April 14, 2016

Mr. Chuck Eaton
Engineering Director
City of Milwaukie
6101 SE Johnson Creek Blvd.
Milwaukie, OR 97206

Re: **Riverfront Park Bridge Foundation Design**
11211 SE McLoughlin Blvd.
Milwaukie, Oregon
154-038-004

Dear Mr. Eaton:

Hart Crowser is pleased to present our geotechnical engineering recommendations for the Riverfront Park Bridge replacement in Milwaukie, Oregon. Hart Crowser was contracted by the City of Milwaukie (City) to evaluate mitigation alternatives for a bank failure at the bridge that occurred during recent storms. Based on the information gathered and the compromised condition of the existing bridge, we understand that the City has determined that the most cost effective solution is to replace the existing bridge. The bridge will be constructed using design-build construction techniques. The City has retained Hart Crowser to aid in preparation of preliminary foundation support recommendations for the new bridge. Our recommendations have been prepared in general accordance with our contract with the City (#C2106-011) dated March 7, 2016.

Project Description and Understanding

The existing bridge spans the outfall of Kellogg Creek into the Willamette River and connects the boat ramp and park area at Riverfront Park to the park's southern parking lot. It also provides access to the City's wastewater treatment plant. The southern parking lot was recently constructed, and the Willamette River banks (within the construction area from the bridge abutment south) were regraded and landscaped as part of that project.

The bridge is a single-span structure that was built prior to 1957 by a logging company. Based on a 2014 scour study provided by the City, we understand that adverse scour has occurred at the bridge since construction. Both riprap and concrete surface protection have been used to address scour and erosion; however, both have failed. The resulting continued scour and erosion have caused undermining of bridge footings and pile caps and exposed the pile tops. A preliminary hydraulics analysis indicated that the 500-year scour of 25 feet may compromise the bridge structure.

A bank failure was reported on December 8, 2015, at the southwestern corner of the bridge abutment. Due to continued heavy rains and resulting high flows, the failure continued to lose material into the Willamette River and, as of December 11, 2015, a section of the roadway was severely undermined. At the City's request, we completed an evaluation of options to stabilize the site and mitigate the ongoing erosion. Our evaluation was summarized in a letter report dated January 27, 2016. Based on our evaluation, the City has determined that the most cost effective option is to replace the bridge. This report summarizes our preliminary foundation recommendations for the new structure.

Site Conditions

Geologic Mapping

The subsurface conditions at the site are mapped as Qal – Quaternary Alluvium. Qal has been described as Holocene-age, "River and stream deposits of silt, sand and gravel…largely confined to the Willamette River channel and valley bottom of tributary streams; may include local lacustrine, paludal and eolian deposits." and Pleistocene catastrophic flood deposits consisting of, "boulders, gravel, sandy gravel, and sand consisting of Columbia River Basalt clasts (Beeson, Tolan, and Madin 1989). Underlying the alluvium and flood deposits soils are expected to be Twh – Basalt of Waverly Heights and undifferentiated sedimentary rocks of Eocene age. Twh has been described as subaerial basaltic lava flows and associated sediments that underlie the Columbia River Basalt Group in the region. Twh is typically deeply weathered in the upper 30 feet except where scoured away by flood deposits or river downcutting (Beeson, Tolan, and Madin 1989). Local well logs estimate the top of the Twh at approximately 50 to 80 feet below ground surface (bgs) (OWRD 2015). As noted in the following sections, the geologic mapping correlates well with soils encountered below anthropogenic fill at the project site.

Subsurface Conditions

We explored the subsurface conditions at the site be drilling two borings to depths between 47.5 and 50 feet below the existing road surface. Based on our borings, we interpret the site stratigraphy to consist of the following units (listed from shallowest to deepest): pavement section, fill, alluvium, residual soil, and basalt of Waverly Heights. These units are briefly described separately below.

Pavement Section

Boring B-1 encountered 6 inches of asphaltic concrete (AC) pavement over approximately 6 inches of base aggregate. Boring B-2 encountered 4 inches of AC pavement over approximately 26 inches of base aggregate. The aggregate consisted primarily of 1-inch minus, angular gravel and appeared dense and well-compacted.

Fill

We encountered material that we interpret as fill from below the base course aggregate and extending to a depth of approximately 4.5 to 9.5 feet bgs in boring B-1 and B-2, respectively. The fill in boring B-1 consists of gray, wet, subangular to angular gravel with sand. The fill in boring B-2 consists of sandy silt to silt with sand with scattered organics. Fill consisting of subrounded to rounded coarse gravel to small cobble-size material was observed beneath the south approach to the bridge during our surface reconnaissance where exposed by the bank failure. These differences suggest that the fill is relatively variable.

A Standard Penetration Test (SPT) blow count ("N-value") measured in the fill in boring B-1 yielded 28 blows per foot (bpf) and in boring B-2 N-values ranged from 13 to 29 bpf for three samples, indicating a medium dense relative density and a stiff to very stiff relative consistency in the case of the silt fill. Moisture contents in the fill range from 21 to 31 percent. Based on Atterberg limits testing of the silt fill encountered in boring B-2, the plastic limit is 20 percent, the liquid limit is 41 percent, and the plasticity index is 21.

Alluvium

Underlying the fill we encountered silty sandy soils, which we interpret to be recent Willamette River alluvium, to approximately 15.0 to 20.0 feet bgs in boring B-1 and B-2, respectively. The alluvium consists of red-brown to brown, moist to wet silty sand to sand with silt with trace gravel and brown silty sand to sandy silt. A thin lens of alluvial wet, red-brown, silt with fine sand and trace gravel was encountered in boring B-1 between the depths of 4 and 6 feet bgs.

N-values in the alluvium ranged from 4 to 11 bpf, indicating a very loose to loose relative density and a medium stiff relative consistency in the case of the silt alluvium. Moisture contents in the alluvium range from 20 to 34 percent. Grain size analysis indicates the fines content of the silty sand to sandy silt alluvium ranges from approximately 47 to 51 percent. Grain size analysis indicates the fines content of the silt alluvium is approximately 73 percent.

Residual Soil

Underlying the alluvium we encountered materials, which we interpret as residual soil, that extend to approximately 28 to 32 feet bgs in borings B-1 and B-2, respectively. The residual soil is derived from the basalt of Tertiary Waverly Heights Formation (Twh). The residual soil consists of moist to wet, brown, gray, and gray-brown sandy lean clay, silty sand, and sand with silt. The residual soil typically displayed relict rock texture with occasional pieces of weak intact basalt.

N-values in the residual soil ranged from approximately 5 to 100 bpf, indicating a medium dense to very dense relative density in the sand and medium stiff relative consistency in the silt. Moisture contents in the residual soil ranged from 19 to 40 percent. Based on Atterberg limits testing of the residual soil encountered in both borings, the plastic limit ranges from 21 to 41 percent, the liquid limit ranges from 42 to 47 percent, and the plasticity index ranges from 6 to 21.

Basalt of Waverly Heights (Twh)

Basalt was encountered beneath the residual soil extending to the maximum depth explored, which we interpret as Twh. The Twh was slightly weathered to predominantly decomposed, soft to medium hard, closely fractured basalt with some vesicles. The rock samples recovered from boring B-1 were highly fractured (rubble) with generally poor recovery (10 to 60 percent) and Rock Quality Designation (RQD) values of zero. The rock samples recovered from boring B-2 were relatively intact with recoveries ranging from 80 to 97 percent and RQD ranging from 0 to 62, with increasing RQD with depth. Interbedded zones of decomposed residual soil consisting of very dense sandy clay and silty sand with gravel were encountered in the basalt ranging from approximately 1/4 inch to 2 feet thick.

Groundwater

Our borings were advanced using mud rotary and wire-line rock coring drilling techniques, which do not allow direct measurement of groundwater. Wet samples were encountered in the fill and alluvium at a depth of approximately 2.5 feet bgs, which suggests that shallow infiltrating water may perch on top of the dense residual soil at least during the wet season. Although the perched water can be intermittently shallow, we anticipate that static groundwater will closely follow the level of the water surface in adjacent Kellogg Creek. Based on information from the City, the Ordinary High Water level (OHWL) in the creek is approximately elevation 19 feet.

Geologic Hazard Evaluation

Hazard Mapping

Seismic hazards at the site are mapped in the METRO/DOGAMI IMS-1, *Relative Earthquake Hazard Map for the Portland Metro Region, Clackamas, Multnomah, and Washington Counties, Oregon* (Mabey and others 1997). This publication includes maps for overall seismic hazard, liquefaction, ground amplification, and seismically induced landsliding.

Mabey et. al. (1997) maps the project site as within the overall hazard Zone B, defined as “moderate to high” where relative overall earthquake hazard is mapped between Zones A (highest hazard) and D (lowest hazard).

The ground motion amplification hazard at the site is mapped as Category 3 (“high”) the highest on a four category scale from 3 (“high”) to “no hazard.” Seismically induced landsliding is mapped as no hazard, lowest of a four category scale from “greatest hazard” to “no hazard.” The hazard to the site from soil liquefaction is also shown as 1 (“low”), on the same scale. The results of our subsurface explorations are generally consistent with the published mapping, except we conclude that a higher liquefaction potential exists at the site than indicated by the mapping, as noted below.

The hazard mapping in DOGAMI Oregon HazVu, the online geologic hazards viewer, shows no significant landslide hazards present in the project vicinity.

A review of nearby earthquake faults found that the closest local fault mapped by the U.S. Geological Survey (USGS) (Personius 2002) is the northwest striking Portland Hills Fault, which is mapped passing 3/4 mile northeast of the site. Sense of displacement on the Portland Hills fault is poorly known and controversial. No fault scarps on surficial Quaternary deposits have been described along the fault trace; however, some geomorphic (steep, linear escarpment, triangular facets, oversteepened, and knickpointed tributaries) and geophysical (aeromagnetic, seismic reflection, and ground-penetrating radar) evidence suggest Quaternary displacement. No studies have identified Holocene disturbance, and no surficial expression of the Portland Hills Fault has been identified in latest Pleistocene or Holocene deposits.

The northwest striking Oatfield fault is mapped passing approximately 1 mile to the southwest of the site. The fault has also been modeled as an east-dipping reverse fault. Reverse displacement with a right lateral strike-slip component is consistent with the tectonic setting, mapped geologic relations, and microseismicity in the area. No fault scarps on surficial deposits have been described; however, exposures in a light-rail tunnel showing offset of approximately 1 Ma Boring Lava across the fault indicate Quaternary displacement. No studies have identified Holocene disturbance, and no surficial expression of the Oatfield Fault has been identified in latest Pleistocene or Holocene deposits.

Based on the distance of the site to mapped faults, surface rupture is unlikely to be a hazard at the site.

Seismic Hazard Level and Magnitude of Shaking

General

The site is in a seismically active area. In this section, we describe the seismic setting of the project site, identify the seismic basis of design, provide a code-based design response spectra, and discuss the seismic hazards at the site.

The seismicity of western Oregon is controlled by the Cascadia Subduction Zone. Plate tectonics cause the oceanic Juan de Fuca Plate to subduct beneath the continental North American Plate. Three types of earthquakes are associated with subduction zones: intraslab, interface, and crustal earthquakes. Contributions from each of these sources to the total site seismic hazard were evaluated using the USGS 2008 Interactive Deaggregations (USGS 2013).

Intraslab and Interface Sources

Subduction zones are characterized by the interaction of the oceanic Juan de Fuca plate and continental North American plates. As the oceanic plate subducts beneath the continental plate, the two plates lock together. As the plates move together, stresses similar to a spring build in the overlying continental plate. This stress acts to unlock the two plates. When the magnitude of the *spring* stresses become large enough to overcome the stresses locking the plates together, the plates will suddenly rupture causing an interface earthquake. Interface earthquakes (such as the 2011 M9.0 Tohoku earthquake in northern Japan) are typically the largest magnitude earthquakes on record.

Intraslab earthquakes originate from a deeper zone of seismicity that is associated with bending and fracturing of the subducting Juan de Fuca plate below the continental North American plate. Intraslab earthquakes (such as the 2001 M7.0 Nisqually earthquake in west central Washington) occur at depths of 40 to 70 kilometers (km) and can produce earthquakes with magnitudes up to and greater than M7.0. Our review of the interactive deaggregations indicate interface and intraslab earthquakes contribute over 50 percent of the total seismic hazard to the site.

Crustal Sources

Shallow crustal faults are caused by cracking of the continental crust resulting from the stress that builds as the subducting plates remain locked together. Crustal earthquakes (such as the 1989 Loma Prieta earthquake in the San Francisco Bay Area) occur at relatively shallow depths and can produce earthquakes with magnitudes up to M7.0. Based on our review of the deaggregations, we note the Portland Hills Fault, Grant Butte Fault, and Bolton Fault all lie within 11 km of the site and contribute significantly to the site seismic hazard.

Site Specific

We evaluated potential seismic shaking at the site in accordance with 2014 American Association of State Highway and Transportation Officials (AASHTO) LRFD Bridge Design Specifications (AASHTO BDS) (AASHTO 2014) and 2014 Oregon Department of Transportation (ODOT) Geotechnical Design Manual (ODOT GDM) (ODOT 2014). The AASHTO BDS and ODOT GDM consider the design earthquake to be seismic shaking having a 7 percent probability of exceedance in 75 years (approximately 1,000-year return period).

We evaluated potential seismic shaking at the site using data obtained from the U.S. Seismic Design Maps (USGS 2014). The expected peak bedrock acceleration having a 7 percent probability of exceedance in 75 years is 0.27g. This value represents the peak acceleration on bedrock beneath the site and does not account for ground motion amplification due to site-specific effects. The peak ground acceleration (PGA) is determined by applying a Site Class factor to the peak bedrock acceleration, which will be discussed below. Based on the deaggregation of the site seismic hazard, the modal magnitude and distance for this site and hazard level are M9.0 and 93 km (hypocentral distance), respectively.

Seismic Site Class

Thick sequences of unconsolidated, soft sediments typically amplify the shaking of long-period ground motions, such as those associated with subduction zone earthquakes; whereas, areas underlain by shallow soil profiles are not likely to amplify seismic waves.

The “Site Class” is a designation used by the AASHTO BDS to quantify ground motion amplification. The classification is based on the stiffness in the near surface soil above the bedrock materials at a site. At the Riverfront Park Bridge, the upper 30 feet of subsurface stratigraphy is characterized by sequences of alluvial soils and residual bedrock, which is underlain by intact bedrock. We evaluated the site class by determining the average blow count of the soils in the upper 30 feet. This information, without regard for liquefaction potential (see below), leads us to classify the site as Site Class E.

Our analyses have identified a liquefaction hazard present underlying the north approach to the bridge. The ODOT GDM indicates that where a liquefaction hazard is identified, a site-specific ground response analysis should be considered to determine the response spectrum for design. However, we note that in the ODOT GDM for structures with fundamental periods less than 1.0 second, a design response spectrum used without regard to the presence of a liquefaction hazard will likely be more conservative than a response spectrum derived from a site-specific ground response analysis. Based on our understanding of the project, we anticipate that the fundamental period of the replacement bridge will be less than 1.0 second. Therefore, we recommend the sites be classified as site class E and the typical AASHTO BDS design response spectrum without regard for liquefaction be used for design.

Design Response Spectrum

We obtained the design parameters for the design spectral acceleration from the U.S. Seismic Design Maps (USGS 2014) at Latitude 45.44196 and Longitude -122.64197. The parameters provided in Table 1 are appropriate for AASHTO BDS code-based seismic design.

Table 1 - Seismic Design Parameters

Parameter	Value
Spectral Response Acceleration (Short Period), S_s	0.639
Spectral Response Acceleration (1-Second Period), S_1	0.222
Peak Ground Acceleration (0-second Period), PGA	0.270
Site Class	E
Site Coefficient, F_a	1.423
Site Coefficient, F_v	3.114
Site Coefficient, F_{pga}	1.352
Spectral Response Acceleration (Short Period), S_{DS}	0.909
Spectral Response Acceleration (1-Second Period), S_{D1}	0.690
PGA Adjusted for Site Amplification, As	0.364

Liquefaction

General

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

We performed site-specific liquefaction potential analysis on the silty sand soils encountered at each approach using procedures outlined in Idriss and Boulanger (2008). In accordance with AASHTO BDS, we completed the liquefaction hazard analysis using the site class adjusted acceleration coefficient (A_S). We used an A_S of 0.36 g and associated earthquake magnitude of 9.0 in our analysis. We assumed hydrostatic groundwater at elevation 19 feet, or OHWL, for the purpose of our liquefaction analyses.

