrounded sand; (Pliocene / Pleistocene Sediments)					
						76.50 End of hole				
80										
83										

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DRILL LOG
OREGON DEPARTMENT OF TRANSPORTATION

		Hole No. **B-3**
Project **I-5 Pedestrian Bridge Barber St. to Wilsonville Town Center**	Purpose **Bridge Foundation**	E.A. No. **N/A**
Highway **Interstate 5**	County **Clackamas**	Key No. **N/A**
Hole Location Northing: **~ 93,909** Easting: **~ 323,104**		Start Card No. **N/A**
Equipment **CME 75 Truck Rig (Hammer Efficiency = 69.2%**	Driller **Western States**	Bridge No. **N/A**
Project Geologist **Seth C. Sonnier, RG**	Recorder **David Jacobson**	Ground Elev. **~ 174 ft.**
Start Date **August 19, 2020** End Date **August 31, 2020**	Total Depth **76.50 ft**	Tube Height **N/A**

Test Type
"A" - Auger Core
"GP" - GeoProbe®
"X" - Auger
"C" - Core, Barrel Type
"N" - Standard Penetration
"U" - Undisturbed Sample
"T" - Test Pit

Rock Abbreviations

Discontinuity	Shape	Surface Roughness
J - Joint	Pl - Planar	P - Polished
F - Fault	C - Curved	Sl - Slickensided
B - Bedding	U - Undulating	Sm - Smooth
Fo - Foliation	St - Stepped	R - Rough
S - Shear	Ir - Irregular	VR - Very Rough

Typical Drilling Abbreviations

Drilling Methods	Drilling Remarks
WL - Wire Line	LW - Lost Water
HS - Hollow Stem Auger	WR - Water Return
DF - Drill Fluid	WC - Water Color
SA - Solid Auger	DP - Down Pressure
CA - Casing Advancer	DR - Drill Rate
HA - Hand Auger	DA - Drill Action

Material Description — SOIL: Soil Name, USCS, Color, Plasticity, Moisture, Consistency/Relative Density, Texture, Cementation, Structure, Origin. ROCK: Rock Name, Color, Weathering, Hardness, Discontinuity Spacing, Joint Filling, Core Recovery, Formation Name.

Depth (ft)	Test Type, No.	Percent Recovery	Soil: Driving Resistance / Rock: Discontinuity Data Or RQD%	Percent Natural Moisture	Material Description	Unit Description	Graphic Log	Drilling Methods, Size and Remarks	Water Level/ Date	Backfill/ Instrumentation
0						**0.00 - 0.50 Topsoil**				
						0.50 - 7.00 Silty CLAY with trace sand; CL; Brown; Low to medium plasticity; Moist; Very stiff; Fine sand; Micaceous; Trace organics; (Fill)				
	N1	22	11-12-8		N- 1 (2.50-4.00) Silty CLAY with trace sand; CL; Brown; Low to medium plasticity; Moist; Very stiff; Fine sand; Micaceous; Trace organics; (Fill)					
5	N2	22	4-4-5		N- 2 (5.00-6.50) Silty CLAY with trace sand; CL; Brown; Low to medium plasticity; Moist; Stiff; Fine sand; Micaceous; Trace organics; (Fill)					
	N3a	100	4-36-50/4"		N- 3a (7.50-8.20) SILT with trace sand; ML; Brown; Low to medium plasticity; Moist; Hard; Fine sand; Micaceous; Trace organics; (Fill)	**7.00 - 8.20 SILT with trace sand; ML; Brown; Low to medium plasticity; Moist; Hard; Fine sand; Micaceous; Trace organics; (Fill)**				
	N3b				N- 3b (8.17-8.80) Silty GRAVEL with some sand; GM; Brown to gray; Low plasticity fines; Moist; Very dense; Fine to coarse, subangular to subrounded gravel; Fine to medium sand; Micaceous; (Fill)	**8.20 - 9.50 Silty GRAVEL with some sand; GM; Brown to gray; Low plasticity fines; Moist; Very dense; Fine to coarse, subangular to subrounded gravel; Fine to medium sand Micaceous; (Fill)**				
10	N4	93	26-28-50/3"		N- 4 (10.00-11.30) Sandy silty GRAVEL; GM; Brown to gray; Nonplastic fines; Moist; Very dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse sand; Micaceous; (Fill)	**9.50 - 12.00 Sandy silty GRAVEL; GM; Brown to gray; Nonplastic fines; Moist; Very dense; Fine to coarse, subangular to subrounded gravel;**		Lost DF circulation at 13 ft. Regained DF circulation at 14 ft.		
15										

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Depth (ft)	Test Type, No.	Percent Recovery	Soil: Driving Resistance	Rock: Discontinuity Data Or RQD%	Percent Natural Moisture	Material Description SOIL: Soil Name, USCS, Color, Plasticity, Moisture, Consistency/Relative Density, Texture, Cementation, Structure, Origin. ROCK: Rock Name, Color, Weathering, Hardness, Discontinuity Spacing, Joint Filling, Core Recovery, Formation Name.	Unit Description	Graphic Log	Drilling Methods, Size and Remarks	Water Level/ Date	Backfill/ Instrumentation
15	N5	17	37-20-9			N- 5 (15.00-16.50) Silty GRAVEL with some sand; GM; Brown to gray; Low plasticity fines; Moist; Medium dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse sand; Micaceous; (Missoula Flood Deposits - Coarse)	**Fine to coarse sand; Micaceous; (Fill)** **12.00 - 52.50** **Silty GRAVEL with some sand to Sandy silty GRAVEL, with possible cobbles and boulders; GM; Brown to gray; Nonplastic to low plasticity fines; Moist; Medium dense to dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse sand; Micaceous; Trace decomposed gravel fragments; (Missoula Flood Deposits - Coarse)**		Lost DF circulation at 15 ft.		
20	N6	72	16-15-11			N- 6 (20.00-21.50) Silty GRAVEL with some sand; GM; Brown to gray; Low plasticity fines; Moist; Medium dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse sand; Micaceous; (Missoula Flood Deposits - Coarse)			Advanced 6-inch OD casing to 20 ft. Regained circulation briefly after install, then lost DF circulation at 22 ft. Lost approx. 1250 gallons DF between 22 and 25 ft.		
25	N7	67	17-19-20			N- 7 (25.00-26.50) Silty GRAVEL with some sand; GM; Brown to gray; Low plasticity fines; Moist; Dense; Fine to coarse, subangular to subrounded gravel; Fine to medium sand; Micaceous; (Missoula Flood Deposits - Coarse)			Backfilled hole with bentonite chips on August 19, 2020. Continued drilling on boring B-3 on August 31, 2020.		
30	N8	67	16-18-12			N- 8 (30.00-31.50) Sandy silty GRAVEL; GM; Brown to gray; Nonplastic fines; Moist; Medium dense to dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse sand; Micaceous; Trace decomposed gravel fragments; (Missoula Flood Deposits - Coarse)					
35	N9	0	20-12-16			N- 9 (35.00-36.50) No recovery					
38											

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Depth (ft)	Test Type, No.	Percent Recovery	Soil: Driving Resistance / Rock: Discontinuity Data Or RQD%	Percent Natural Moisture	Material Description (SOIL: Soil Name, USCS, Color, Plasticity, Moisture, Consistency/Relative Density, Texture, Cementation, Structure, Origin. ROCK: Rock Name, Color, Weathering, Hardness, Discontinuity Spacing, Joint Filling, Core Recovery, Formation Name.)	Unit Description	Graphic Log	Drilling Methods, Size and Remarks	Water Level/ Date	Backfill/ Instrumentation
38										
40	N10	50	10-12-12		N- 10 (40.00-41.50) Sandy silty GRAVEL; GM; Brown to gray; Low plasticity fines; Moist; Medium dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse sand; Micaceous; (Missoula Flood Deposits - Coarse)					
45	N11	56	15-27-20		N- 11 (45.00-46.50) Sandy silty GRAVEL; GM; Brown to gray; Low plasticity fines; Moist to wet; Dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse sand; Micaceous; (Missoula Flood Deposits - Coarse)					
50	N12	44	13-9-11		N- 12 (50.00-51.50) Silty GRAVEL with some sand; GM; Brown to gray; Nonplastic fines; Moist to wet; Medium dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse sand; Micaceous; (Missoula Flood Deposits - Coarse)					
						52.50 - 57.50 SILT; ML; Blue-gray; Low plasticity; Moist; Stiff; Micaceous; (Pliocene / Pleistocene Sediments)				
55	N13	100	3-4-5		N- 13 (55.00-56.50) SILT; ML; Blue-gray; Low plasticity; Moist; Stiff; Micaceous; (Pliocene / Pleistocene Sediments)					
						57.50 - 62.50 Clayey SAND to Clayey SAND with trace to some gravel; SC; Brown; Low to medium plasticity fines; Moist; Medium dense; Fine to				
60										