Based on our analysis, liquefaction is likely to occur in the saturated alluvial soils encountered underlying the north approach to the bridge. Liquefaction of sand soils typically means strength loss and post-liquefaction settlement.

Based on the soil stratigraphy, depth to groundwater table, and high relative density of the soils, we anticipate the liquefaction hazard under the south approach to the bridge is low.

Settlement

Post-liquefaction settlement results from densification of liquefiable sandy soils following an earthquake. The permanent ground surface settlement is not typically uniform across the area and can result in significant differential settlement.

Based on our analysis, we estimate up to approximately 4 inches of liquefaction-induced settlement could occur at the north approach of the bridge. This settlement will induce “downdrag” on the proposed shaft foundations at the north abutment.

Earthquake-Induced Landsliding/Lateral Spread

Based on the water table close to the river level and the soil conditions at the site, it is our opinion the potential for large scale earthquake-induced landsliding or lateral spread at the bridge location is medium to low; however, local instabilities in the northern approach slope are possible during a design level event, especially in areas where perched or high groundwater is present above the weathered basalt. For the purpose of this preliminary report, we have not evaluated lateral forces due to slope instability on the bridge foundations. Once the final approach geometry is determined, this should be evaluated by the design build engineer.

Preliminary Foundation Support Recommendations

General

We understand that the replacement bridge will be reconstructed with a longer span so that abutments and foundations are beyond the anticipated scour zone. Further, we understand that the adjacent ODOT bridge and fish ladder cannot be damaged during construction. The existing bridge is 46 feet long and 18 feet wide according to inspection reports provided by the City.

Based on the slope geometry and soil conditions, we anticipate that a 90-foot bridge would be required to avoid scour potential. Abutment walls would also be required to accommodate 2 horizontal to 1 vertical (2H:1V) slopes below the bridge in the native soils. We estimate that the abutment walls would be 10 to 15 feet high, unless reinforced or engineered slopes are constructed that allow for a steeper slope and lower walls while still protecting the new bridge from scour. Further, based on the scour potential and subsurface conditions, we anticipate the bridge will be founded on deep foundations, such as small diameter drilled shafts or drilled in piles that will extend between 35 and 50 feet.

Our preliminary recommendations for design and construction of the shafts is included below. We have estimated the design top of shafts at elevation 30 feet (assumed pile cape of 5 feet) with design groundwater at OHWL (19 feet). Our design is based on the 2014 edition of AASHTO BDS and the ODOT GDM.

Design Soil Profiles for Axial Resistance of Drilled Shafts

The design soil unit profile for each abutment is summarized in Tables 1 and 2.

Table 1 – Design Soil Profiles at South Abutment

Soil Unit	Top Elevation of Soil Unit (feet)
A1 (Loose Sand)	30
R1 (Dense to Very Dense Residual Soil)	22.5
B1 (Basalt of Waverly Heights)	7

Table 2 – Design Soil Profiles at North Abutment

Soil Unit	Top Elevation of Soil Unit (feet)
F1 (Very Stiff Clay Fill)	30
A2 (Loose Sand Alluvium)	24.5
R2 (Medium Stiff Residual Soil)	14
R3 (Medium Dense to Dense Residual Soil)	11
B2 (Basalt of Waverly Heights)	2

Axial Resistance

Our recommended axial shaft resistances for each pier are provided on Figures 3 through 14. We have provided resistances for 2-, 3-, and 4-foot-diameter shafts. We calculated nominal shaft resistance in accordance with AASHTO BDS Chapter 10. Resistance factors used to derive the axial resistance curves provided on Figures 3 through 14 are presented in Table 3. Resistance factors were obtained from AASHTO BDS.

We note that vertical downdrag forces on shafts due to liquefaction will need to be applied to shafts at the north abutment. The magnitudes of the downdrag forces are listed on the appropriate figures. Additionally, due to the very large unconfined compressive strength of the intact basalt bedrock encountered at the site, the nominal shaft axial side resistance is limited to the unconfined compressive strength of the shaft concrete in accordance with the AASHTO BDS Section C10.8.3.5.4a. For the purpose of this design, we assumed a concrete unconfined compressive strength of 4,000 pounds per square inch (psi).

Table 3 – Axial Resistance Factors for Single Drilled Shafts

Limit State	Tip	Side Compression	Side Uplift
Service (All)	1.00	1.00	1.00
Strength (All soils except B1 and B2)	0.50	0.50	0.40
Strength (B1, B2)	0.50	0.55	0.45
Extreme (All)	1.00	1.00	0.80

Drilled Shaft Length Considerations

The drilled shaft tips should be embedded into the intact basalt bedrock to a depth equal to at least two shaft diameters. Additionally, we note that a layer of residual sand was encountered between two layers of intact bedrock in boring B-1 at a depth of 40 feet bgs. We recommend that shaft tips be placed directly on intact bedrock. If interbedded layers of residual soil are encountered at plan tip elevation, additional shaft excavation will be needed to place the shaft toe in intact bedrock. Additional excavation should be a minimum of the shaft diameter.

Axial Resistance Group Effects

For shaft center-to-center spacing greater than 3D (single row) or 4D (multiple rows), where D is the shaft diameter, no axial group reduction is required. If multiple rows of shafts are used with spacing less than 4D, we recommend using the group reduction factors in Table 10.8.3.6.3-1 of the AASHTO BDS. However, for either single or multiple row configurations, we recommend a minimum shaft spacing of 3D.

Lateral Shaft Resistance

We anticipate lateral deflections of shafts will be calculated using LPILE software. For LPILE analysis, we recommend using the LPILE parameters in Tables 4 and 5. AASHTO BDS stipulates using a resistance factor of 1.0 for lateral analysis. For shaft analysis, we recommend a design groundwater table elevation of 19 feet.

For extreme limit state analysis of the north abutment, soil unit A2 should be modeled using a P-multiplier of 0.1 to account for the effects of liquefaction.

Table 4 – Soil and Rock Properties for Lateral Shaft Analysis South Abutment

Soil Unit	LPILE Model	Eff. Unit Weight (pcf) Above gwt	Eff. Unit Weight (pcf) Below gwt	Effective Friction Angle (deg)	PY Modulus, K (pci) Above gwt	PY Modulus, K (pci) Below gwt
A1	Sand (Reese)	115	53	32	25	20
R1	Sand (Reese)	125	63	38	225	125

Rock Unit	LPILE Model	Eff. Unit Weight (pcf) Above gwt	Eff. Unit Weight (pcf) Below gwt	Uniaxial Compressive Strength, qu (psi)	Hoek-Brown, mi	GSI	Intact Rock Modulus (psi)	Rock Mass Modulus (psi)
B1	Massive Rock	150	107	13,000	Basalt - 25	30	1,540,000	0

Notes: pcf = pounds per cubic foot • pci = pounds per cubic inch • gwt = groundwater table

Table 5 – Soil and Rock Properties for Lateral Shaft Analysis North Abutment

Soil Unit	LPILE Model	Eff. Unit Weight (pcf)		Effective Friction Angle (deg)	PY Modulus, K (pci)		Cohesion (psf)	E50
		Above gwt	Below gwt		Above gwt	Below gwt		
F1	Sand (Reese)	120	58	32	25	N/A	N/A	N/A
A2	Sand (Reese)	115	53	32	25	20	N/A	N/A
R2	Soft Clay (Matlock)	115	53	N/A	N/A	N/A	750	0.015
R3	Sand (Reese)	125	63	38	225	125	N/A	N/A
Rock Unit	**LPILE Model**	**Eff. Unit Weight (pcf)**		**Uniaxial Compressive Strength, qu (psi)**	**Hoek-Brown, mi**	**GSI**	**Intact Rock Modulus (psi)**	**Rock Mass Modulus (psi)**
		Above gwt	**Below gwt**					
B2	Massive Rock	169	107	13,000	Basalt - 25	35	1,540,000	0

Notes: psf = pounds per square foot

Lateral Pile Group effects

We recommend analyzing for lateral group effects using Section 10.8.3.6 of AASHTO BDS by applying P-multipliers, P_m, to the soil units in LPILE. P-multipliers for various shaft configurations are displayed in Table 6. As previously noted, we recommend minimum shaft spacing of three shaft diameters (3D).

Table 6 – P-multipliers for Lateral Resistance of Drilled Shafts

Shaft Spacing	P-multiplier		
	Row 1	Row 2	Row 3 or Higher
3D	0.8	0.4	0.3
5D	1.0	0.85	0.7

Intermediate values can be interpolated.

Drilled Shaft Construction

Drilled shafts should be constructed in conformance to ODOT Standard Specification for Construction (OSSC) Section 00512 – Drilled Shafts (ODOT 2015).

Final shaft toe elevations will be located below the river level and groundwater elevation. Large groundwater inflows should be assumed during shaft excavation. We recommend the construction planning assume concrete placement under water in conformance to OSSC Section 00512.47(c) – Wet Shaft Concrete Placement.

Temporary casing should be installed through the alluvial and residual soils that overly the basalt bedrock. The casing should be installed a minimum of 2 feet below the contact between these soils and basalt.

Limitations

We have prepared this report for the exclusive use of the City of Milwaukie and their authorized agents for the proposed Riverfront Park Bridge Replacement project in Milwaukie, Oregon in accordance with our subconsultant agreement dated March 7, 2015. Our report is intended to provide our opinion of geotechnical parameters for design and construction of the proposed project based on exploration locations that are believed to be representative of site conditions. However, conditions can vary significantly between exploration locations and our conclusions should not be construed as a warranty or guarantee of subsurface conditions or future site performance.

Within the limitations of scope, schedule, and budget, our services have been executed in accordance with generally accepted practices in the field of geotechnical engineering in this area at the time this report was prepared. No warranty, express or implied, should be understood.

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

Closing

We trust that this report meets your project needs. If you have questions or if we can be of further assistance, please call.

Sincerely,

HART CROWSER, INC.

REGISTERED PROFESSIONAL ENGINEER
89452PE
OREGON
Nov. 14, 2006
ALLISON MARIE PYRCH
EXPIRES: 12/31/16

ALLISON M. PYRCH, PE, GE
Associate, Geotechnical Engineer

TIMOTHY W. BLACKWOOD, PE, GE, CEG
Principal, Geotechnical Engineer

Attachments:
Figure 1 – Site Location
Figure 2 – Site Plan
Figures 3 through 14 – Axial Resistance for Drilled Shafts
Attachment A – Field Explorations and Laboratory Testing

References

American Association of State Highway and Transportation Officials (AASHTO) 2014. AASHTO LRFD Bridge Design Specifications. Seventh Edition, 2014. American Association of State Highway and Transportation Officials.

Beeson, M.H., T.L Tolan, and I.P. Madin 1989. Geologic Map of the Lake Oswego Quadrangle, Clackamas, Multnomah, and Washington Counties, Oregon: Oregon Department of Geology and Mineral Industries, Geological Map Series GMS-59, scale 1:24,000.

Idriss, I.M. and R.W. Boulanger 2008. *Soil Liquefaction during Earthquakes* by Earthquake Engineering Research Institute MNO-12.

Mabey, M.A., I.P Madin, G. Black, D.B Meier, T.L. Youd, C. Jones, and B. Rice 1997. Relative Earthquake Hazard for the Portland Metro Region, Clackamas, Multnomah, and Washington Counties, Oregon, Oregon Department of Geology and Mineral Industries (DOGAMI) Interpretive Map Series IMS-1, 1 pl., 1:62,500 scale.

Oregon Department of Transportation (ODOT) 2015. Oregon Standard Specifications for Construction, 2015.

Oregon Department of Transportation (ODOT) 2014. Geotechnical Design Manual (GDM), 2014.

Oregon Water Resources Department (OWRD) 2015. Well log query, accessed 12/28 /2015, http://apps.wrd.state.or.us/apps/gw/well_log/.

Personius, S.F., compiler, 2002. Quaternary Fault and Fold Database of the United States, U.S. Geological Survey website: http://earthquakes.usgs.gov/regional/qfaults.

U.S. Geologic Survey (USGS) 2014. U.S. Seismic Design Maps. Accessed September 13, 2015. http://earthquake.usgs.gov/designmaps/us/application.php.

USGS 2013. National Seismic Hazard Mapping Project - Probabilistic Seismic Hazard Assessment Interactive Deaggregation website. https://geohazards.usgs.gov/deaggint/2008/ .

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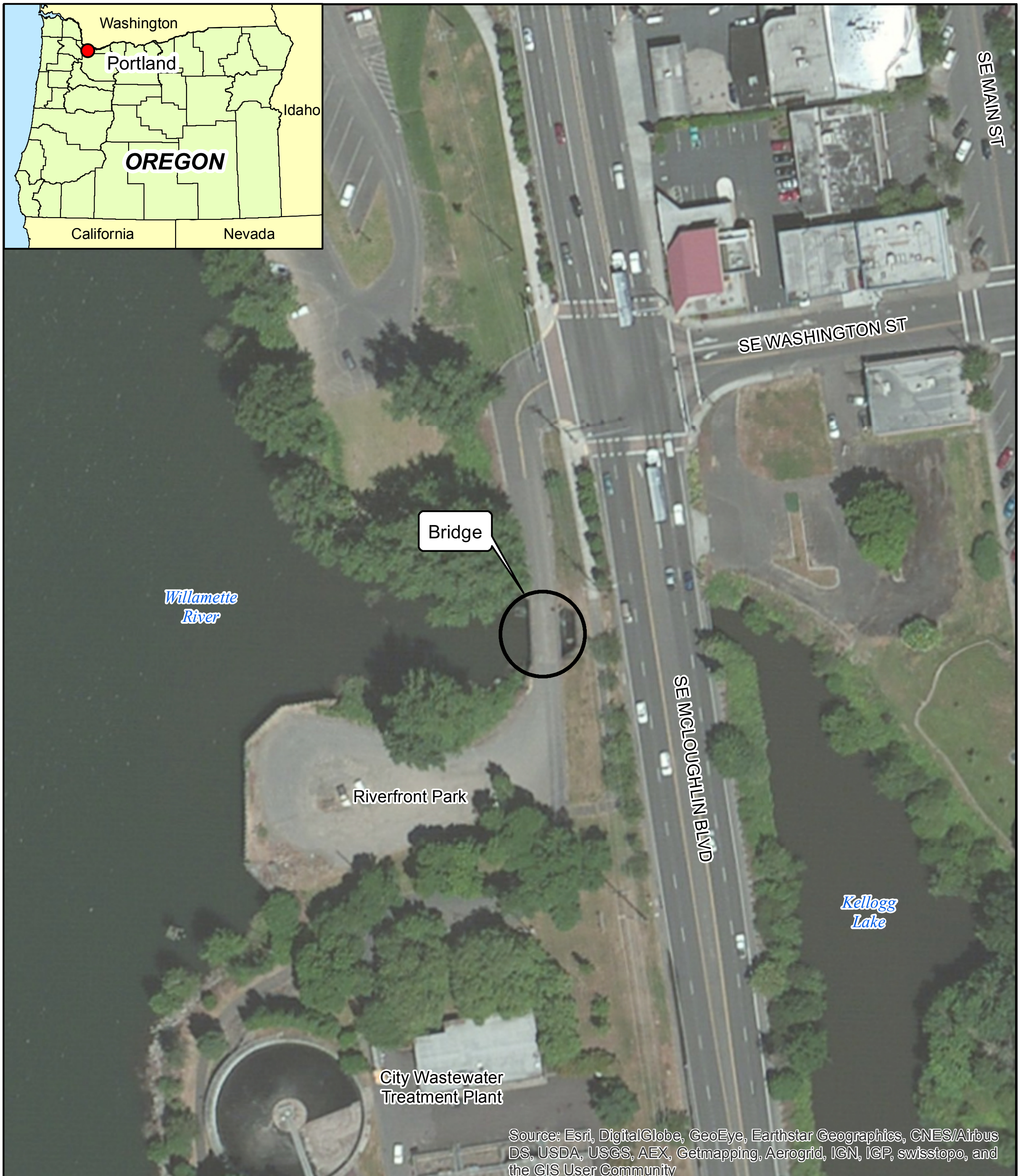

Riverfront Park Bridge Foundation Design
Milwaukie, Oregon

Site Location

154-038-004 4/16

HARTCROWSER

Figure
1

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LEGEND

Boring

0 15 30 60
Feet

N

Note: Feature locations are approximate.