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Depth (ft)	Test Type, No.	Percent Recovery	Soil: Driving Resistance	Rock: Discontinuity Data Or RQD%	Percent Natural Moisture	Material Description (SOIL: Soil Name, USCS, Color, Plasticity, Moisture, Consistency/Relative Density, Texture, Cementation, Structure, Origin. ROCK: Rock Name, Color, Weathering, Hardness, Discontinuity Spacing, Joint Filling, Core Recovery, Formation Name.)	Unit Description	Graphic Log	Drilling Methods, Size and Remarks	Water Level/ Date	Backfill/ Instrumentation
60	U1	100				U- 1 (60.00-61.00) Clayey SAND; SC; Brown; Low to medium plasticity fines; Moist; Fine to coarse sand; Micaceous; (Pliocene / Pleistocene Sediments)	**coarse, subangular to subrounded gravel; Fine to coarse sand Slight iron oxide staining; (Pliocene / Pleistocene Sediments)**				
	N14	100	6-7-7			N- 14 (61.00-62.50) Clayey SAND with trace to some gravel; SC; Brown; Medium plasticity fines; Moist; Medium dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse sand; Slight iron oxide staining; (Pliocene / Pleistocene Sediments)					
							62.50 - 67.50 CLAY with trace to some sand; CH; Gray to brown; Medium plasticity; Moist; Very stiff; Fine to coarse sand; Micaceous; Slight to moderate iron oxide staining; (Pliocene / Pleistocene Sediments)				
65	N15	100	8-10-15			N- 15 (65.00-66.50) CLAY with trace to some sand; CH; Gray to brown; Medium plasticity; Moist; Very stiff; Fine to coarse sand; Micaceous; Slight to moderate iron oxide staining; (Pliocene / Pleistocene Sediments)					
							67.50 - 76.50 Sandy silty CLAY with trace to some gravel; CL; Brown to gray; Medium plasticity; Moist; Hard; Fine subangular to subrounded gravel; Fine to coarse sand; Micaceous; Slight iron oxidation and staining; (Pliocene / Pleistocene Sediments)				
70	N16	56	12-17-19			N- 16 (70.00-71.50) Sandy silty CLAY with some gravel; CL; Brown to gray; Medium plasticity; Moist; Hard; Fine subangular to subrounded gravel; Fine to coarse sand; Micaceous; Slight iron oxidation and staining; (Pliocene / Pleistocene Sediments)					
75	N17	56	18-27-29			N- 17 (75.00-76.50) Sandy silty CLAY with trace gravel; CL; Blue-gray; Low to medium plasticity; Moist; Hard; Fine subangular to subrounded gravel; Fine to coarse sand; Micaceous; Slight iron oxidation and staining; (Pliocene / Pleistocene Sediments)					
							76.50 End of hole				
80											
83											

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DRILL LOG
OREGON DEPARTMENT OF TRANSPORTATION

		Hole No. **B-4**
Project **I-5 Pedestrian Bridge Barber St. to Wilsonville Town Center**	Purpose **Bridge Foundation**	E.A. No. **N/A**
Highway **Interstate 5**	County **Clackamas**	Key No. **N/A**
Hole Location Northing: **~ 93,963** Easting: **~ 323,372**		Start Card No. **N/A**
Equipment **CME 75 Truck Rig (Hammer Efficiency = 78.4%)**	Driller **Western States**	Bridge No. **N/A**
Project Geologist **Seth C. Sonnier, RG**	Recorder **Laruen Sherman**	Ground Elev. **~ 172 ft.**
Start Date **April 6, 2020** End Date **April 6, 2020**	Total Depth **75.50 ft**	Tube Height **N/A**

Test Type
"A" - Auger Core; "GP" - GeoProbe®; "X" - Auger; "C" - Core, Barrel Type; "N" - Standard Penetration; "U" - Undisturbed Sample; "T" - Test Pit

Rock Abbreviations

Discontinuity	Shape	Surface Roughness
J - Joint	Pl - Planar	P - Polished
F - Fault	C - Curved	Sl - Slickensided
B - Bedding	U - Undulating	Sm - Smooth
Fo - Foliation	St - Stepped	R - Rough
S - Shear	Ir - Irregular	VR - Very Rough

Typical Drilling Abbreviations

Drilling Methods	Drilling Remarks
WL - Wire Line	LW - Lost Water
HS - Hollow Stem Auger	WR - Water Return
DF - Drill Fluid	WC - Water Color
SA - Solid Auger	DP - Down Pressure
CA - Casing Advancer	DR - Drill Rate
HA - Hand Auger	DA - Drill Action

Material Description — SOIL: Soil Name, USCS, Color, Plasticity, Moisture, Consistency/Relative Density, Texture, Cementation, Structure, Origin. ROCK: Rock Name, Color, Weathering, Hardness, Discontinuity Spacing, Joint Filling, Core Recovery, Formation Name.

Depth (ft)	Test Type, No.	Percent Recovery	Soil: Driving Resistance / Rock: Discontinuity Data Or RQD%	Percent Natural Moisture	Material Description	Unit Description	Graphic Log	Drilling Methods, Size and Remarks	Water Level/ Date	Backfill/ Instrumentation
0						**0.00 - 0.50 Topsoil**				
						0.50 - 6.50 Silty CLAY with trace sand; CL; Red-brown mottled; Medium plasticity; Damp; Stiff to very stiff; Fine to medium sand; Trace organics; Slight iron oxide staining; (Fill)				
	N1	100	3-5-7		N- 1 (2.50-4.00) Silty CLAY with trace sand; CL; Red-brown mottled; Medium plasticity; Damp; Stiff; Fine to medium sand; Trace organics; Slight iron oxide staining; (Fill)					
5	N2	100	5-6-13		N- 2 (5.00-6.50) Silty CLAY with trace sand; CL; Red-brown mottled; Medium plasticity; Damp; Very stiff; Fine sand; Trace organics; Slight iron oxide staining; (Fill)			N2 Specific gravity at 20°C = 2.6306.		
						6.50 - 9.50 Clayey GRAVEL with some sand, with possible cobbles and boulders; GC; Brown to gray; Medium plasticity fines; Moist to wet; Very dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse sand; (Missoula Flood Deposits - Coarse)				
	N3	81	19-23-28		N- 3 (7.50-9.00) Clayey GRAVEL with some sand; GC; Brown to gray; Medium plasticity fines; Moist to wet; Very dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse sand; (Missoula Flood Deposits - Coarse)					
						9.50 - 12.50 Sandy clayey GRAVEL, with possible cobbles and boulders; GC; Brown to gray; Medium plasticity; Moist; Very dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse sand; (Missoula Flood Deposits - Coarse)				
10	N4	72	26-20-49		N- 4 (10.00-11.50) Sandy clayey GRAVEL; GC; Brown to gray; Medium plasticity; Moist; Very dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse sand; (Missoula Flood Deposits - Coarse)					
15										

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Depth (ft)	Test Type, No.	Percent Recovery	Soil: Driving Resistance / Rock: Discontinuity Data Or RQD%	Percent Natural Moisture	Material Description (SOIL: Soil Name, USCS, Color, Plasticity, Moisture, Consistency/Relative Density, Texture, Cementation, Structure, Origin. ROCK: Rock Name, Color, Weathering, Hardness, Discontinuity Spacing, Joint Filling, Core Recovery, Formation Name.)	Unit Description	Graphic Log	Drilling Methods, Size and Remarks	Water Level/ Date	Backfill/ Instrumentation
15	N5	44	7-13-18		N- 5 (15.00-16.50) Silty GRAVEL with some sand; GM; Brown to gray; Low to medium plasticity; Wet; Dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse sand; (Missoula Flood Deposits - Coarse)	**12.50 - 42.50 Silty GRAVEL with some sand to Sandy silty GRAVEL, with possible cobbles and boulders; GM; Brown to gray; Low to medium plasticity; Moist to wet; Medium dense to dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse sand; (Missoula Flood Deposits - Coarse)**				
20	N6	0	36-50/5"		N- 6 (20.00-20.90) No Recovery					
25	N7	39	13-14-9		N- 7 (25.00-26.50) Silty GRAVEL with some sand; GM; Brown to gray; Low plasticity fines; Moist to wet; Medium dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse sand; (Missoula Flood Deposits - Coarse)					
30	N8	28	11-12-12		N- 8 (30.00-31.50) Silty GRAVEL with some sand; GM; Brown to gray; Low plasticity fines; Moist; Medium dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse sand; (Missoula Flood Deposits - Coarse)					
35	N9	33	10-14-10		N- 9 (35.00-36.50) Sandy silty GRAVEL; GM; Brown to gray; Low plasticity fines; Moist; Medium dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse sand; (Missoula Flood Deposits - Coarse)					
38										