Riverfront Park Bridge Foundation Design
Milwaukie, Oregon

Site Plan

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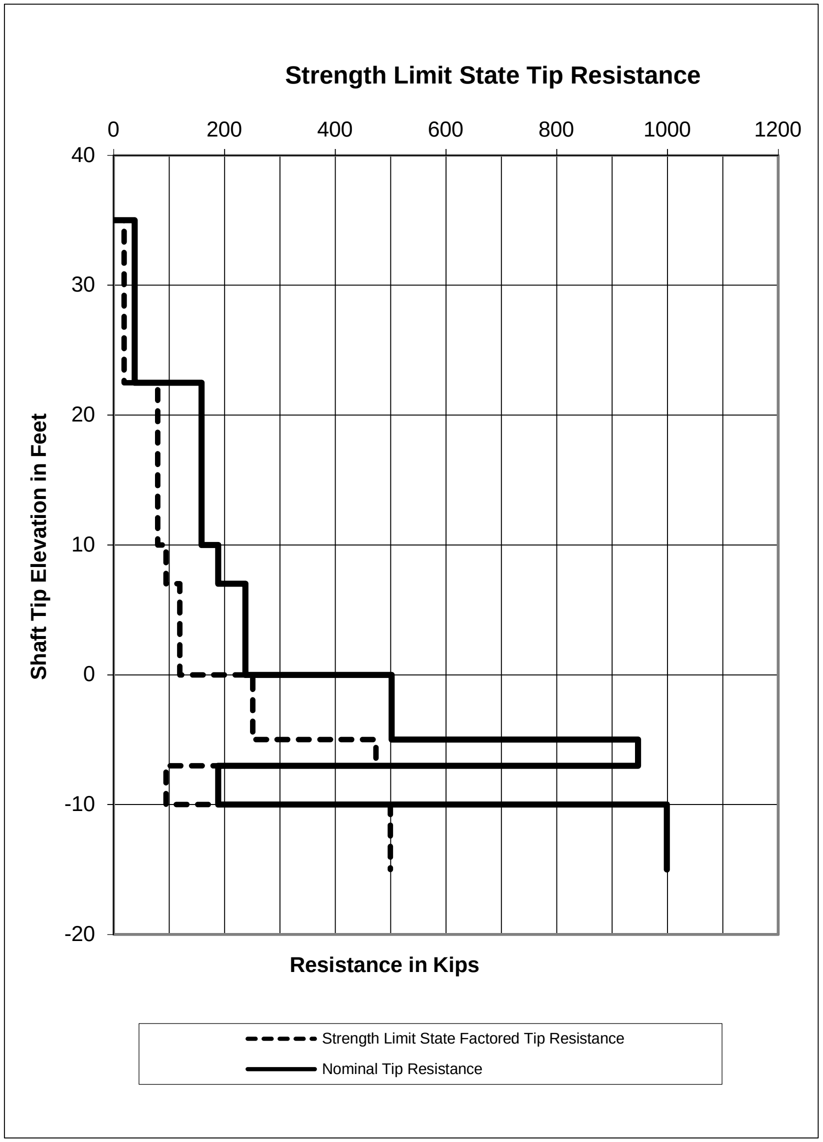
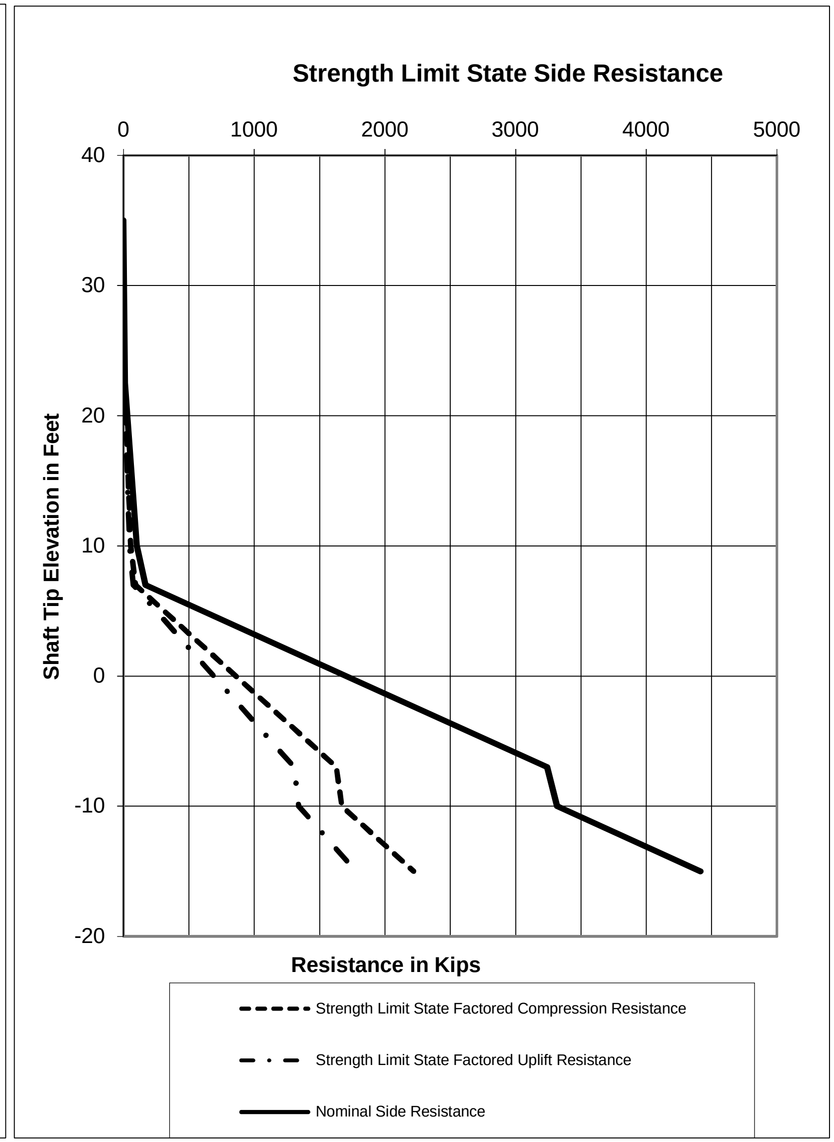
HARTCROWSER

Figure

2

Document Path: F:\Notebooks\154038004_Riverfront Park Bridge Foundation Design\GIS\154038004_SP.mxd Date: 4/13/2016 User Name: melissaschweitzer





Notes:

1) The net weight of the shaft is not included in these charts and should be treated as a load applied to the top of the shaft.

2) Factored resistances are based on resistance factors from 2014 AASHTO LRFD Bridge Design Specifications, Table 10.5.5.2.4-1.

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Riverfront Park Bridge Foundation Design
Milwaukie, Oregon

Axial Resistance Chart for : 2 ft Diameter Drilled Shaft Strength Limit State - South Abutment

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Figure

3

Riverfront Park Bridge Foundation Design
Milwaukie, Oregon

Axial Resistance Chart for : 2 ft Diameter Drilled Shaft Extreme Limit State - South Abutment

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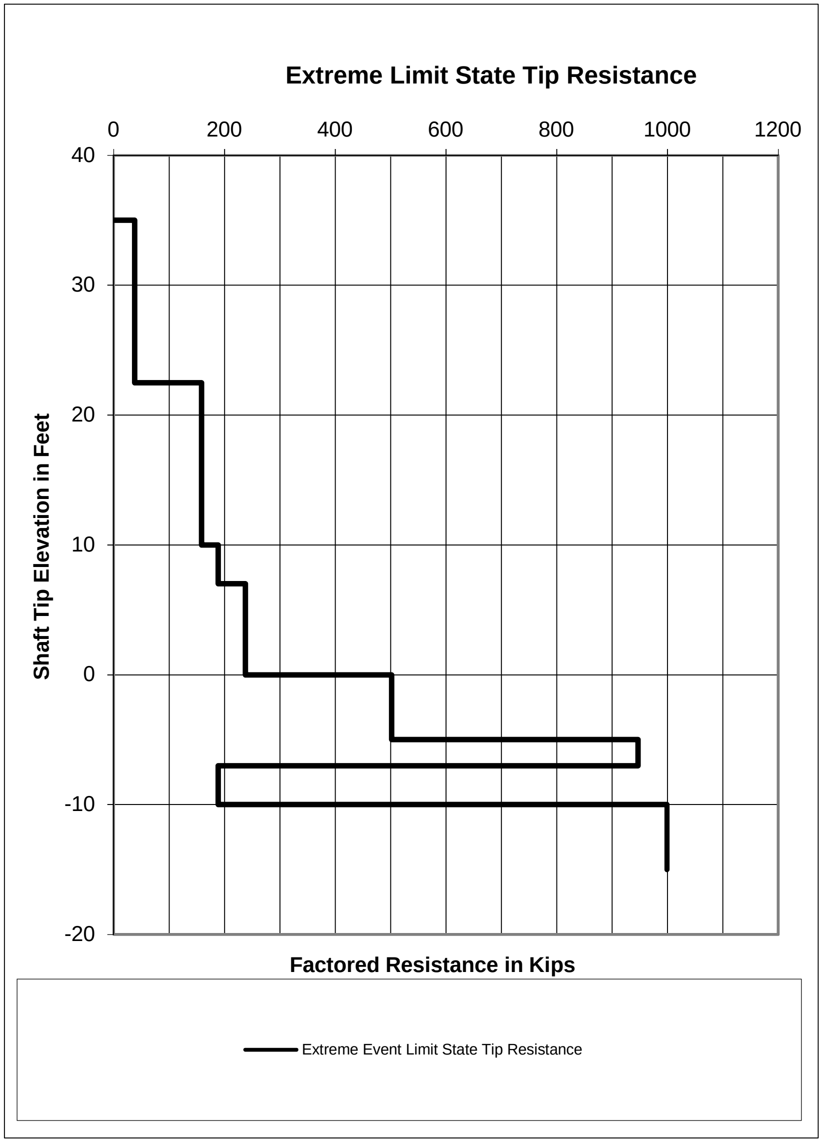
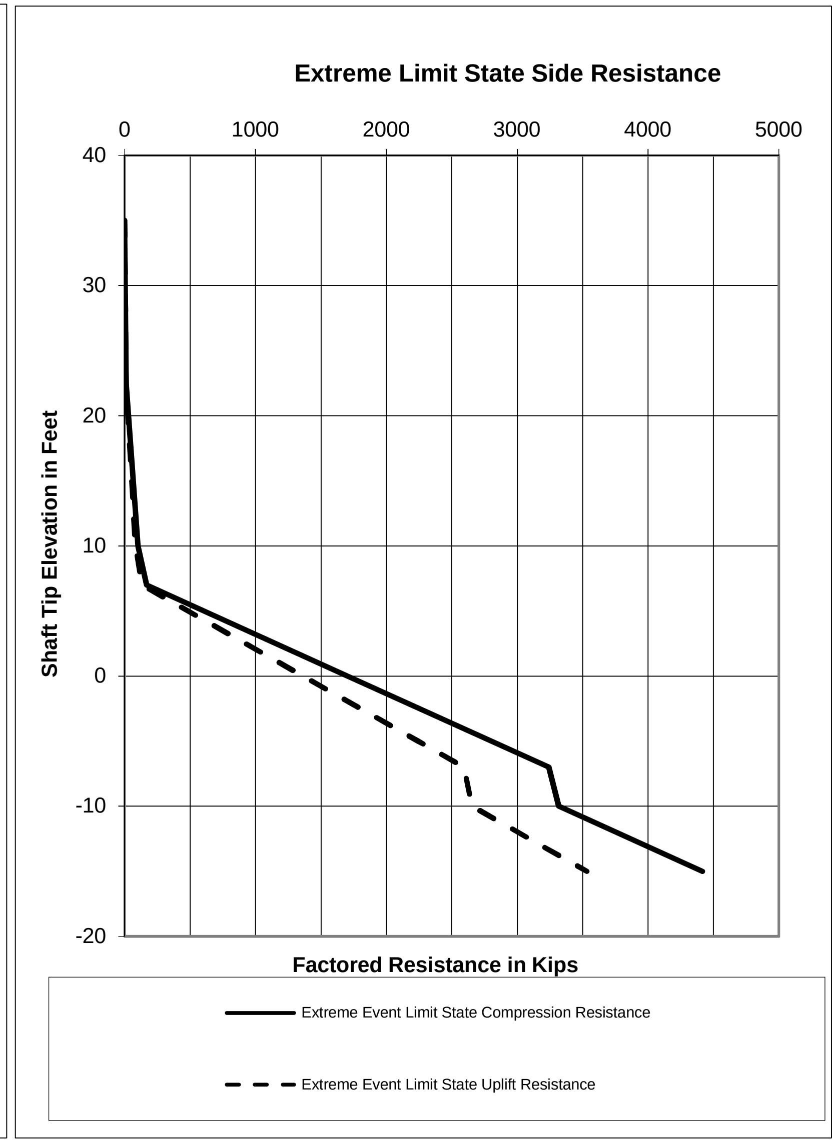
Figure 4

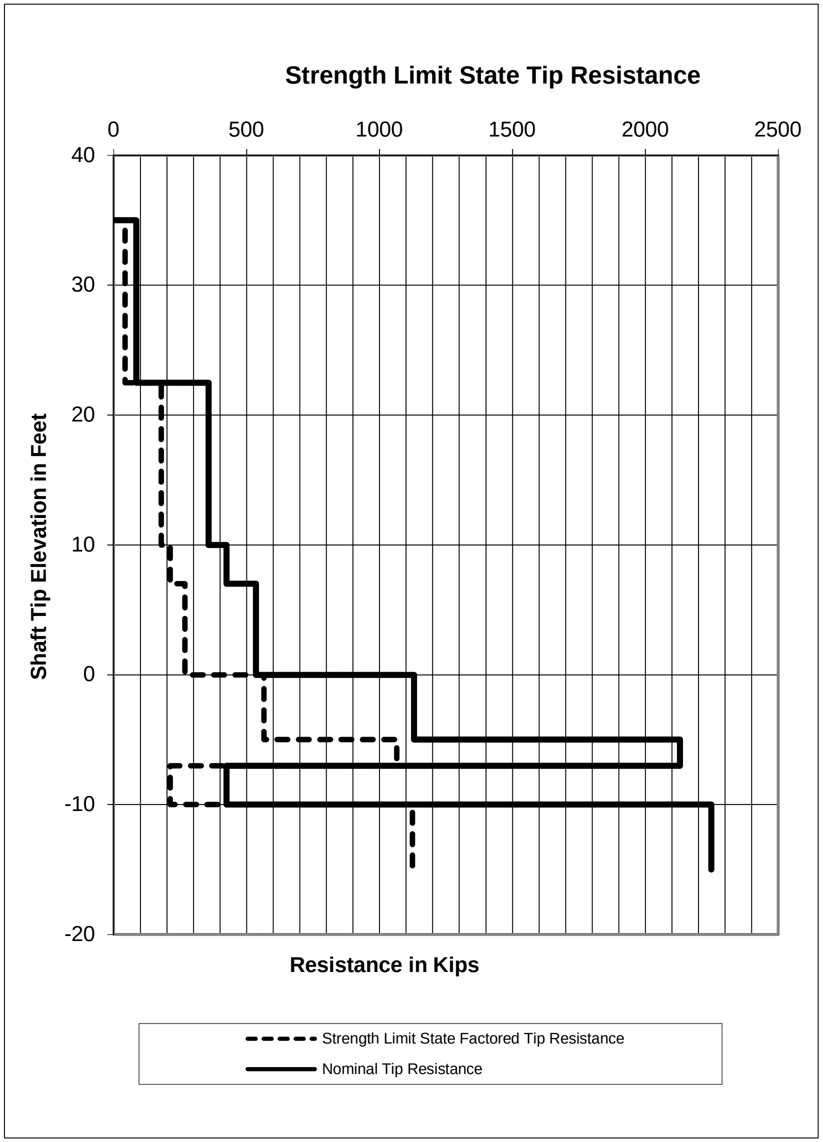


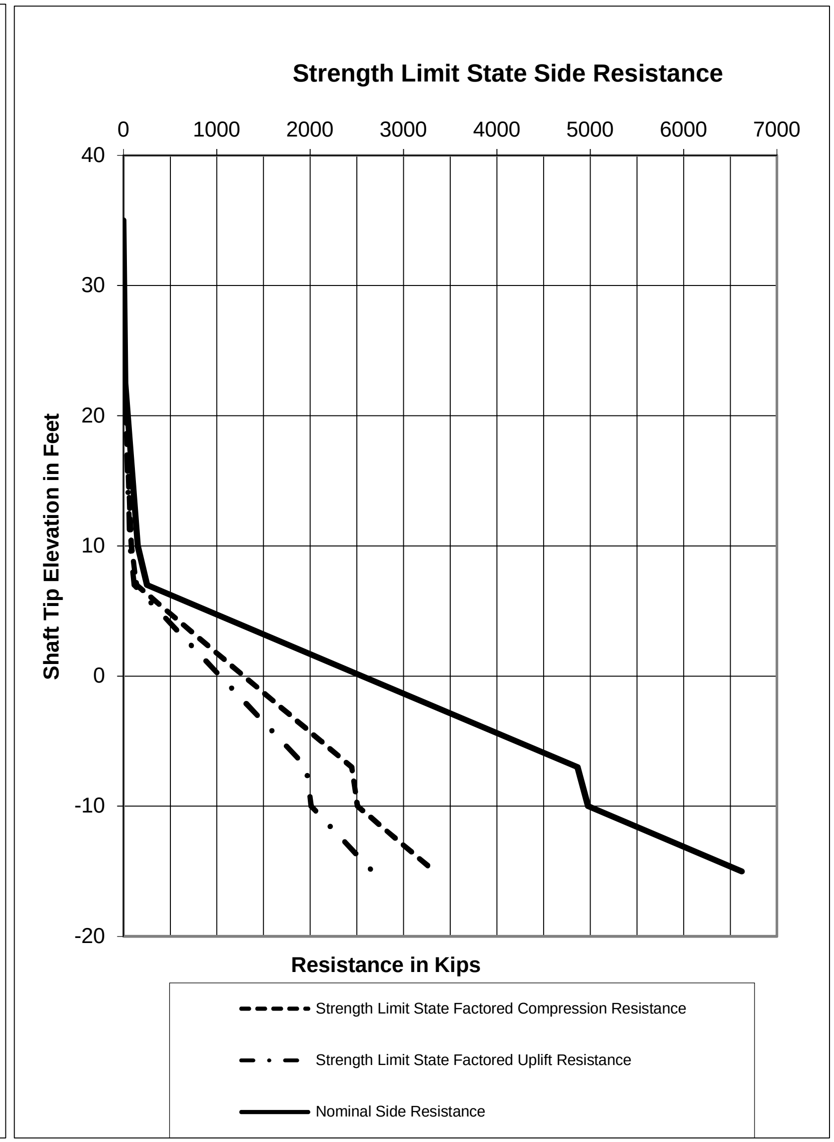


Notes:

1) The net weight of the shaft is not included in these charts and should be treated as a load applied to the top of the shaft.