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Depth (ft)	Test Type, No.	Percent Recovery	Soil: Driving Resistance / Rock: Discontinuity Data Or RQD%	Percent Natural Moisture	Material Description (SOIL: Soil Name, USCS, Color, Plasticity, Moisture, Consistency/Relative Density, Texture, Cementation, Structure, Origin. ROCK: Rock Name, Color, Weathering, Hardness, Discontinuity Spacing, Joint Filling, Core Recovery, Formation Name.)	Unit Description	Graphic Log	Drilling Methods, Size and Remarks	Water Level/ Date	Backfill/ Instrumentation
38								Lost DF circulation at 38 ft. Borehole caving and sloughing from approx. 35 to 40 ft. Installed 6-inch OD casing to 40 ft. to prevent further caving.		
40	N10	3	8-16-17		N- 10 (40.00-41.50) Silty GRAVEL with some sand; GM; Brown to gray; Nonplastic fines; Moist to wet; Dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse sand; (Missoula Flood Deposits - Coarse)					
						42.50 - 52.50 SILT; ML; Gray; Low plasticity; Moist; Stiff to very stiff; Micaceous; (Pliocene / Pleistocene Sediments)				
45	N11	89	4-5-8	31	N- 11 (45.00-46.50) SILT; ML; Gray; Low plasticity; Moist; Stiff; Micaceous; (Pliocene / Pleistocene Sediments)			Atterberg Limits N11: LL=37, PL=26, PI=11. N11: 99% fines		
50	N12	67	8-12-12		N- 12 (50.00-51.50) SILT; ML; Gray; Low plasticity; Moist; Very stiff; Micaceous; (Pliocene / Pleistocene Sediments)					
						52.50 - 62.50 CLAY with trace to some sand; CH; Blue-gray mottled; High plasticity; Moist; Very stiff; Fine to coarse sand; Moderate iron oxide staining; (Pliocene / Pleistocene Sediments)				
55	N13	100	12-14-15		N- 13 (55.00-56.50) CLAY with some sand; CH; Blue-gray mottled; High plasticity; Moist; Very stiff; Fine to coarse sand; Moderate iron oxide staining; (Pliocene / Pleistocene Sediments)					
60										

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Depth (ft)	Test Type, No.	Percent Recovery	Soil: Driving Resistance / Rock: Discontinuity Data Or RQD%	Percent Natural Moisture	Material Description (SOIL: Soil Name, USCS, Color, Plasticity, Moisture, Consistency/Relative Density, Texture, Cementation, Structure, Origin. ROCK: Rock Name, Color, Weathering, Hardness, Discontinuity Spacing, Joint Filling, Core Recovery, Formation Name.)	Unit Description	Graphic Log	Drilling Methods, Size and Remarks	Water Level/ Date	Backfill/ Instrumentation
60	N14	100	3-8-10		N- 14 (60.00-61.50) CLAY with trace sand; CH; Blue-gray mottled; High plasticity; Moist; Very stiff; Fine to coarse sand; Moderate iron oxide staining; (Pliocene / Pleistocene Sediments)					
						62.50 - 67.50 Silty CLAY with some sand and trace gravel; CL; Blue-gray and brown; High plasticity; Moist; Very stiff; Fine to coarse, subangular to subrounded gravel; Fine to coarse sand; Moderate iron oxide staining; (Pliocene / Pleistocene Sediments)				
65	N15	100	10-13-16		N- 15 (65.00-66.50) Silty CLAY with some sand and trace gravel; CL; Blue-gray and brown; High plasticity; Moist; Very stiff; Fine to coarse, subangular to subrounded gravel; Fine to coarse sand; Moderate iron oxide staining; (Pliocene / Pleistocene Sediments)					
						67.50 - 72.50 Silty CLAY; CL; Green-gray; Low to medium plasticity; Damp; Hard; (Pliocene / Pleistocene Sediments)				
70	N16	100	13-14-22		N- 16 (70.00-71.50) Silty CLAY; CL; Green-gray; Low to medium plasticity; Damp; Hard; (Pliocene / Pleistocene Sediments)					
						72.50 - 75.50 Clayey GRAVEL with some sand; GC; Blue-gray; Medium plasticity; Moist; Very dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse sand; Moderate iron oxide staining; (Pliocene / Pleistocene Sediments)				
75	N17	33	50/1st 6"		N- 17 (75.00-75.50) Clayey GRAVEL with some sand; GC; Blue-gray; Medium plasticity; Moist; Very dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse sand; Moderate iron oxide staining; (Pliocene / Pleistocene Sediments)	**75.50 End of hole**				
80										
83										

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SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

DRILL LOG
OREGON DEPARTMENT OF TRANSPORTATION

		Hole No. **B-5**
Project **I-5 Pedestrian Bridge Barber St. to Wilsonville Town Center**	Purpose **Bridge Foundation**	E.A. No. **N/A**
Highway **Interstate 5**	County **Clackamas**	Key No. **N/A**
Hole Location Northing: **~ 93,991** Easting: **~ 323,476**		Start Card No. **N/A**
Equipment **CME 75 Truck Rig (Hammer Efficiency = 69.2%**	Driller **Western States**	Bridge No. **N/A**
Project Geologist **Seth C. Sonnier, RG**	Recorder **David Jacobson**	Ground Elev. **~ 173 ft.**
Start Date **August 21, 2020** End Date **August 21, 2020**	Total Depth **61.50 ft**	Tube Height **N/A**

Test Type
"A" - Auger Core
"GP" - GeoProbe®
"X" - Auger
"C" - Core, Barrel Type
"N" - Standard Penetration
"U" - Undisturbed Sample
"T" - Test Pit

Rock Abbreviations

Discontinuity	Shape	Surface Roughness
J - Joint	Pl - Planar	P - Polished
F - Fault	C - Curved	Sl - Slickensided
B - Bedding	U - Undulating	Sm - Smooth
Fo - Foliation	St - Stepped	R - Rough
S - Shear	Ir - Irregular	VR - Very Rough

Typical Drilling Abbreviations

Drilling Methods	Drilling Remarks
WL - Wire Line	LW - Lost Water
HS - Hollow Stem Auger	WR - Water Return
DF - Drill Fluid	WC - Water Color
SA - Solid Auger	DP - Down Pressure
CA - Casing Advancer	DR - Drill Rate
HA - Hand Auger	DA - Drill Action

Material Description: SOIL: Soil Name, USCS, Color, Plasticity, Moisture, Consistency/Relative Density, Texture, Cementation, Structure, Origin. ROCK: Rock Name, Color, Weathering, Hardness, Discontinuity Spacing, Joint Filling, Core Recovery, Formation Name.

Depth (ft)	Test Type, No.	Percent Recovery	Soil: Driving Resistance / Rock: Discontinuity Data Or RQD%	Percent Natural Moisture	Material Description	Unit Description	Graphic Log	Drilling Methods, Size and Remarks	Water Level/ Date	Backfill/ Instrumentation
0						**0.00 - 0.50 Topsoil**				
						0.50 - 4.50 CLAY; CH; Brown; Medium plasticity; Moist; Stiff to very stiff; Micaceous; (Fill)				
	N1	67	3-7-8	38	N- 1 (2.50-4.00) CLAY; CH; Brown; Medium plasticity; Moist; Stiff to very stiff; Micaceous; (Fill)			Atterberg Limit N1: LL=51, PL=26, PI=25.		
						4.50 - 7.00 SILT with some sand; ML; Brown; Low plasticity; Moist; Stiff; Fine sand; Micaceous; (Fill)				
5	N2	100	4-6-8	37	N- 2 (5.00-6.50) SILT with some sand; ML; Brown; Low plasticity; Moist; Stiff; Fine sand; Micaceous; (Fill)			N2: 18% sand, 82% fines.		
						7.00 - 14.50 Silty GRAVEL with some sand, with possible cobbles and boulders; GM; Brown to gray; Nonplastic to low plasticity fines; Moist; Dense to very dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse sand; Micaceous; (Missoula Flood Deposits - Coarse)				
	N3	78	27-48-29		N- 3 (7.50-9.00) Silty GRAVEL with some sand; GM; Brown to gray; Low plasticity fines; Moist; Very dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse sand; Micaceous; (Missoula Flood Deposits - Coarse)					
10	N4	67	17-27-27		N- 4 (10.00-11.50) Silty GRAVEL with some sand; GM; Brown to gray; Low plasticity fines; Moist; Very dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse sand; Micaceous; (Missoula Flood Deposits - Coarse)					
	N5	67	22-20-19		N- 5 (12.50-14.00) Silty GRAVEL with some sand; GM; Brown to gray; Nonplastic to low plasticity fines; Moist; Dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse sand; Micaceous; (Missoula Flood Deposits - Coarse)					
						14.50 - 19.00				
15										

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Depth (ft)	Test Type, No.	Percent Recovery	Soil: Driving Resistance / Rock: Discontinuity Data Or RQD%	Percent Natural Moisture	Material Description (SOIL: Soil Name, USCS, Color, Plasticity, Moisture, Consistency/Relative Density, Texture, Cementation, Structure, Origin. ROCK: Rock Name, Color, Weathering, Hardness, Discontinuity Spacing, Joint Filling, Core Recovery, Formation Name.)	Unit Description	Graphic Log	Drilling Methods, Size and Remarks	Water Level/ Date	Backfill/ Instrumentation
15	N6	22	13-12-14		N- 6 (15.00-16.50) GRAVEL with some silt and sand; GP-GM; Brown to gray; Nonplastic fines; Moist to wet; Medium dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse sand; Micaceous; (Missoula Flood Deposits - Coarse)	**GRAVEL with some silt and sand, with possible cobbles and boulders; GP-GM; Brown to gray; Nonplastic fines; Moist to wet; Medium dense to very dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse sand; Micaceous; (Missoula Flood Deposits - Coarse)**		Lost DF circulation at 15 ft after SPT for sample N6. Never regained circulation throughout entire drilling of the borehole. Total drilling mud loss of 1200 gallons.		
	N7	33	10-50/3"		N- 7 (17.50-18.25) GRAVEL with some silt and sand; GP-GM; Brown to gray; Nonplastic fines; Moist to wet; Very dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse sand; Micaceous; (Missoula Flood Deposits - Coarse)					
20	N8	86	40-50/1"		N- 8 (20.00-20.60) Sandy silty GRAVEL; GM; Brown to gray; Low plasticity fines; Moist to wet; Very dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse sand; Micaceous; (Missoula Flood Deposits - Coarse)	**19.00 - 22.50 Sandy silty GRAVEL; GM; Brown to gray; Low plasticity fines; Moist to wet; Very dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse sand; Micaceous; (Missoula Flood Deposits - Coarse)**				
25	N9	17	10-10-10		N- 9 (25.00-26.50) Silty GRAVEL with some sand; GM; Brown to gray; Nonplastic fines; Moist; Medium dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse sand; Micaceous; (Missoula Flood Deposits - Coarse)	**22.50 - 42.50 Silty GRAVEL with some sand to Sandy silty GRAVEL, with possible cobbles and boulders; GM; Brown to gray; Nonplastic fines; Moist; Medium dense to dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse sand; Micaceous; (Missoula Flood Deposits - Coarse)**				
30	N10	78	18-18-12		N- 10 (30.00-31.50) Sandy silty GRAVEL; GM; Brown to gray; Nonplastic fines; Moist; Medium dense to dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse sand; Micaceous; (Missoula Flood Deposits - Coarse)					
35	N11	56	6-12-10		N- 11 (35.00-36.50) Silty GRAVEL with some sand; GM; Brown to gray; Nonplastic fines; Moist; Medium dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse sand; Micaceous; (Missoula Flood Deposits - Coarse)			Driller notes significant borehole instability in loose gravels at approx. 30 to 40 ft.		
38										

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Depth (ft)	Test Type, No.	Percent Recovery	Soil: Driving Resistance / Rock: Discontinuity Data Or RQD%	Percent Natural Moisture	Material Description (SOIL: Soil Name, USCS, Color, Plasticity, Moisture, Consistency/Relative Density, Texture, Cementation, Structure, Origin. ROCK: Rock Name, Color, Weathering, Hardness, Discontinuity Spacing, Joint Filling, Core Recovery, Formation Name.)	Unit Description	Graphic Log	Drilling Methods, Size and Remarks	Water Level/ Date	Backfill/ Instrumentation
38										
40	N12	44	8-14-11		N- 12 (40.00-41.50) Silty GRAVEL with some sand; GM; Brown to gray; Low plasticity fines; Moist to wet; Medium dense; Fine to coarse, subangular to subrounded gravel; Fine to coarse sand; Micaceous; (Missoula Flood Deposits - Coarse)					
						42.50 - 54.00 SILT; ML; Blue-gray; Low plasticity; Moist; Stiff to very stiff; Micaceous; (Pliocene / Pleistocene Sediments)				
45	N13	100	4-8-7	34	N- 13 (45.00-46.50) SILT; ML; Blue-gray; Low plasticity; Moist; Stiff to very stiff; Micaceous; (Pliocene / Pleistocene Sediments)					
50	U1	100			U- 1 (50.00-52.00) SILT; ML; Blue-gray; Low plasticity; Moist; (Pliocene / Pleistocene Sediments)					
	N14	100	8-11-8	34	N- 14 (52.00-53.50) SILT; ML; Blue-gray; Low plasticity; Moist; Very stiff; (Pliocene / Pleistocene Sediments)					
						54.00 - 57.50 Silty CLAY; CL; Gray; Medium plasticity; Moist; Stiff; Micaceous; Trace organics; (Pliocene / Pleistocene Sediments)				
55	N15	100	2-4-7	39	N- 15 (55.00-56.50) Silty CLAY; CL; Gray; Medium plasticity; Moist; Stiff; Micaceous; Trace organics; (Pliocene / Pleistocene Sediments)					
						57.50 - 61.50 Silty CLAY with some sand; CL; Gray to brown; Medium to high plasticity; Moist; Very stiff; Fine subangular to subrounded gravel;				
60										

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Depth (ft)	Test Type, No.	Percent Recovery	Soil: Driving Resistance / Rock: Discontinuity Data Or RQD%	Percent Natural Moisture	Material Description (SOIL: Soil Name, USCS, Color, Plasticity, Moisture, Consistency/Relative Density, Texture, Cementation, Structure, Origin. ROCK: Rock Name, Color, Weathering, Hardness, Discontinuity Spacing, Joint Filling, Core Recovery, Formation Name.)	Unit Description	Graphic Log	Drilling Methods, Size and Remarks	Water Level/ Date	Backfill/ Instrumentation
60	N16	100	4-9-10		N- 16 (60.00-61.50) Silty CLAY with some sand; CL; Gray to brown; Medium to high plasticity; Moist; Very stiff; Fine subangular to subrounded gravel; Fine to coarse sand; Moderate iron oxide staining; (Pliocene / Pleistocene Sediments)	**Fine to coarse sand; Moderate iron oxide staining; (Pliocene / Pleistocene Sediments)**				
						61.50 End of hole				
65										
70										
75										
80										
83										

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APPENDIX B: LABORATORY TEST RESULTS

Appendix B

Laboratory Test Results

CONTENTS

B.1 General B-1
B.2 Soil Testing B-1
B.2.1 Moisture (Natural Water) Content B-1
B.2.2 Atterberg Limits B-1
B.2.3 Particle-Size Analyses B-2
B.2.4 Specific Gravity Testing B-2
B.3 Corrosivity Testing B-2

Figures

Figure B1: Atterberg Limits Results
Figure B2: Grain Size Distribution Results

Attachments

GeoTesting Express Inc., Technical Report, dated September 15, 2020

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APPENDIX B: LABORATORY TEST RESULTS

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B.1 GENERAL

Soil samples obtained during the field exploration activities were described and identified in the field by Shannon & Wilson, Inc. Physical characteristics of the collected samples were noted, and field descriptions and identifications were modified, as necessary, in accordance with the ODOT Soil and Rock Classification Manual (1987). During the review, representative soil samples were selected for further testing. The material descriptions and identifications were refined/revised, as necessary, based on the results of the laboratory tests.

The soil testing program included natural moisture contents, Atterberg limits testing, particle size analyses, specific gravity testing, and soil corrosivity testing. Laboratory testing was performed by Shannon & Wilson and by GeoTesting Express of Acton, Massachusetts. 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 Drill Logs 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 were conducted on samples to determine their grain-size distributions. Grain size distributions were determined in accordance with ASTM D422, D6913 and D1140 as applicable. For all samples, 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. For some tests, only the percentage of the sample passing the No. 200 (0.075mm) sieve was determined (ASTM D1140). For one sample from boring B-5, a hydrometer analysis was performed on material passing the No. 200 (0.075mm) sieve (ASTM D422). Results of all particle-size analyses are presented on Figure B2, Grain Size Distribution. The resulting gravel, sand and fines percentages are also presented on the Drill Logs in Appendix A.

B.2.4 Specific Gravity Testing

Specific gravity testing was conducted on one sample in accordance with ASTM D854. The specific gravity is the density of the mineral solids in soil, normalized to the density of water. In accordance with ASTM D854, the soil slurry was de-aired by boiling without the use of a vacuum. The result is presented on the Drill Log for Boring B-4 in Appendix A.

B.3 CORROSIVITY TESTING

Analytical testing was performed on a sample from boring B-4 to evaluate the corrosivity potential of the soil at the site. The corrosivity test suite included resistivity, chloride concentration, soil pH, and sulfate concentration. An analytical testing report, prepared by GeoTesting Express, is attached to the end of this appendix.

The corrosion potential of a soil is primarily evaluated by comparing measured pH, resistivity, and sulfate and chloride concentrations to the values specified in Section 10.7.5 of the AASHTO LFRD Bridge Design Specifications (9th Edition 2020).

Soil pH is a measurement of the hydrogen ion activity of the soil. Soil pH is reported in Standard Units (S.U.) on a scale ranging from 0 to 14, with 7 being neutral. Soils with a pH less than 7 are considered acidic, and soils with a pH greater than 7 are considered alkaline. According to the AASHTO specifications, soils with a pH less than 5.5 and soils with a pH between 5.5 and 8.5 that also have high organic content are considered potentially corrosive.

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Resistivity (expressed as ohms-centimeter or ohms-cm) is the numerical expression of the ability of a soil to impede the transmission of an electrical current. Resistivity is the inverse of conductivity and is dependent on the presence of ions, their concentrations, mobility, and valence, as well as soil moisture and temperature. The AASHTO specifications state that effects of corrosion and deterioration shall be considered if resistivity values are less than 2,000 ohms-cm.