2) Factored resistances are based on resistance factors from 2014 AASHTO LRFD Bridge Design Specifications, Table 10.5.5.2.4-1.





Notes:

1) The net weight of the shaft is not included in these charts and should be treated as a load applied to the top of the shaft.

2) Factored resistances are based on resistance factors from 2014 AASHTO LRFD Bridge Design Specifications, Table 10.5.5.2.4-1.

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Riverfront Park Bridge Foundation Design
Milwaukie, Oregon

Axial Resistance Chart for : 3 ft Diameter Drilled Shaft Strength Limit State - South Abutment

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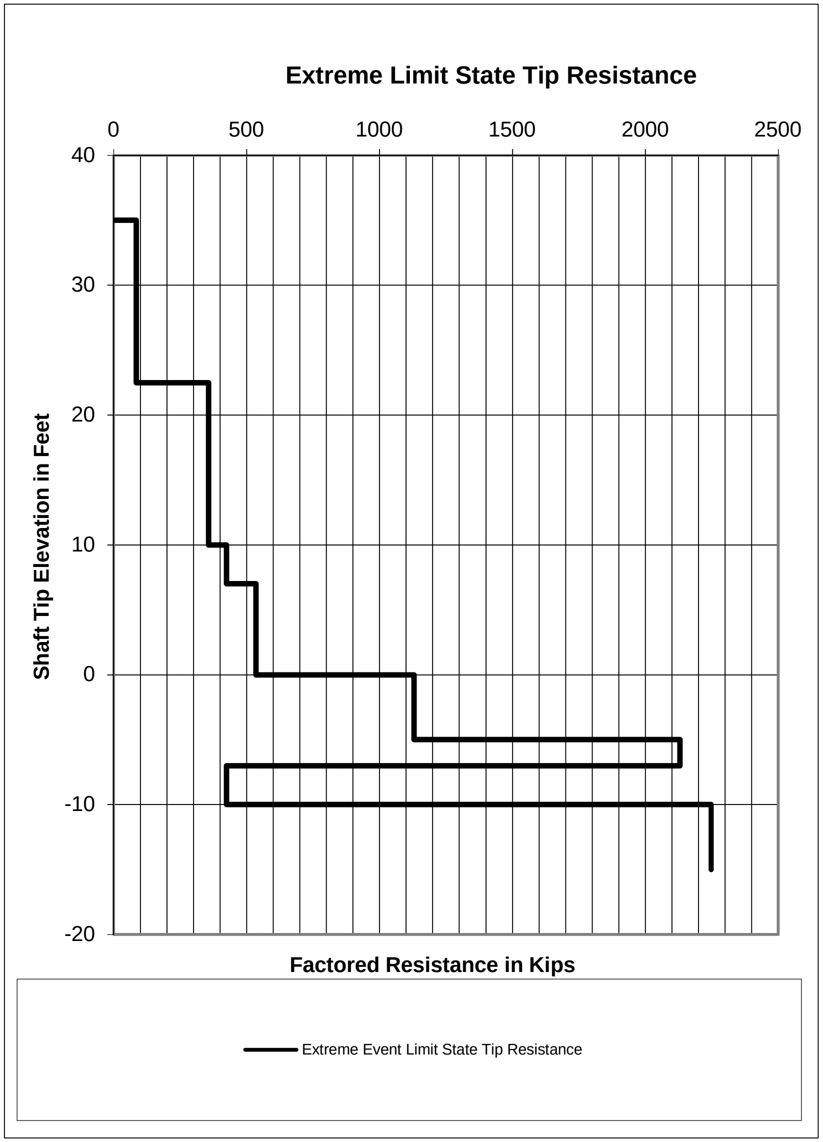
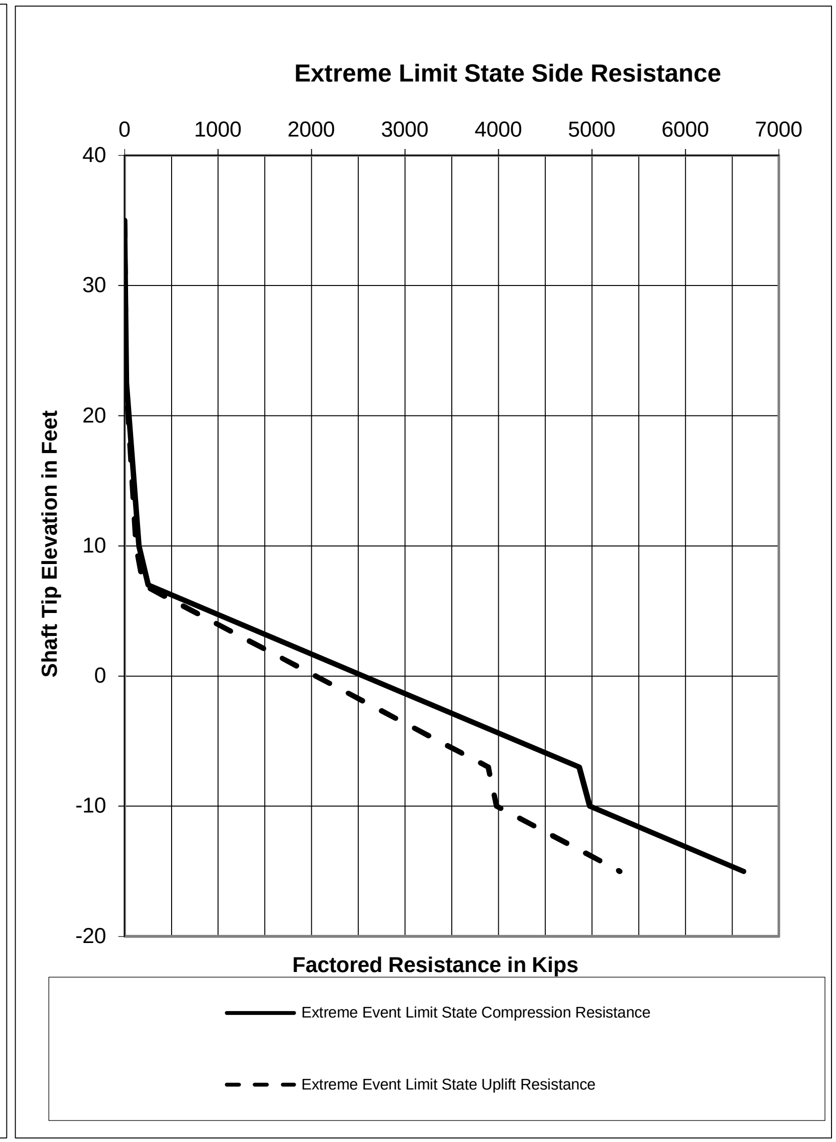
HARTCROWSER

Figure

5





Notes:
1) The net weight of the shaft is not included in these charts and should be treated as a load applied to the top of the shaft.
2) Factored resistances are based on resistance factors from 2014 AASHTO LRFD Bridge Design Specifications, Table 10.5.5.2.4-1.

Riverfront Park Bridge Foundation Design
Milwaukie, Oregon

Axial Resistance Chart for : 3 ft Diameter Drilled Shaft Extreme Limit State - South Abutment

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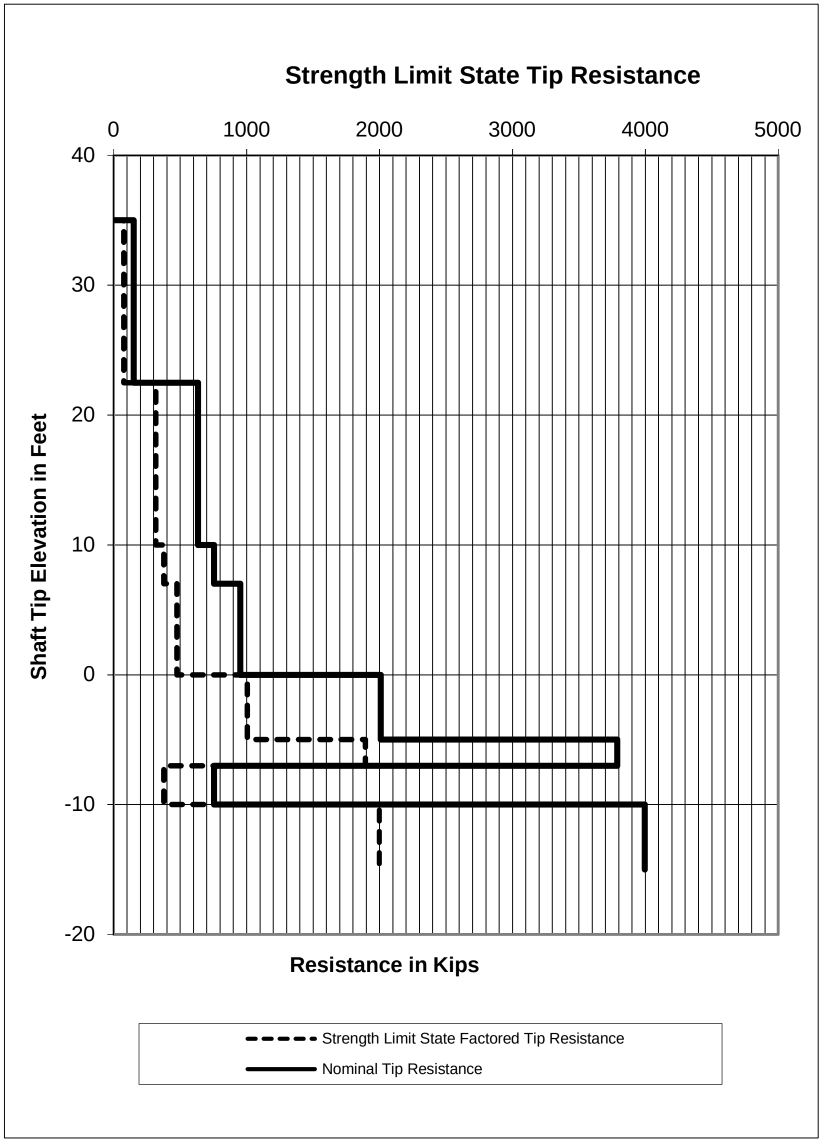
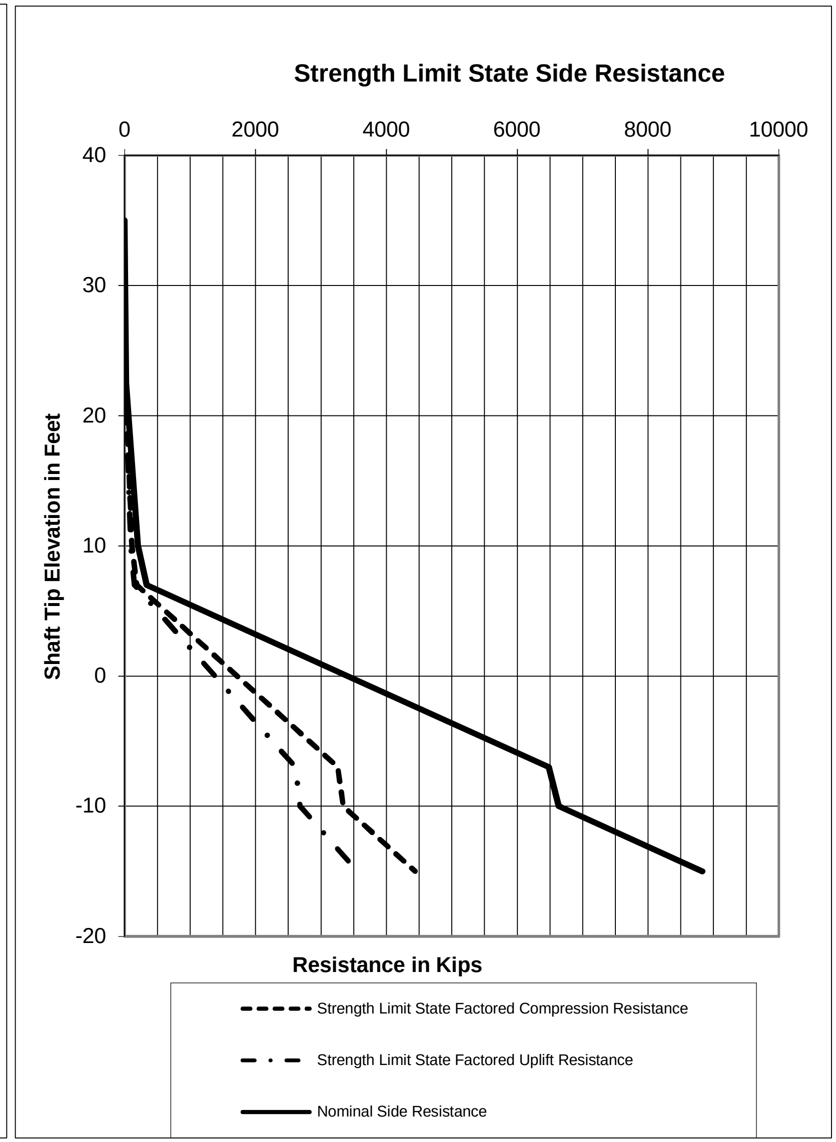
HARTCROWSER

Figure
6





Notes:

1) The net weight of the shaft is not included in these charts and should be treated as a load applied to the top of the shaft.

2) Factored resistances are based on resistance factors from 2014 AASHTO LRFD Bridge Design Specifications, Table 10.5.5.2.4-1.