Sulfate and chloride concentrations were measured in each soil sample. Sulfates can be converted to sulfides by naturally occurring bacteria. Sulfides, when allowed to oxidize, will produce sulfuric acid, which is highly corrosive. Chlorides will also chemically react and facilitate dissolution reactions with metals and concrete. According to the AASHTO specifications, the soil is considered corrosive if the concentration of sulfate is greater than 1,000 parts per million (ppm) or the concentration of chloride is greater than 500 ppm.

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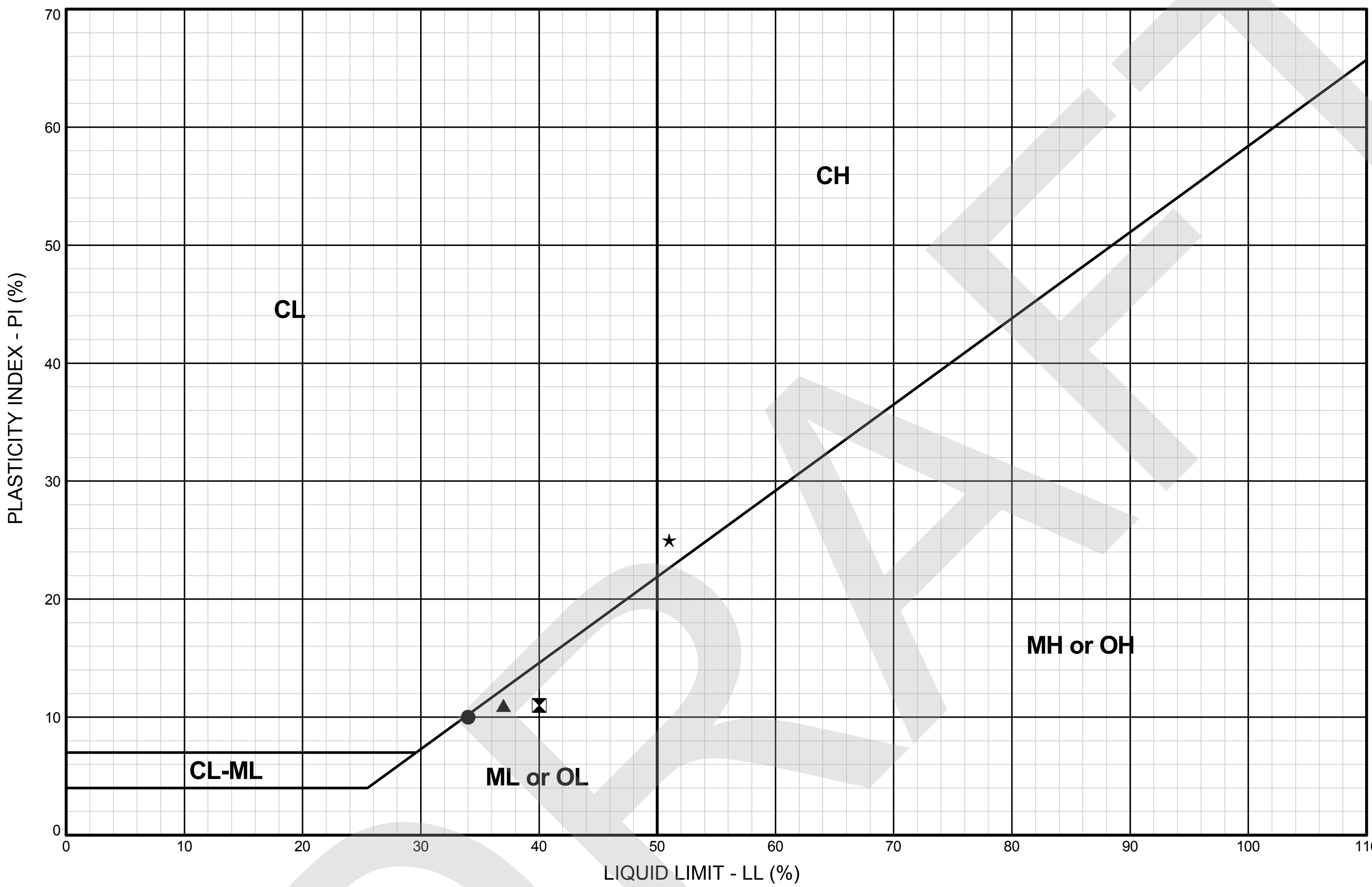

NOTES

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) (< 3%)
- Low Plasticity (3 to 15%)
- Medium Plasticity (15 to 30%)
- High Plasticity (> 30%)

BORING AND SAMPLE NO.	DEPTH (feet)	GROUP SYMBOL[2]	GROUP NAME[2]	LL %	PL %	PI %[3]	NAT. W.C. %	FINES %
● B-1, N14	50.0	*ML*	*SILT with trace sand*	34	24	10	31	
⧗ B-2, N16	60.0	*ML*	*SILT with some sand*	40	29	11	33	78
▲ B-4, N11	45.0	*ML*	*SILT*	37	26	11	31	99
★ B-5, N1	2.5	*CH*	*CLAY*	51	26	25	38	

I-5 Pedestrian Bridge
Barber St. to Wilsonville Town Center
Wilsonville, Oregon

ATTERBERG LIMITS RESULTS

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FIG. B1

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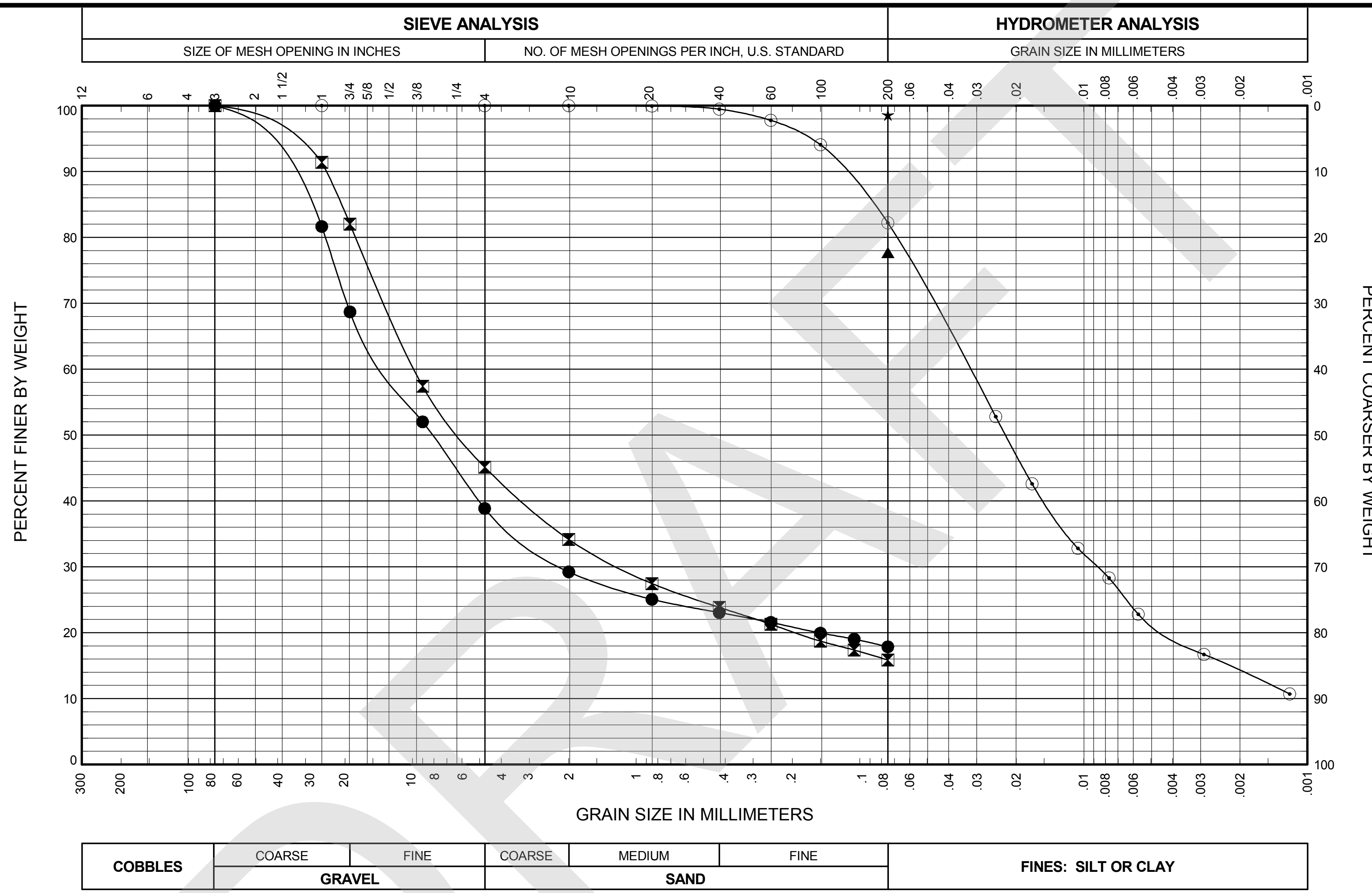

NOTES:
1) Sieve analyses were performed in general accordance with ASTM D6913, sieve with hydrometer analyses were performed in general accordance with ASTM D422, and amount finer than #200 sieve analyses were performed in general accordance with ASTM D1140 unless otherwise noted in the report.
2) Group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