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Riverfront Park Bridge Foundation Design

Milwaukie, Oregon

Axial Resistance Chart for : 4 ft Diameter Drilled Shaft Strength Limit State - South Abutment

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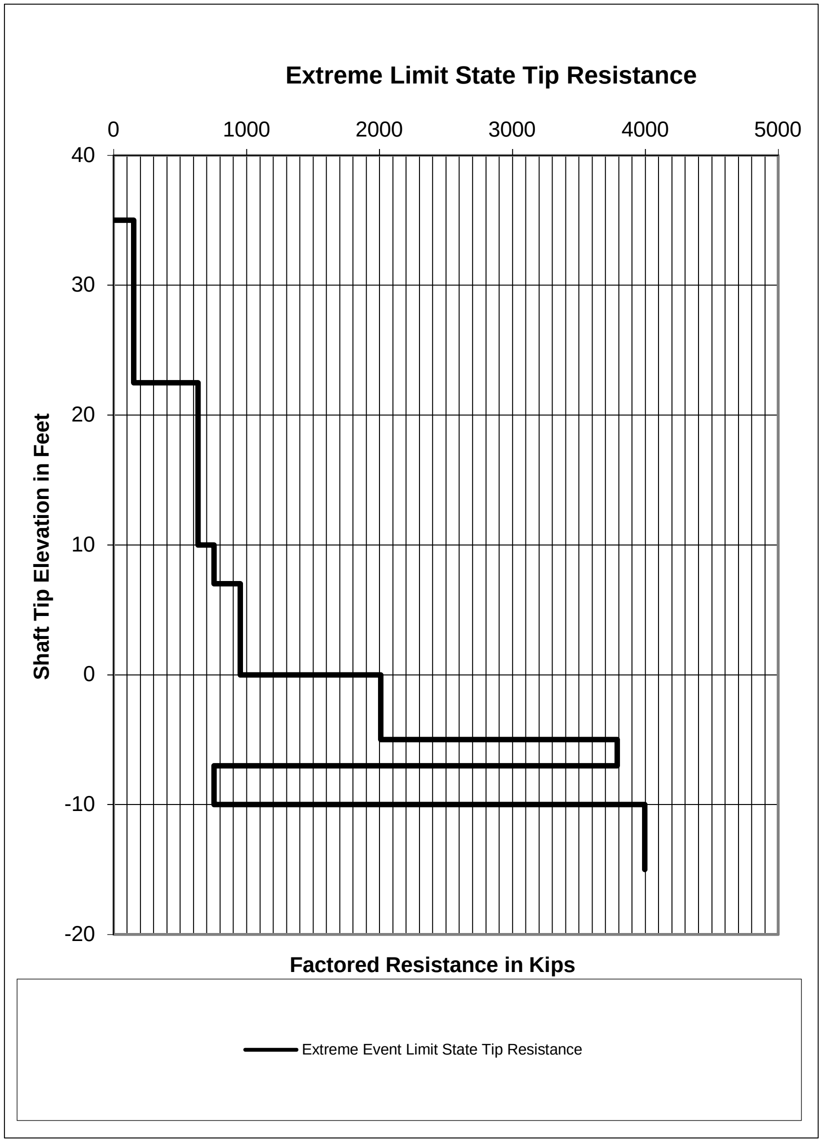
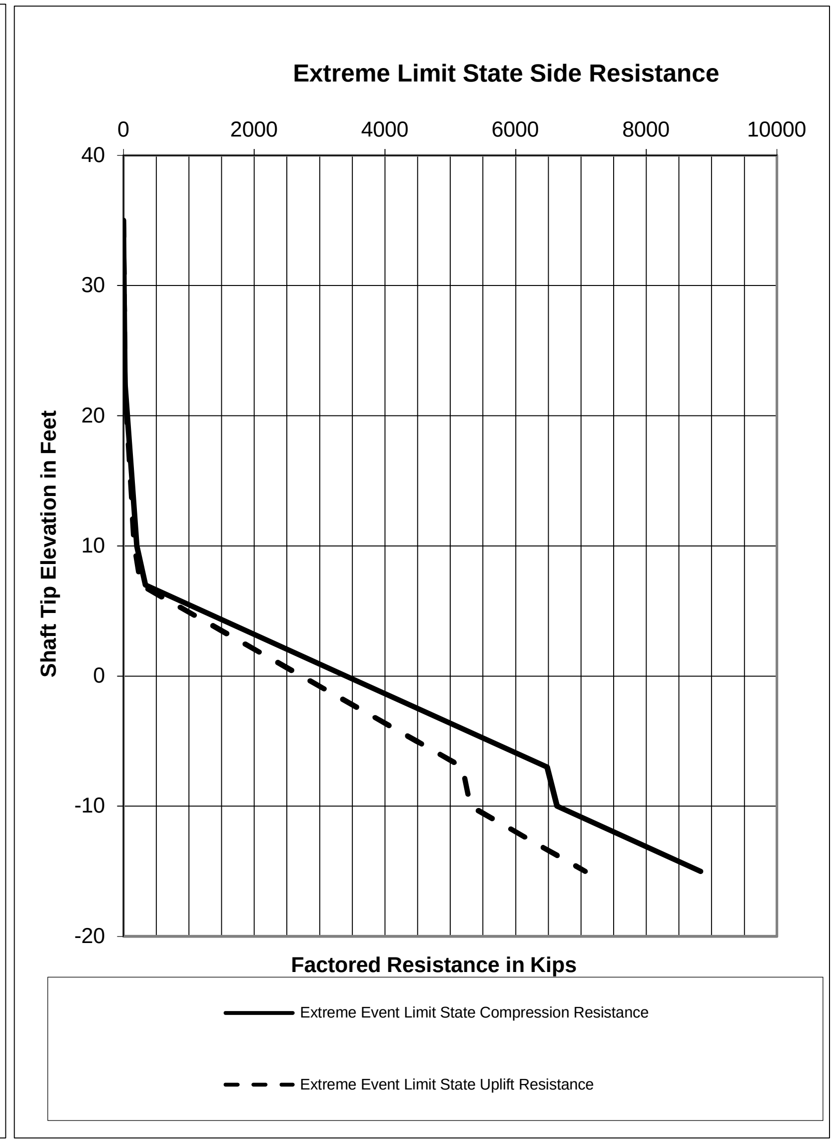
HARTCROWSER

Figure

7





Notes:
1) The net weight of the shaft is not included in these charts and should be treated as a load applied to the top of the shaft.
2) Factored resistances are based on resistance factors from 2014 AASHTO LRFD Bridge Design Specifications, Table 10.5.5.2.4-1.

Riverfront Park Bridge Foundation Design
Milwaukie, Oregon

Axial Resistance Chart for : 4 ft Diameter Drilled Shaft Extreme Limit State - South Abutment

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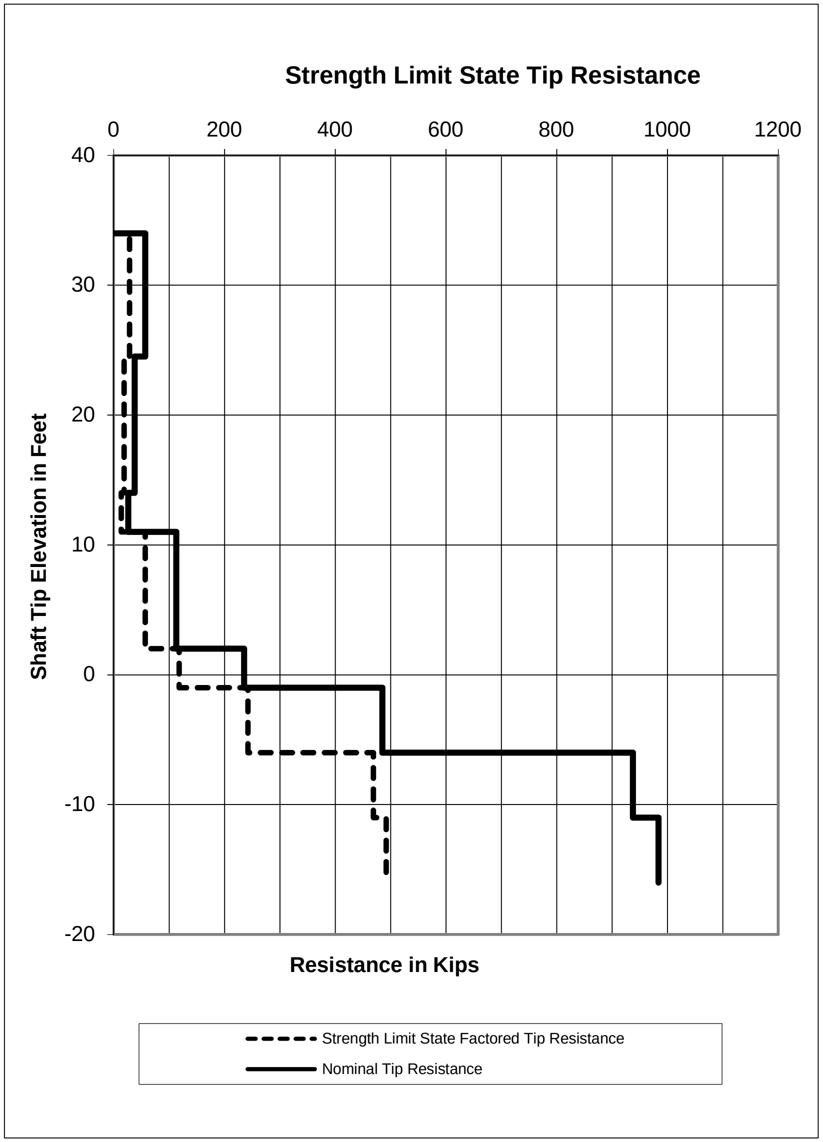
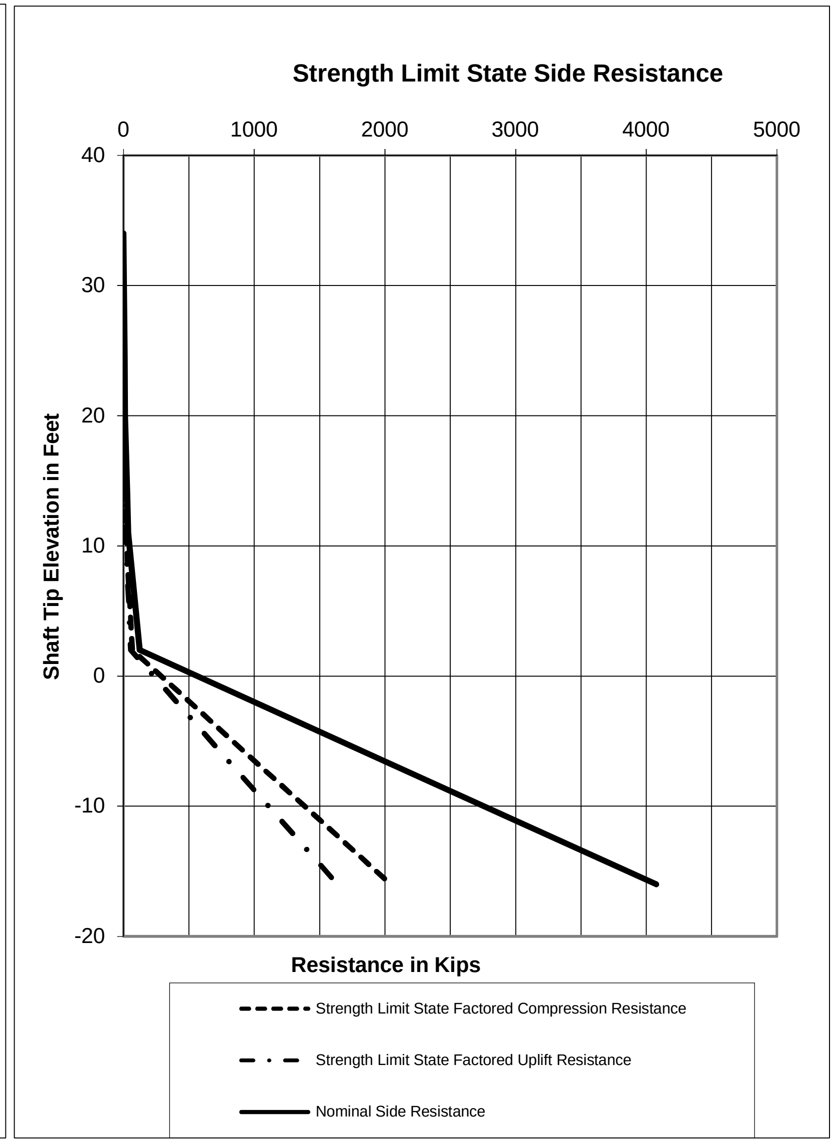
HARTCROWSER

Figure
8





Notes:

1) The net weight of the shaft is not included in these charts and should be treated as a load applied to the top of the shaft.

2) Factored resistances are based on resistance factors from 2014 AASHTO LRFD Bridge Design Specifications, Table 10.5.5.2.4-1.

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Riverfront Park Bridge Foundation Design
Milwaukie, Oregon

Axial Resistance Chart for : 2 ft Diameter Drilled Shaft Strength Limit State - North Abutment

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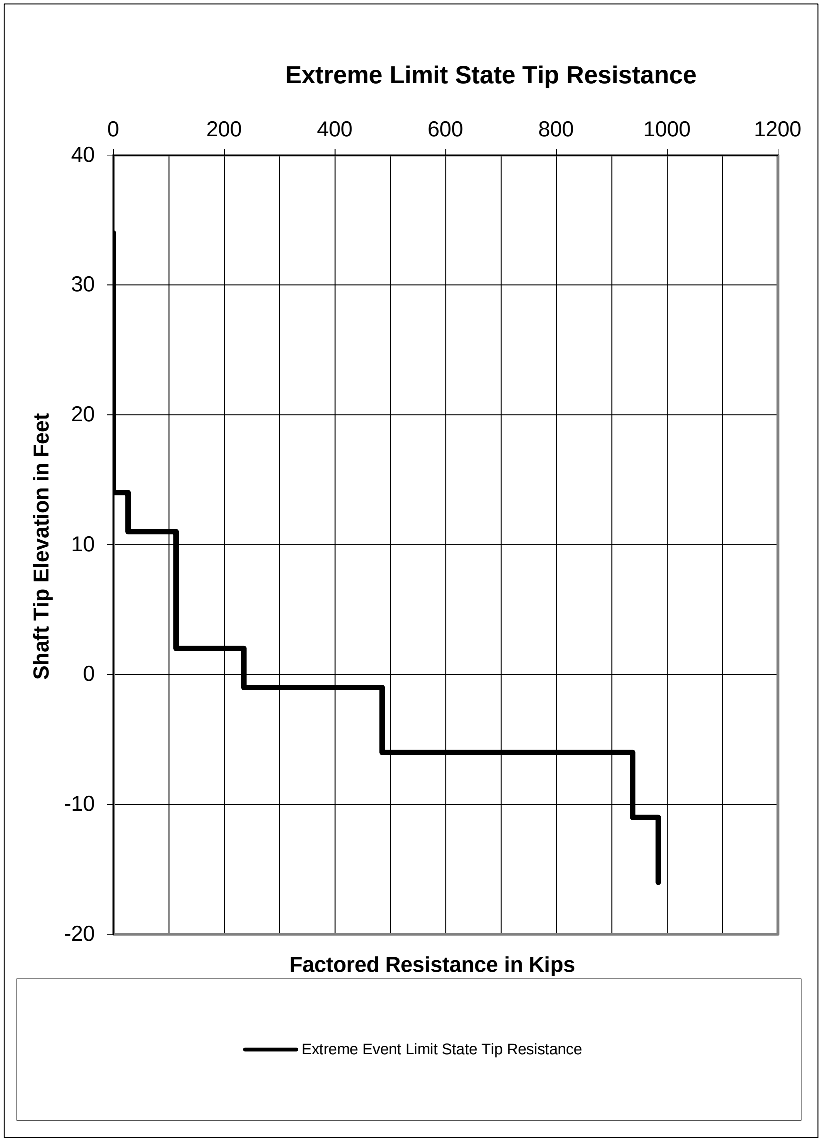
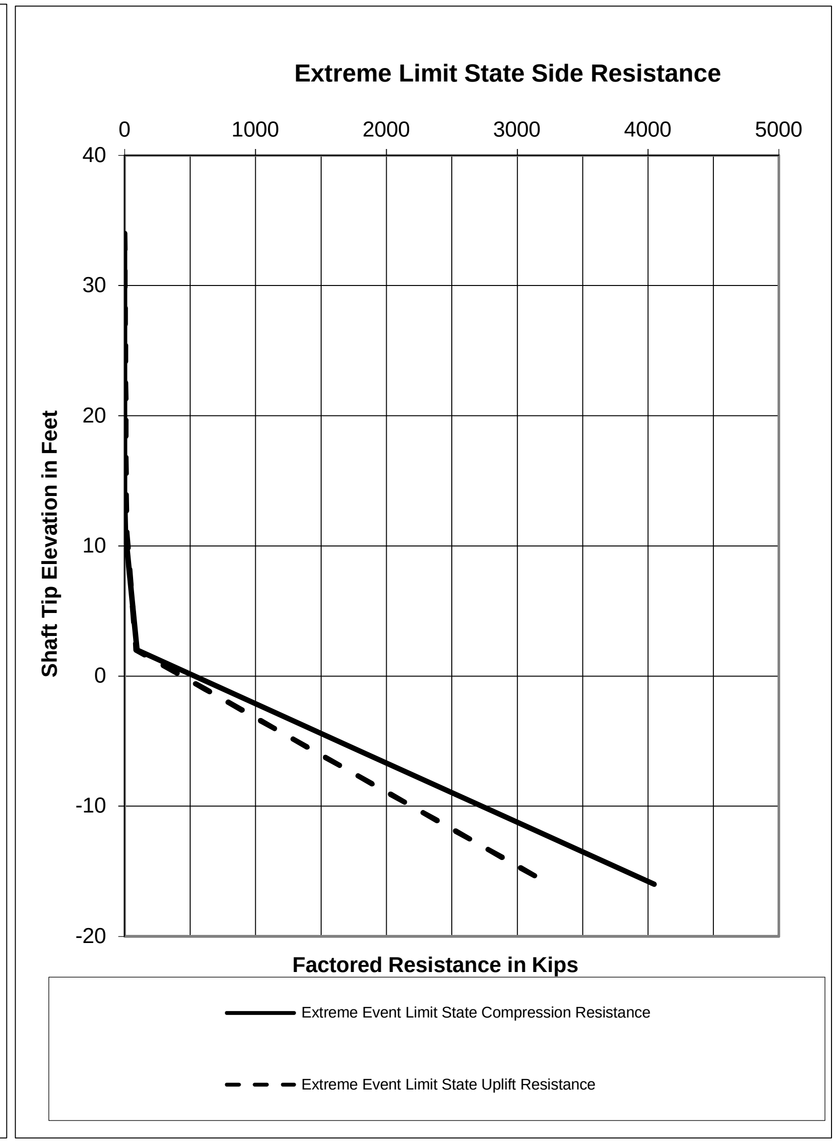
Figure

9





Notes:

1) The net weight of the shaft is not included in these charts and should be treated as a load applied to the top of the shaft.

2) Factored resistances are based on resistance factors from 2014 AASHTO BDS, Table 10.5.5.2.4-1.

3) In Accordance with the 2014 AASHTO BDS tip and side resistance is set to zero in liquefiable layers above elevation14 feet.

4) We estimate approximately 14 kips of liquefaction induced downdrag load in the extreme limit state. This downdrag load can be be considered unfactored uplift resistance. The resistance factor for uplift resistance in the extreme limit state is 0.8.

Riverfront Park Bridge Foundation Design
Milwaukie, Oregon

Axial Resistance Chart for : 2 ft Diameter
Drilled Shaft Extreme Limit State - North Abutment

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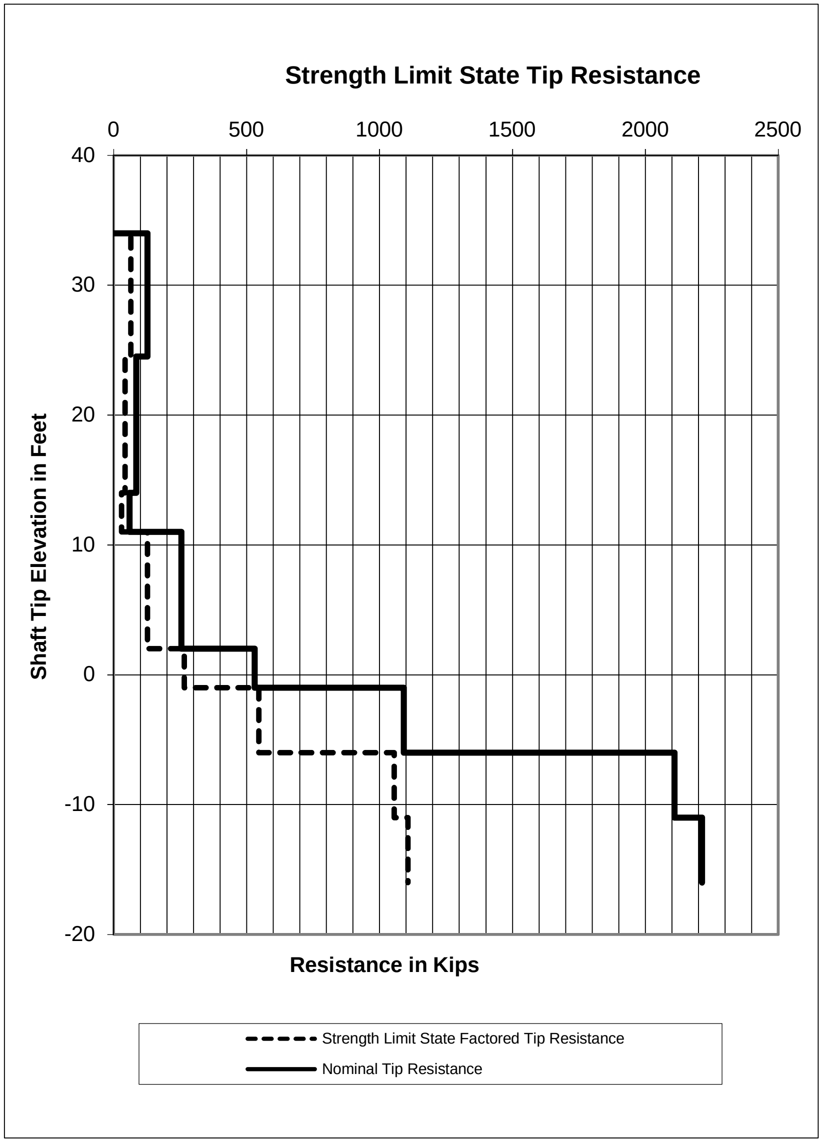
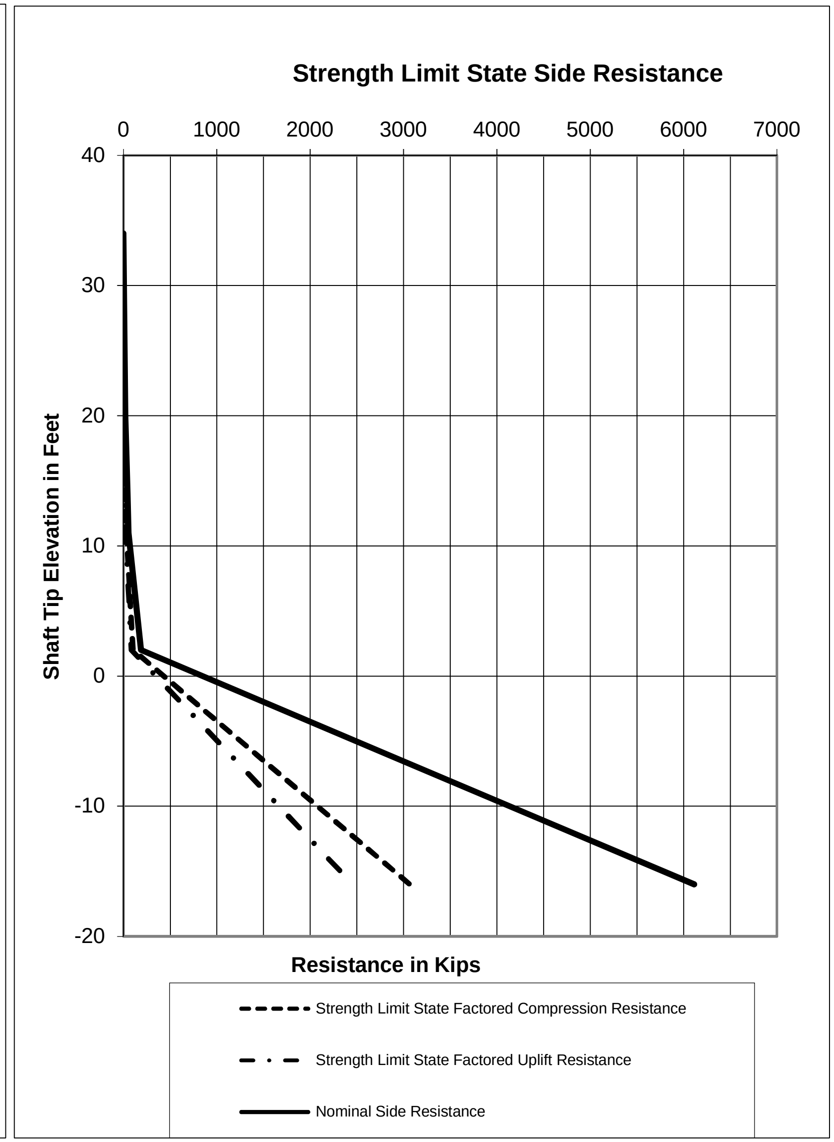
HARTCROWSER

Figure
10





Notes:
1) The net weight of the shaft is not included in these charts and should be treated as a load applied to the top of the shaft.
2) Factored resistances are based on resistance factors from 2014 AASHTO LRFD Bridge Design Specifications, Table 10.5.5.2.4-1.
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Riverfront Park Bridge Foundation Design
Milwaukie, Oregon

Axial Resistance Chart for : 3 ft Diameter Drilled Shaft Strength Limit State - North Abutment

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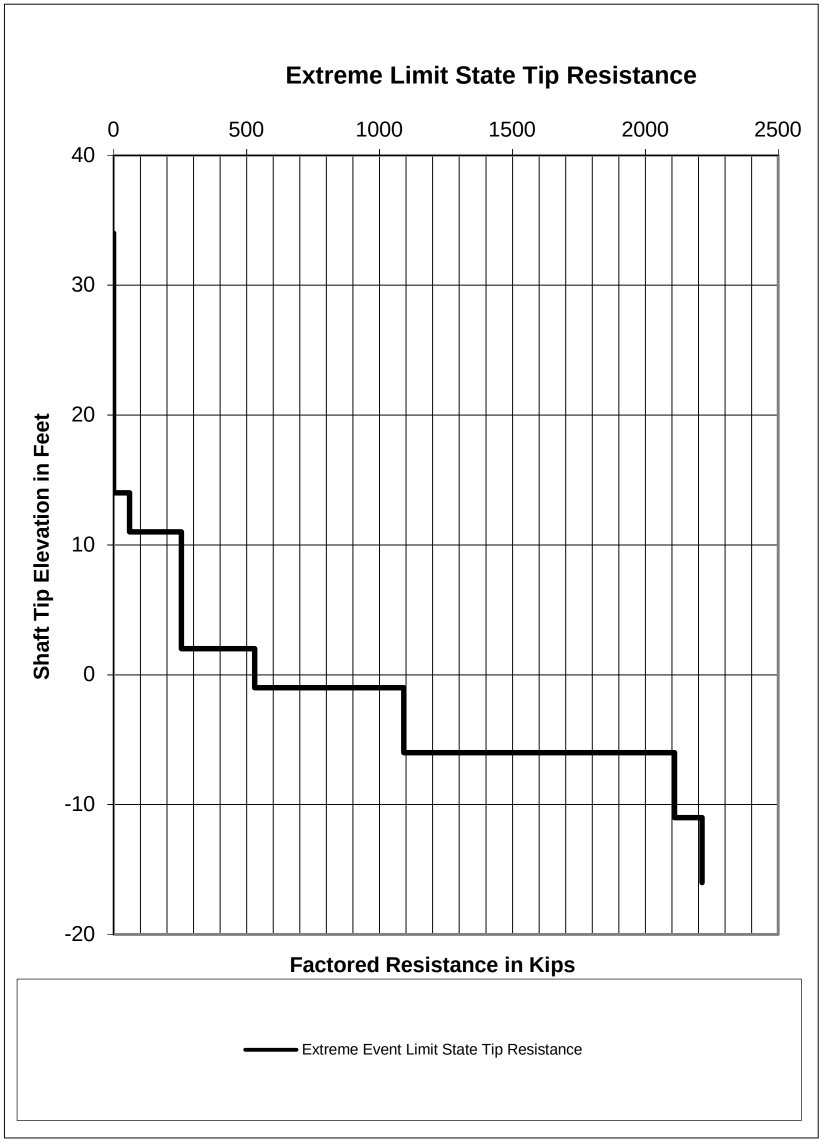
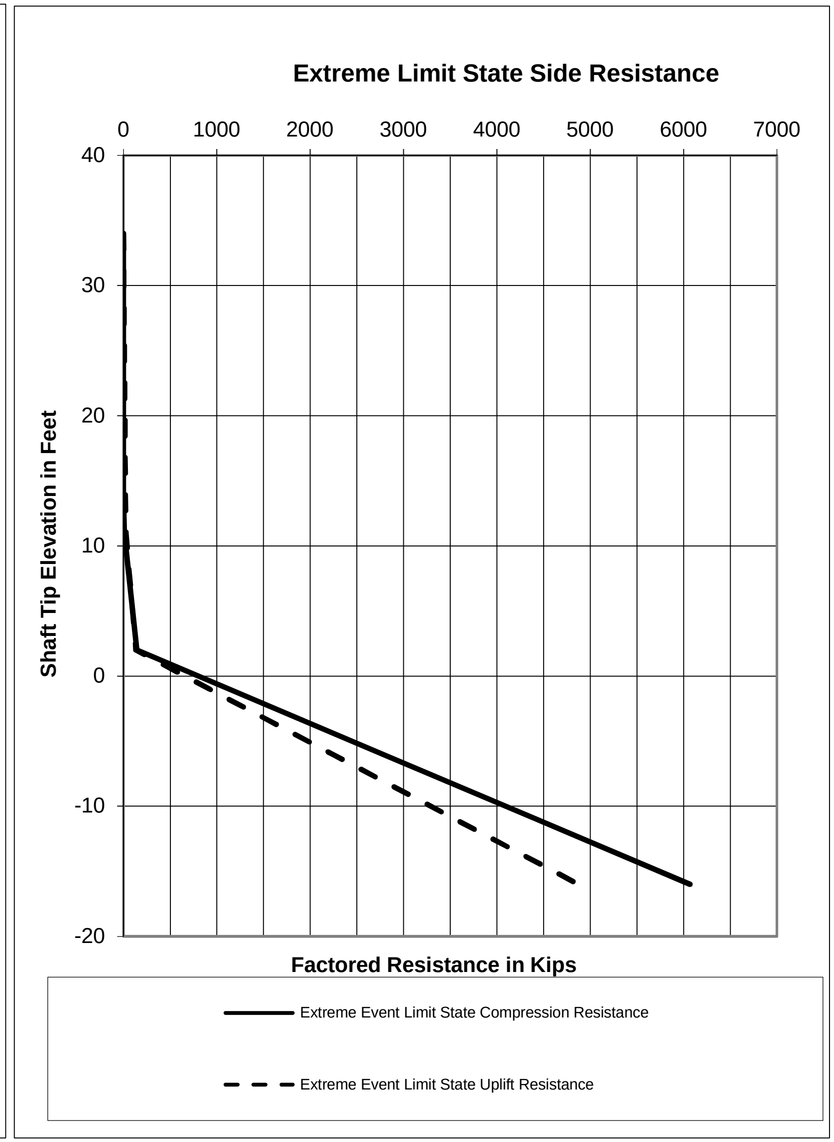
Figure 11





Notes:

1) The net weight of the shaft is not included in these charts and should be treated as a load applied to the top of the shaft.

2) Factored resistances are based on resistance factors from 2014 AASHTO BDS, Table 10.5.5.2.4-1.

3) In Accordance with the 2014 AASHTO BDS tip and side resistance is set to zero in liquefiable layers above elevation14 feet.

4) We estimate approximately 21 kips of liquefaction induced downdrag load in the extreme limit state. This downdrag load can be be considered unfactored uplift resistance. The resistance factor for uplift resistance in the extreme limit state is 0.8.