GSA_MAIN 103953.GPJ SHAN_WIL.GDT 11/13/20

BORING AND SAMPLE NO.	DEPTH (feet)	GROUP SYMBOL[2]	GROUP NAME[2]	GRAVEL %	SAND %	FINES %	NAT. W.C. %	DRY DENSITY PCF
● B-2, N2	5.0	*GM*	*(Composite N2, N3, N4) Sandy silty GRAVEL*	61	21	18	17	
⧗ B-2, N11	35.0	*GM*	*(Composite N11, N12, N13) Sandy silty GRAVEL*	55	29	16	19	
▲ B-2, N16	60.0	*ML*	*SILT with some sand*	-	-	78	33	
★ B-4, N11	45.0	*ML*	*SILT*	-	-	99	31	
⊙ B-5, N2	5.0	*ML*	*SILT with some sand*	0	18	82	37	

I-5 Pedestrian Bridge
Barber St. to Wilsonville Town Center
Wilsonville, Oregon

GRAIN SIZE DISTRIBUTION

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FIG. B2

FIG. B2

Client:	Shannon & Wilson, Inc.				
Project:	Wilsonville I-5				
Location:	Portland, OR			Project No:	GTX-312321
Boring ID:	B-4	Sample Type:	bag	Tested By:	PK
Sample ID:	N1	Test Date:	09/08/20	Checked By:	bfs
Depth :	2.5 - 4 ft	Test Id:	577751		

Test Comment: ---
Visual Description: Moist, brown clay
Sample Comment: ---

pH of Soil by ASTM D4972

Boring ID	Sample ID	Depth	Visual Description	pH of Soil in Distilled Water	pH of Soil in Calcium Chloride
B-4	N1	2.5 - 4 ft	Moist, brown clay	5.9	5.0

Notes: Sample Preparation: screened through #10 sieve
Method A, pH meter used

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Client:	Shannon & Wilson, Inc.
Project:	Wilsonville I-5
Location:	Portland, OR
GTX#:	312321
Test Date:	09/08/20
Tested By:	FMJ
Checked By:	bfs

Laboratory Measurement of Soil Resistivity Using the Wenner Four-Electrode Method by ASTM G57 (Laboratory Measurement)

Boring ID	Sample ID	Depth, m	Sample Description	Electrical Resistivity, ohm-cm	Electrical Conductivity, $(ohm\text{-}cm)^{-1}$
B-4	N1	2.5-4	Moist, brown clay	5,475	1.83E-04

Notes: Test Equipment: Nilsson Model 400 Soil Resistance Meter, MC Miller Soil Box
Water added to sample to create a thick slurry prior to testing (saturated condition).
Electrical Conductivity is calculated as inverse of Electrical Resistivity (per ASTM G57)
Test conducted in standard laboratory atmosphere: 68-73 F

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testing services sm
PO Box 572455 / Salt Lake City UT 84157-2455 / USA
TEL +1 801 262 2448 · FAX +1 801 262 9870 · www.TEi-TS.com

GEOTESTING EXPRESS INCORPORATED
125 NAGOG PARK
ACTON MA 01720-3451
USA

Analysis No.	TS-A2008937
Report Date	12 September 2020
Date Sampled	04 September 2020
Date Received	09 September 2020
Where Sampled	Acton, MA USA
Sampled By	Client

This is to attest that we have examined: Wilsonville I-5; Site Location: Portland, OR; Job Number: GTX-312321

When examined to the applicable requirements of:

ASTM D 512-12	“Standard Test Methods for Chloride Ion in Water” Method B
ASTM D 516-16	“Standard Test Method for Sulfate Ion in Water”
ASTM G 200-14	“Standard Test Method for Measurement of Oxidation-Reduction Potential (ORP) of Soil”

Results:

ASTM D 512 – Chloride Method B

Sample		Results		Detection Limit
		ppm (mg/kg)	%[1]	
N1		17.	0.0017	10.
B-4	2.5 – 4’			

NOTE: [1]Percent by weight as received.

ASTM D 516 – Sulfates (Soluble)

Sample		Results		Detection Limit
		ppm (mg/kg)	%[1]	
N1		12.	0.0012	10.
B-4	2.5 – 4’			

NOTE: [1]Percent by weight as received

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testing services analytical laboratory
3455 South 500 West
Salt Lake City, UT 84115-4234 USA
TEL: +1 801 262 2448
FAX: +1 801 262 9870

ASTM G 200 – Reduction Oxidation Potential (REDOX)

Sample		Results	Detection Limit
N1		209.0 @ 24.2 ºC	0.1 mV
B-4	2.5 – 4’		

END OF ANALYSIS

USEPA Laboratory ID UT00930

Merrill Gee P.E. – Engineer in Charge

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APPENDIX C: GLOBAL STABILITY ANALYSIS RESULTS

Appendix C

Global Stability Analysis Results

CONTENTS

- Figures C1 to C6

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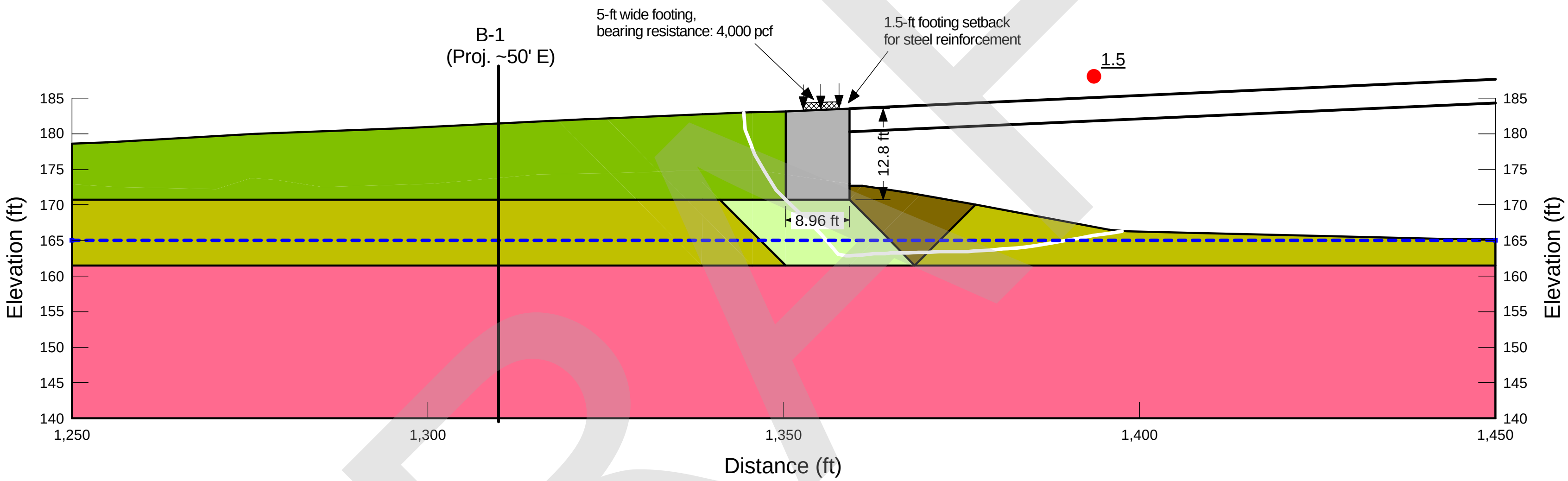

Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	Borrow Material	125	0	32
	Existing Fill (V. Stiff ML)	120	0	33
	MFD-Coarse (GM)	125	0	36
	MSE Abutment Wall	130		
	MSE Fill	130	0	34
	Stone Embankment Material	125	0	36

NOTES

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

I-5 Pedestrian Bridge
Barbur St. to Wilsonville Town Center
Wilsonville, Oregon

WEST MSE ABUTMENT
STATIC CONDITION
GLOBAL STABILITY ANALYSIS

November 2020 103953

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FIG. C1

\\PDX-FS1\Vol1\EF\PDX\103000s\103953 Wilsonville I-5\Analysis\03 - MSE Walls\rev2\I-5 Ped MSE abutments.gsz