Riverfront Park Bridge Foundation Design
Milwaukie, Oregon

Axial Resistance Chart for : 3 ft Diameter
Drilled Shaft Extreme Limit State - North Abutment

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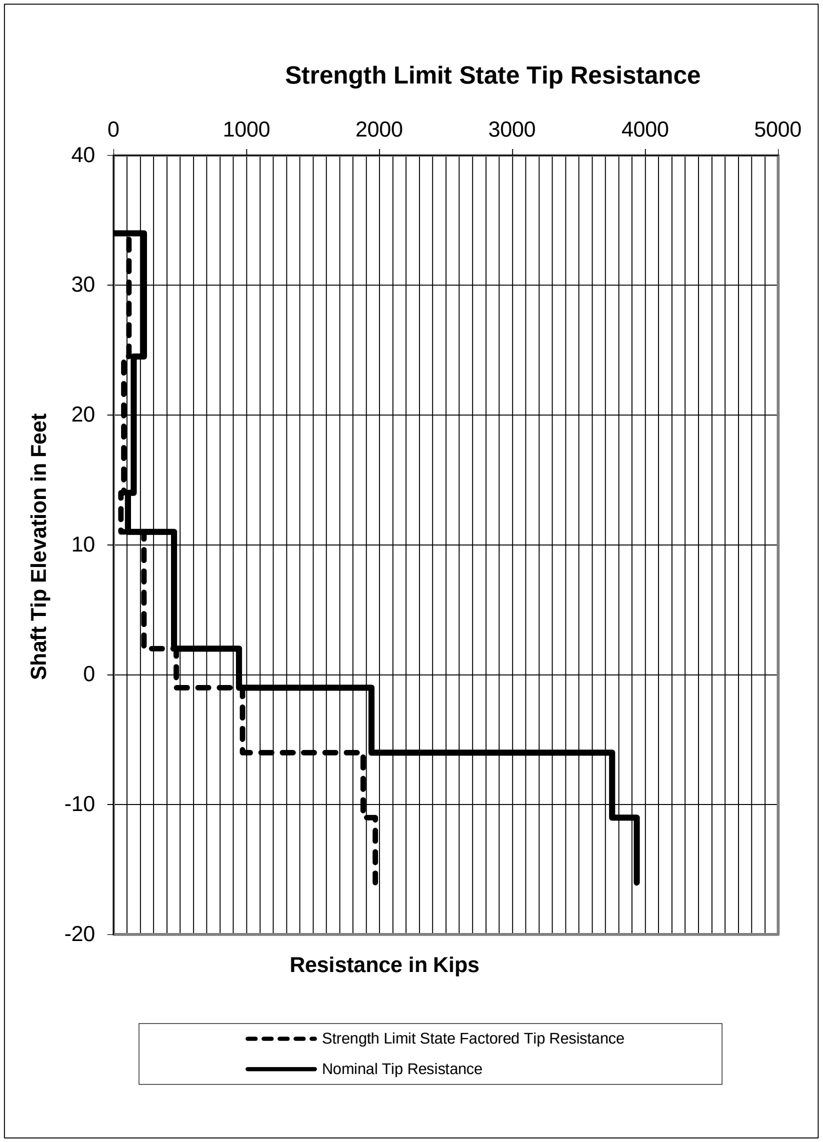
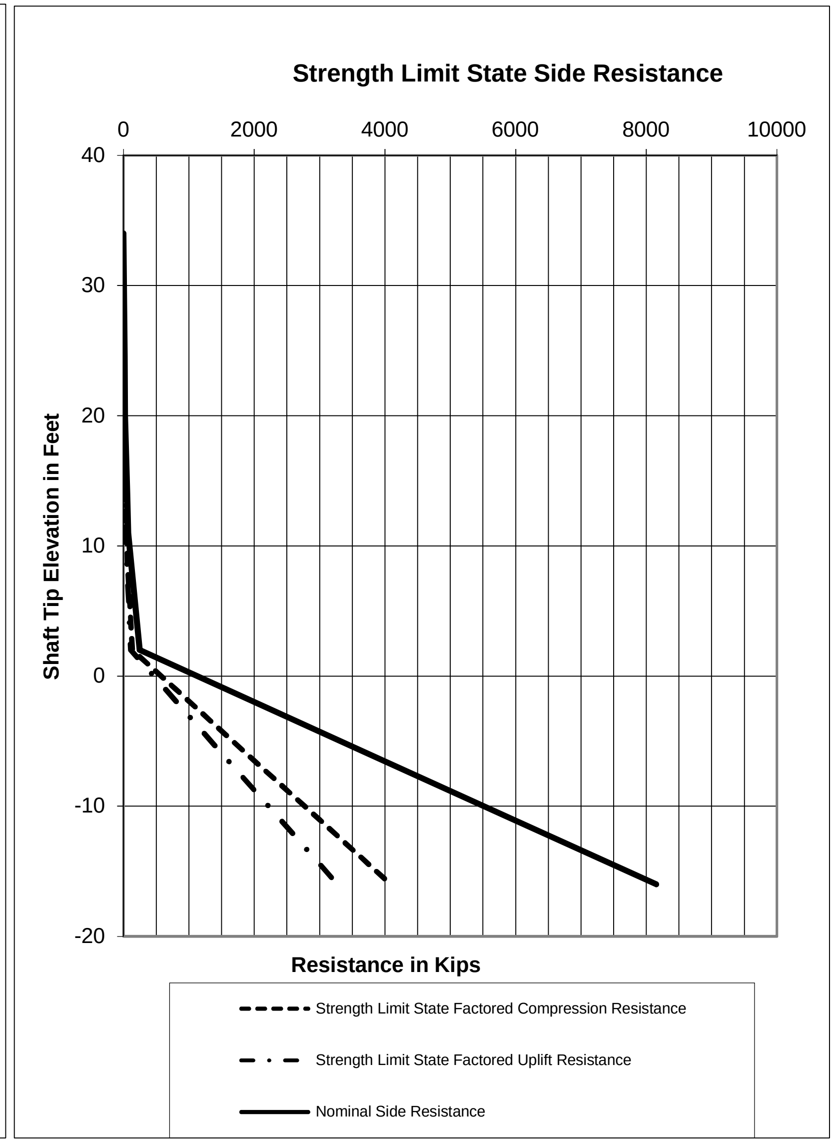
HARTCROWSER

Figure

12





Notes:

1) The net weight of the shaft is not included in these charts and should be treated as a load applied to the top of the shaft.

2) Factored resistances are based on resistance factors from 2014 AASHTO LRFD Bridge Design Specifications, Table 10.5.5.2.4-1.

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Riverfront Park Bridge Foundation Design
Milwaukie, Oregon

Axial Resistance Chart for : 4 ft Diameter Drilled Shaft Strength Limit State - North Abutment

154-038-004

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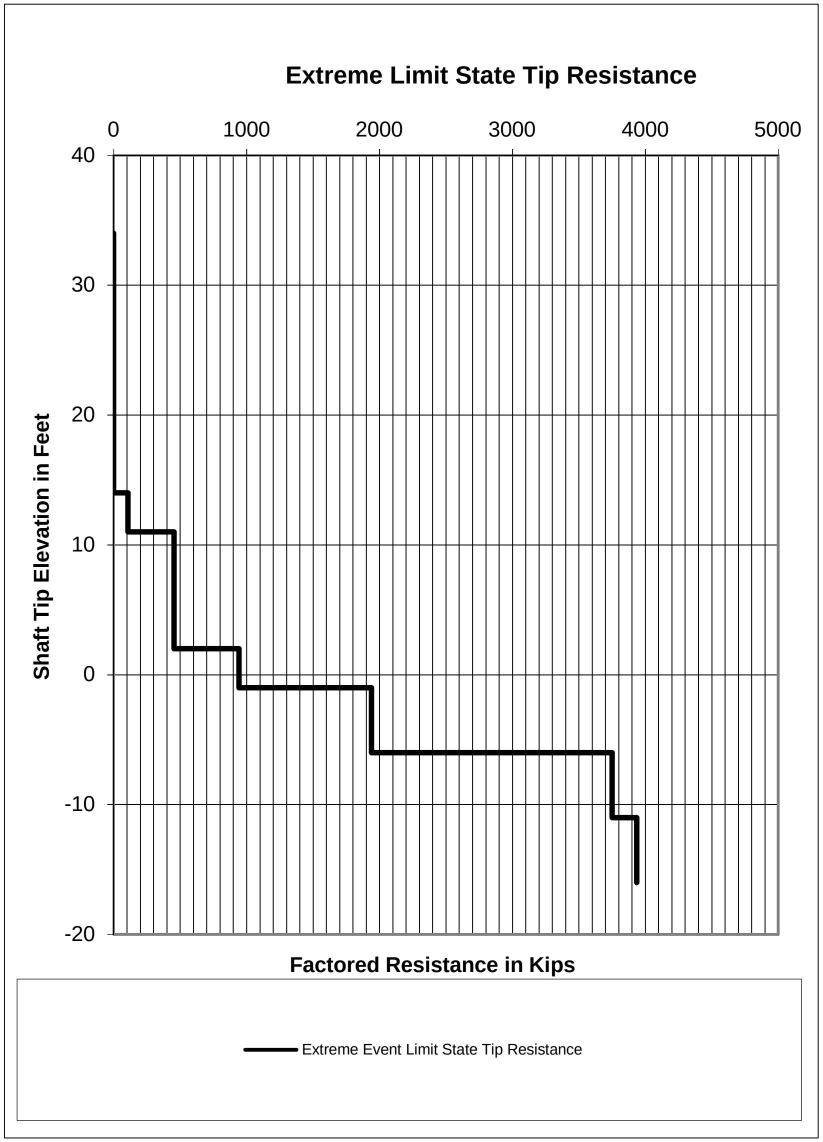
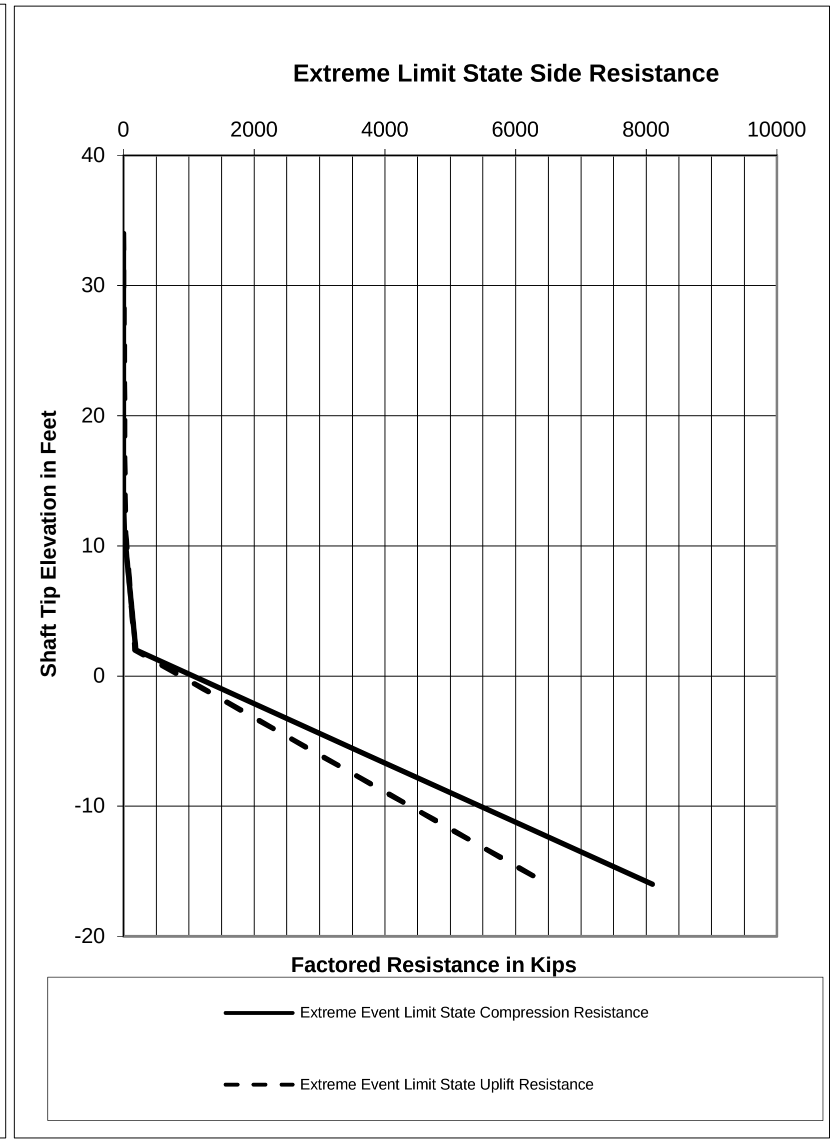
HARTCROWSER

Figure

13





Notes:

1) The net weight of the shaft is not included in these charts and should be treated as a load applied to the top of the shaft.

2) Factored resistances are based on resistance factors from 2014 AASHTO BDS, Table 10.5.5.2.4-1.

3) In Accordance with the 2014 AASHTO BDS tip and side resistance is set to zero in liquefiable layers above elevation14 feet.

4) We estimate approximately 29 kips of liquefaction induced downdrag load in the extreme limit state. This downdrag load can be be considered unfactored uplift resistance. The resistance factor for uplift resistance in the extreme limit state is 0.8.

Riverfront Park Bridge Foundation Design
Milwaukie, Oregon

Axial Resistance Chart for : 4 ft Diameter Drilled Shaft Extreme Limit State - North Abutment

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Figure

14

ATTACHMENT A
Field Explorations and Laboratory Testing

ATTACHMENT A
FIELD EXPLORATIONS AND LABORATORY TESTING

Field Explorations

Explorations and Their Locations

We evaluated subsurface conditions at the site by completing two drilled borings on January 5 and 8, 2016, and March 15, 2016. The borings were drilled using a track-mounted, CME 55 drill rig on January 5, 2016, and a truck-mounted CME-75 drill rig on January 8 and March 15, 2016. The drill rigs are owned and operated by Western States Soil Conservation, Inc. of Hubbard, Oregon.

The field explorations were coordinated by geologists on our staff, who classified the various soil and rock units encountered, obtained representative samples for geotechnical testing, and maintained a detailed log of each boring. The exploration logs are included in this attachment. Results of the laboratory testing are indicated on the exploration logs.

The locations of the explorations are shown on letter report Figure 2. The exploration locations were estimated based on field measurements.

The exploration logs within this attachment show our interpretation of the drilling, sampling, and testing data. They indicate the depth where the soils change. Note that the change may be gradual. In the field, we classified the samples taken from the explorations according to the methods presented on the *Key to Exploration Logs* in this attachment. The key also provides a legend explaining the symbols and abbreviations used in the logs.

Soil Sampling Procedures

Materials encountered in the explorations were classified in the field in general accordance with ASTM International (ASTM) Standard Practice D 2488 "*Standard Practice for the Classification of Soils* (Visual Manual Procedure) and the ODOT *Soil and Rock Classification Manual*. Soil and rock classifications and sampling intervals are shown on the exploration logs in this attachment.

Soil and rock samples were obtained from the borings using the following methods.

- Disturbed samples were obtained using a SPT sampler in general conformance with ASTM Test Method D 1586 "Standard Method for Penetration Test and Split-Barrel Sampling of Soils." The sampler was driven with a 140-pound auto-trip hammer falling 30 inches. The N value, or number of blows required to drive the sampler 1 foot or as otherwise indicated into the soils, is shown adjacent to the sample symbols on the boring logs. Disturbed samples were obtained from the sampler for subsequent classification and testing.

- Representative core samples of competent bedrock were obtained using wire-line HQ drilling techniques in general conformance with ASTM Test Method D 2113-14 “Standard Practice for Rock Core Drilling and Sampling of Rock for Site Exploration.” An HQ3 (3.81 in hole O.D., 2.44 in core O.D.) wireline core barrel sampler was used. The core barrel assembly was drilled into intact rock, and the rock sample then retrieved using a cable to recover a removable inner barrel from which the sample was extruded, logged, and stored in prepared boxes for subsequent laboratory testing and storage. Photographs of the rock core collected is included as Figures A-8 and A-9.

Laboratory Testing

A geotechnical laboratory testing program was performed for this study to evaluate the basic index and geotechnical engineering properties of the site soils and the strength of the *in situ* bedrock. Basic testing was completed at our in-house laboratory in our Portland, Oregon office. Rock strength testing was performed at Columbia West Engineering, Inc. The tests performed and the procedures followed are outlined below.

Soil Classification

Soil samples were visually classified in our laboratory to verify the field classifications in a relatively controlled laboratory environment. Classifications were made in general accordance with the Unified Soil Classification System (USCS) and ASTM Test Method D 2487.

Water Content Determinations

Water contents were determined for select samples recovered in the explorations in general accordance with ASTM Test Method D 2216. The test results are shown on the appropriate exploration logs and shown on Figure A-5 in this attachment.

Atterberg Limits

Atterberg limits (liquid limit, plastic limit and plasticity index) of a selected fine-grained soil sample were obtained in general accordance with ASTM Test Method D 4318-02. The test results are shown on Figure A-6 in this attachment.

Sieve Analyses

A sieve analysis was performed on a selected sample to determine the quantitative distribution of particle sizes in the original sample. The test was performed in general accordance with ASTM Test Method D 6913-04. The test results are indicated on Figure A-7 in this attachment.

Unconfined Compression Test

Unconfined compression testing was performed on two intact rock specimens obtained from rock cores obtained during site subsurface explorations. The tests were performed in general accordance with ASTM D 4543. The test results are shown at the end of this attachment.