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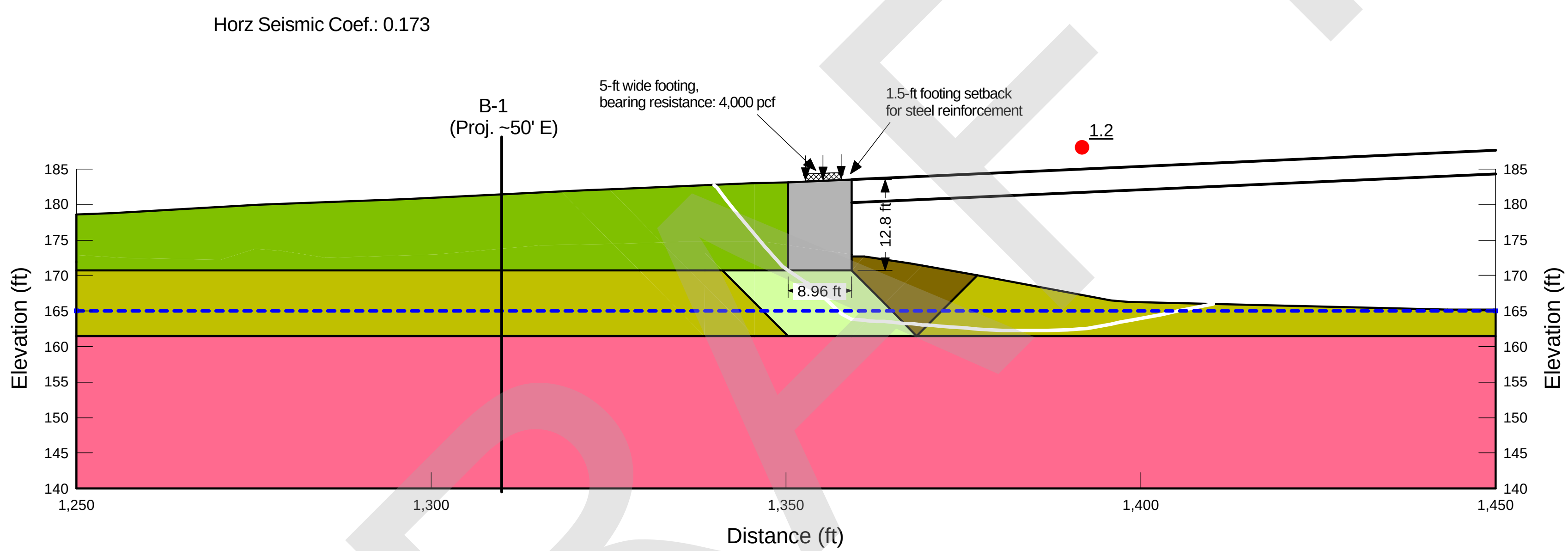

Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	Borrow Material	125	0	32
	Existing Fill (V. Stiff ML)	120	0	33
	MFD-Coarse (GM)	125	0	36
	MSE Abutment Wall	130		
	MSE Fill	130	0	34
	Stone Embankment Material	125	0	36

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

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WEST MSE ABUTMENT SEISMIC CONDITION GLOBAL STABILITY ANALYSIS

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FIG. C2

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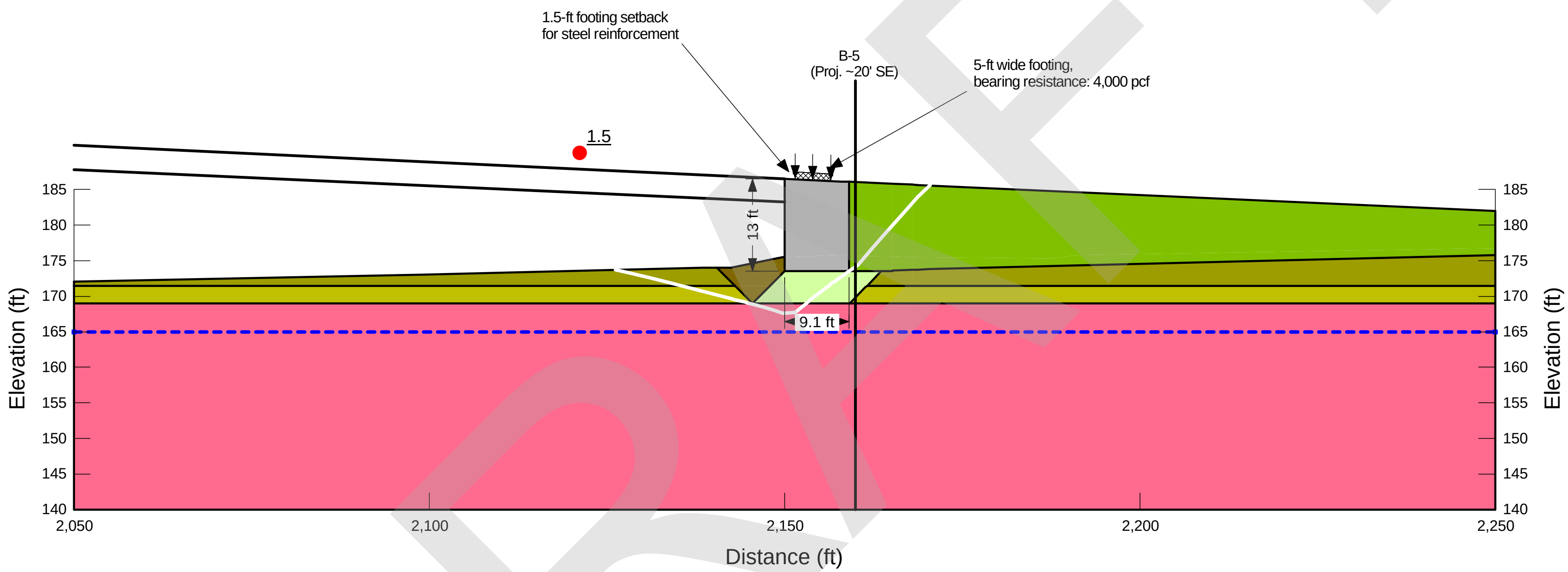
\\PDX-FS1\Vol1\EF\PDX\103000s\103953 Wilsonville I-5\Analysis\03 - MSE Walls\rev2\I-5 Ped MSE abutments.gsz



Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	Borrow Material	125	0	32
	Existing Fill (Stiff CH)	110	0	26
	Existing Fill (Stiff ML)	110	0	31
	MFD-Coarse (GM)	125	0	36
	MSE Abutment Wall	130		
	MSE Fill	130	0	34
	Stone Embankment Material	125	0	36

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

I-5 Pedestrian Bridge
Barbur St. to Wilsonville Town Center
Wilsonville, Oregon

EAST MSE ABUTMENT
STATIC CONDITION
GLOBAL STABILITY ANALYSIS

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FIG. C3

\\PDX-FS1\Vol1\EF\PDX103000s\103953 Wilsonville I-5\Analysis\03 - MSE Walls\rev2\I-5 Ped MSE abutments.gsz

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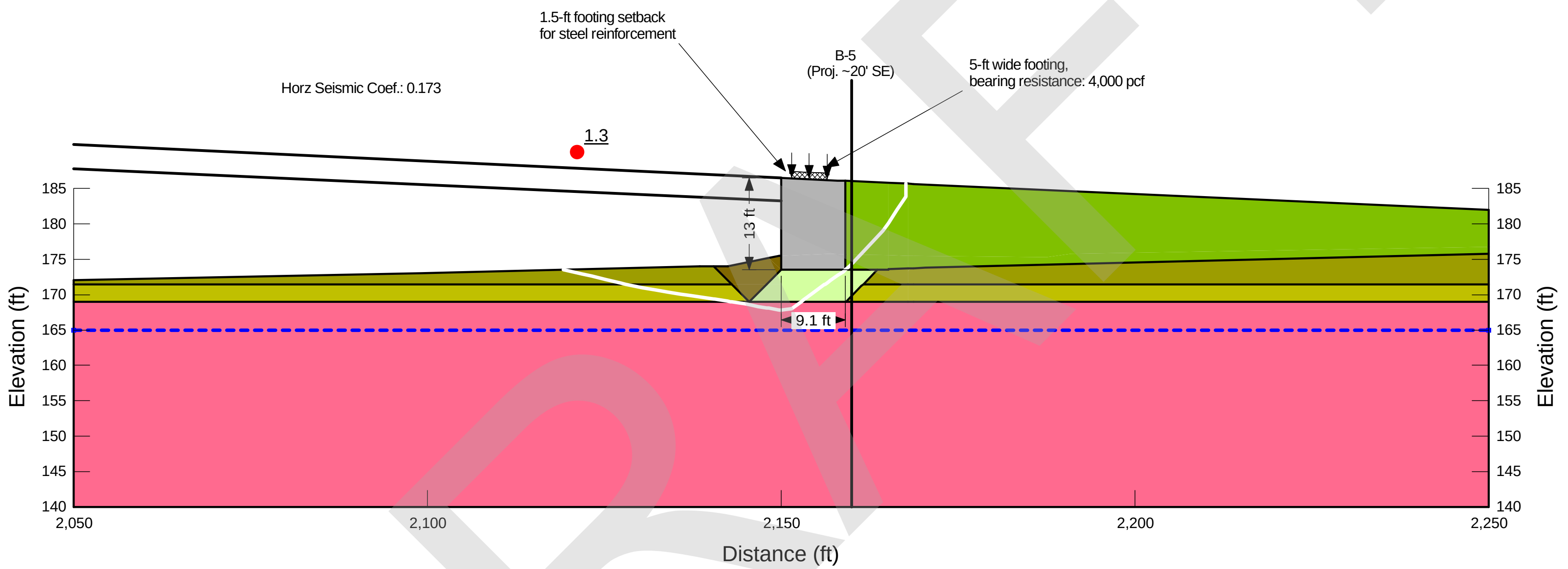

Color	Name	Unit Weight (pcf)	Effective Cohesion (psf)	Effective Friction Angle (°)
	Borrow Material	125	0	32
	Existing Fill (Stiff CH)	110	0	26
	Existing Fill (Stiff ML)	110	0	31
	MFD-Coarse (GM)	125	0	36
	MSE Abutment Wall	130		
	MSE Fill	130	0	34
	Stone Embankment Material	125	0	36

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

I-5 Pedestrian Bridge
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EAST MSE ABUTMENT
SEISMIC CONDITION
GLOBAL STABILITY ANALYSIS

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FIG. C4

\\PDX-FS1\Vol1\EF\PDX103000s\103953 Wilsonville I-5\Analysis\03 - MSE Walls\rev2\I-5 Ped MSE abutments.gsz

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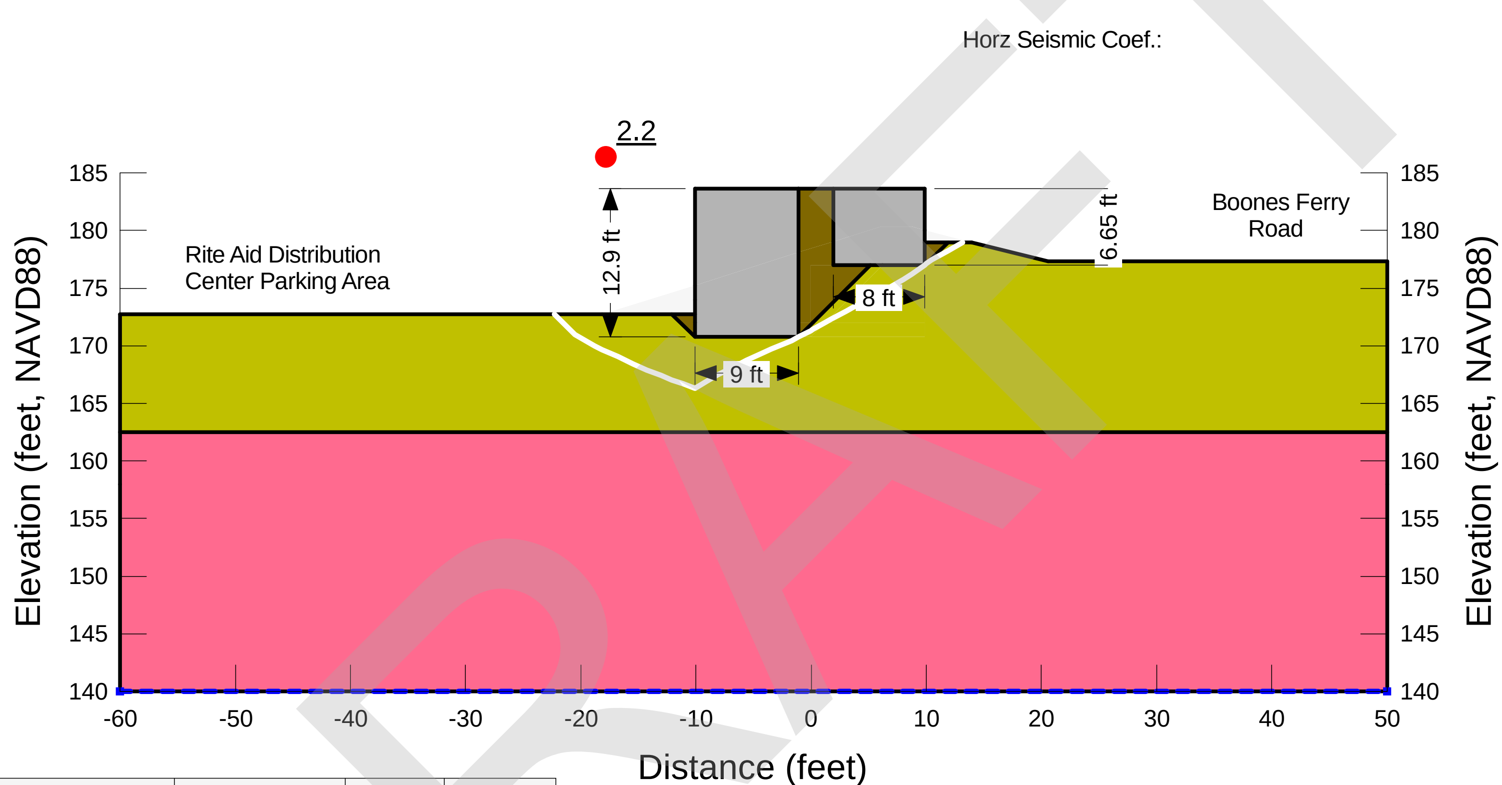

Color	Name	Model	Unit Weight (pcf)	Effective Friction Angle (°)
	Borrow Material	Mohr-Coulomb	125	32
	Existing Fill (V. Stiff ML)	Mohr-Coulomb	120	33
	MFD-Coarse (GM)	Mohr-Coulomb	125	36
	MSE Abutment Wall	High Strength	130	

NOTES

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

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Barbur St. to Wilsonville Town Center
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WEST MSE WALL
STATIC CONDITION
GLOBAL STABILITY ANALYSIS

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FIG. C5

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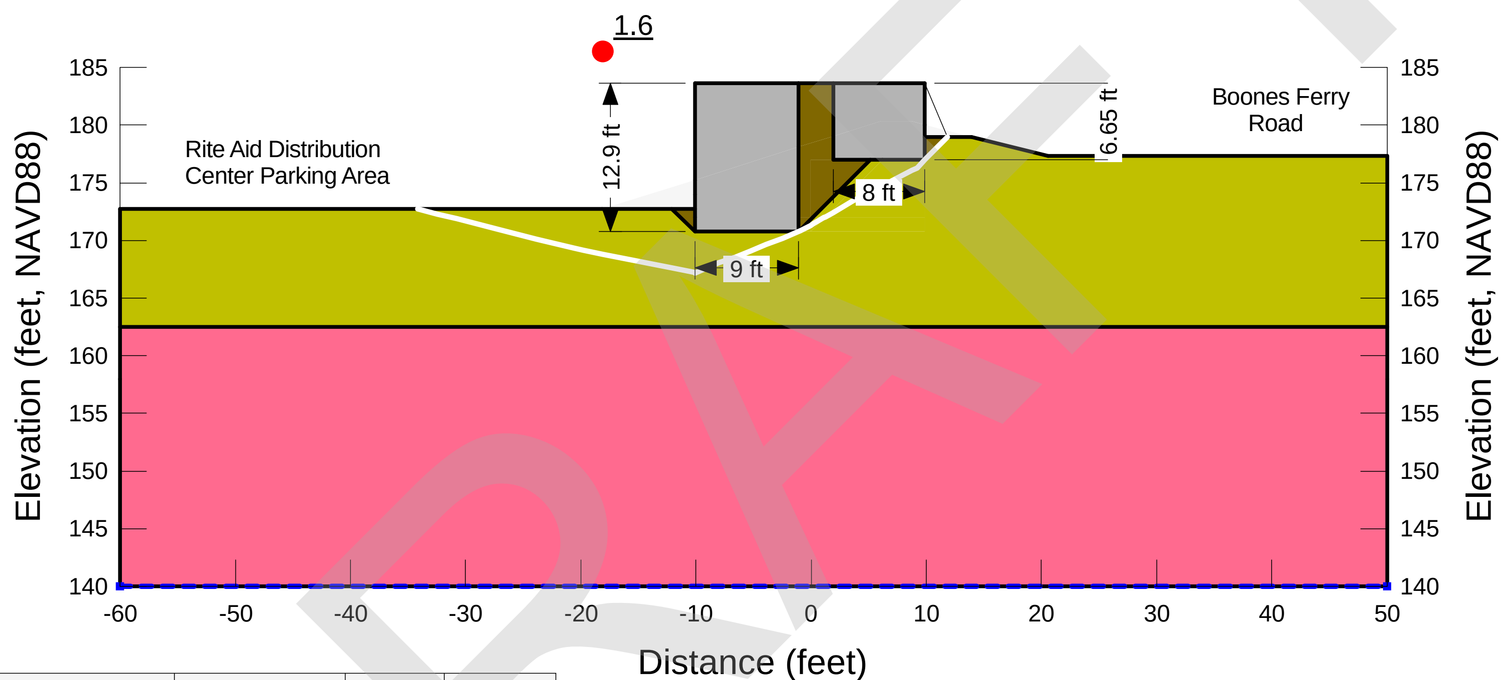

Color	Name	Model	Unit Weight (pcf)	Effective Friction Angle (°)
	Borrow Material	Mohr-Coulomb	125	32
	Existing Fill (V. Stiff ML)	Mohr-Coulomb	120	33
	MFD-Coarse (GM)	Mohr-Coulomb	125	36
	MSE Abutment Wall	High Strength	130	

NOTES

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

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Barbur St. to Wilsonville Town Center
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WEST MSE WALL
SEISMIC CONDITION
GLOBAL STABILITY ANALYSIS

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FIG. C6

I:\EF\PDX\103000s\103953 Wilsonville I-5\Analysis\03 - MSE Walls\rev2\I-5 Ped MSE abutments.gsz

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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

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IMPORTANT INFORMATION

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 ASFE/Association of Engineering Firms Practicing in the Geosciences, Silver Spring, Maryland

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