KEY TO EXPLORATION LOGS

HARTCROWSER

SOIL CLASSIFICATION CHART

MATERIAL TYPES	MAJOR DIVISIONS		GROUP SYMBOL	SOIL GROUP NAMES & LEGEND
COARSE-GRAINED SOILS >50% RETAINED ON NO. 200 SIEVE	GRAVELS >50% OF COARSE FRACTION RETAINED ON NO 4. SIEVE	CLEAN GRAVELS <5% FINES	GW	WELL-GRADED GRAVEL
			GP	POORLY-GRADED GRAVEL
		GRAVELS WITH FINES, >12% FINES	GM	SILTY GRAVEL
			GC	CLAYEY GRAVEL
	SANDS >50% OF COARSE FRACTION PASSES ON NO 4. SIEVE	CLEAN SANDS <5% FINES	SW	WELL-GRADED SAND
			SP	POORLY-GRADED SAND
		SANDS AND FINES >12% FINES	SM	SILTY SAND
			SC	CLAYEY SAND
FINE-GRAINED SOILS >50% PASSES NO. 200 SIEVE	SILTS AND CLAYS LIQUID LIMIT<50	INORGANIC	CL	LEAN CLAY
			ML	SILT
		ORGANIC	OL	ORGANIC CLAY OR SILT
	SILTS AND CLAYS LIQUID LIMIT>50	INORGANIC	CH	FAT CLAY
			MH	ELASTIC SILT
		ORGANIC	OH	ORGANIC CLAY OR SILT
HIGHLY ORGANIC SOILS			PT	PEAT

Note: Multiple symbols are used to indicate borderline or dual classifications

OTHER MATERIAL SYMBOLS

- Concrete
- Asphalt
- Topsoil

MOISTURE MODIFIERS

Dry	-	Absence of moisture, dusty, dry to the touch
Moist	-	Damp, but no visible water
Wet	-	Visible free water or saturated, usually soil is obtained from below the water table

SEEPAGE MODIFIERS

None	-	
Slow	-	< 1 gpm
Moderate	-	1-3 gpm
Heavy	-	> 3 gpm

CAVING MODIFIERS

None	-	
Minor	-	isolated
Moderate	-	frequent
Severe	-	general

MINOR CONSTITUENTS

Trace	-	< 5% (silt/clay)
Occasional	-	< 15% (sand/gravel)
With	-	5-15% (silt/clay) in sand or gravel 15-30% (sand/gravel) in silt or clay

SAMPLE TYPES

- Dames & Moore
- Standard Penetration Test (SPT)
- Shelby Tube
- Bulk or Grab

LABORATORY/ FIELD TESTS

ATT	-	Atterberg Limits
CP	-	Laboratory Compaction Test
CA	-	Chemical Analysis (Corrosivity)
CN	-	Consolidation
DD	-	Dry Density
DS	-	Direct Shear
HA	-	Hydrometer Analysis
OC	-	Organic Content
PP	-	Pocket Penetrometer (TSF)
P200	-	Percent Passing No. 200 Sieve
SA	-	Sieve Analysis
SW	-	Swell Test
TV	-	Torvane Shear
UC	-	Unconfined Compression

GROUNDWATER SYMBOLS

- Water Level (at time of drilling)
- Water Level (at end of drilling)
- Water Level (after drilling)

STRATIGRAPHIC CONTACT

- Distinct contact between soil strata or geologic units
- Gradual or approximate change between soil strata or geologic units

Notes:

Blowcount (N) is recorded for driven samplers as the number of blows required to advance sampler 12 inches (or distance noted) per ASTM D-1586. See exploration log for hammer weight and drop.

When the Dames & Moore (D&M) sampler was driven with a 140-pound hammer (denoted on logs as D+M 140), the field blow counts (N-value) shown on the logs have been reduced by 50% to approximate SPT N-values.

Soil density/consistency in borings is related primarily to the Standard Penetration Resistance. Soil density/consistency in test pits and probes is estimated based on visual observation and is presented parenthetically on the logs.

Refer to the report text and exploration logs for a proper understanding of subsurface conditions. Descriptions on the logs apply only at the exploration locations at the time the explorations were made. The logs are not warranted to be representative of the subsurface conditoins at other locations or times.

KEY TO BEDROCK TERMS (1 of 2)

(ODOT, 1987)

HARTCROWSER

8910 SW Gemini Drive
Beaverton, Oregon 97008

Scale of Relative Rock Weathering

Term	Description
Fresh	Crystals are bright. Discontinuities may show some minor surface staining. No discoloration in rock fabric.
Slightly Weathered	Rock mass is generally fresh. Discontinuities are stained and may contain clay. Some discoloration in rock fabric. Decomposition extends up to 1 inch into rock.
Moderately Weathered	Rock mass is decomposed 50% or less. Significant portions of rock show discoloration and weathering effects. Crystals are dull and show visible chemical alteration. Discontinuities are stained and may contain secondary mineral deposits.
Predominantly Decomposed	Rock mass is more than 50% decomposed. Rock can be excavated with geologist's pick. All discontinuities exhibit secondary mineralization. Complete discoloration of rock fabric. Surface of core is friable and usually pitted due to washing out of highly altered minerals by drilling water.
Decomposed	Rock mass is completely decomposed. Original rock "fabric" may be evident. May be reduced to soil with hand pressure.

Scale of Relative Rock Hardness

Hardness Designation	Term	Field Identification	Uniaxial Compressive Strength
R0	Extremely Soft	Can be indented with difficulty by thumbnail. May be moldable or friable with finger pressure.	< 100 psi
R1	Very Soft	Crumbles under firm blows with point of a geology pick. Can be peeled by a pocket knife. Scratched with fingernail.	100-1000 psi
R2	Soft	Can be peeled by a pocket knife with difficulty. Cannot be scratched with fingernail. Shallow indentation made by firm blow of geology pick.	1000-4000 psi
R3	Medium Hard	Can be scratched by knife or pick. Specimen can be fractured with a single firm blow of hammer/geology pick.	4000-8000 psi
R4	Hard	Can be scratched with knife or pick only with difficulty. Several hard hammer blows required to fracture specimen.	8000-16000 psi
R5	Very Hard	Cannot be scratched by knife or sharp pick. Specimen requires many blows of hammer to fracture or chip. Hammer rebounds after impact.	> 16000 psi

Joint and Bedding Spacing Terms

Spacing	Joint Spacing Terms	Bedding/Foliation Spacing Terms
Less than 2 inches	Very Close	Very Thin (laminated)
2 inches to 1 foot	Close	Thin
1 foot to 3 feet	Moderately Close	Medium
3 feet to 10 feet	Wide	Thick
More than 10 feet	Very Wide	Very Thick (massive)

KEY TO BEDROCK TERMS (2 of 2)
(ODOT, 1987)

HARTCROWSER

8910 SW Gemini Drive
Beaverton, Oregon 97008

Stratification Terms

Term	Characteristics
Laminations	Thin beds (<1 cm)
Fissile	Tendency to break along laminations.
Parting	Tendency to break parallel to bedding.
Foliation	Non-depositional (e.g., segregation and layering of minerals in metamorphic rock)

Igneous Rock Textures

Texture	Grain Size
Pegmatitic	Very large; diameters measured in inches or feet.
Phaneritic	Can be seen with the naked eye
Porphyritic	Grained of two widely different sizes
Aphanitic	Cannot be seen with the naked eye
Glassy	No grains present

Pyroclastic Rocks

Rock Name	Characteristics
Cinders	Uncemented glassy and vesicular ejecta 4-32 mm size
Tuff Breccia (Agglomerate)	Composed of ejecta >32mm size, in ash/tuff matrix, indurated
Lapilli Tuff	Composed of ejecta 4-32 mm size, in ash/tuff matrix, indurated
Tuff	Cemented volcanic ash particles <4mm size, indurated
Pumice	Excessively vesiculated glassy lava

Degree of Vesicularity

Designation	Percentage of Cavities (by volume) of Total Sample
Some Vesicules	5 to 25 Percent
Highly Vesicular	15 to 50 Percent
Scoriaceous	Greater than 50 Percent

OTHER TERMS:

Core Recover (CR) = the ratio of core recovered to the core run length expressed as a percentage.

Rock Quality Designation (RQD) = the percentage of rock core recovered in intact pieces of 4 inches or more in length in the length of a core run. Does not include mechanical breaks caused by drilling.

REFERENCE:

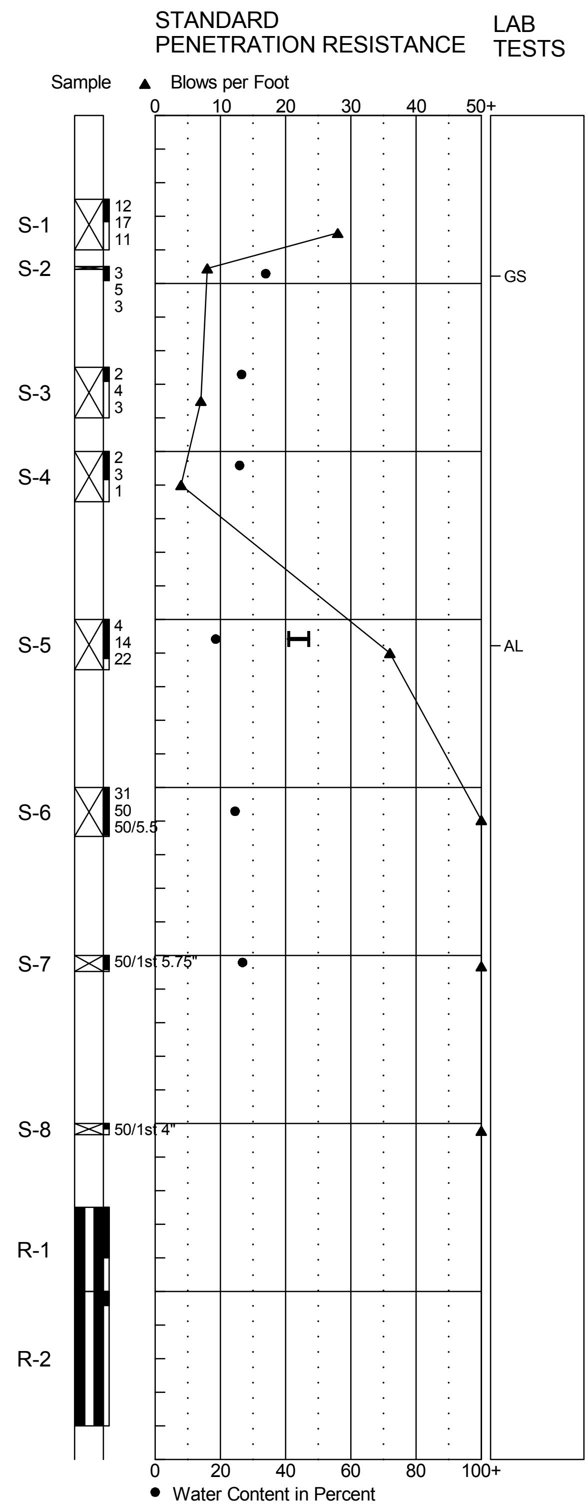
Oregon Department of Transportation (ODOT), 1987. *Soil and Rock Classification Manual*, May 1987.

Boring Log B-1

Location:
Approximate Ground Surface Elevation (feet): 35
Horizontal Datum: N/A
Vertical Datum: N/A

Drill Equipment: CME 55/CME 75 Mud Rotary
Hammer Type: Autohammer
Hole Diameter: 4 7/8 inches
Logged By: R. Pirot Reviewed By: A. Jones

USCS Class	Soil Descriptions	Depth in Feet
	Asphalt (6-inch thick)	0
GP	Medium dense, gray, wet, GRAVEL with sand, medium to coarse gravel, likely some cuttings too. (Fill)	
ML	Loose, wet, red-brown, SILT with fine sand, occasional fine gravel, low plasticity. (Alluvium)	5
SM	Loose, wet, red-brown, silty SAND to SAND with silt, fine to medium sand, low plasticity fines, homogenous.	10
SM	Dense, gray, moist, silty SAND to SAND with silt, medium angular sand, relict rock texture with flecks of yellow, green, and white. (Residual Soil)	15
	grades to very dense	20
SP-SM	Very dense, gray-brown, moist, SAND with silt, occasional gravel, fine to medium angular to subangular sand, relict rock texture with some pieces of very weak intact rock.	25
	BASALT, gray and red-brown, R2-R3, moderately weathered to predominantly decomposed, contains 4" layer of silty sand with gravel, silty sand with gravel infilling, highly fractured, rubble. (Basalt of Waverly Heights)	30
		35
		40

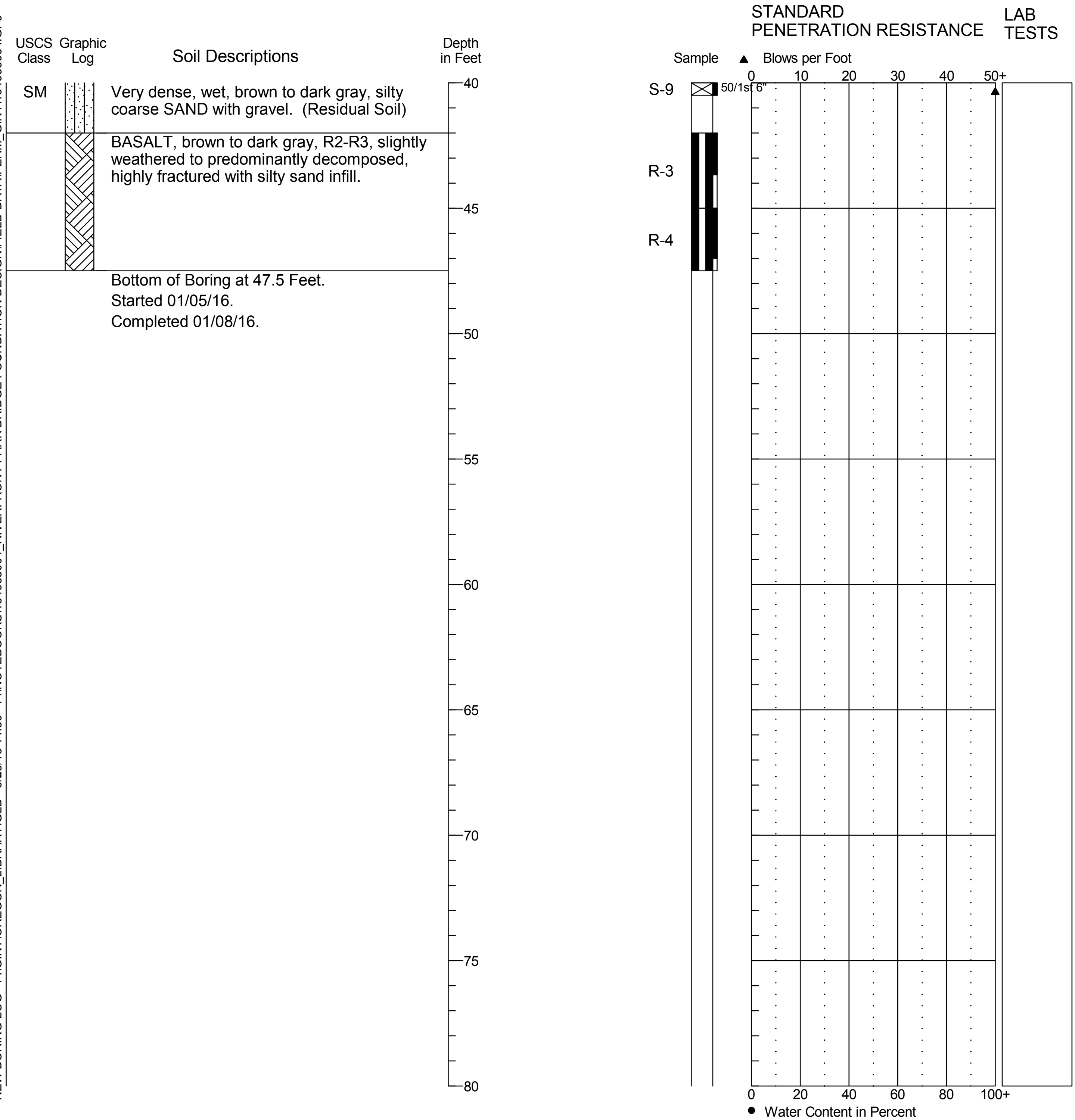


1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

Boring Log B-1

Location:
Approximate Ground Surface Elevation (feet): 35
Horizontal Datum: N/A
Vertical Datum: N/A

Drill Equipment: CME 55/CME 75 Mud Rotary
Hammer Type: Autohammer
Hole Diameter: 4 7/8 inches
Logged By: R. Pirot Reviewed By: A. Jones



1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
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HARTCROWSER

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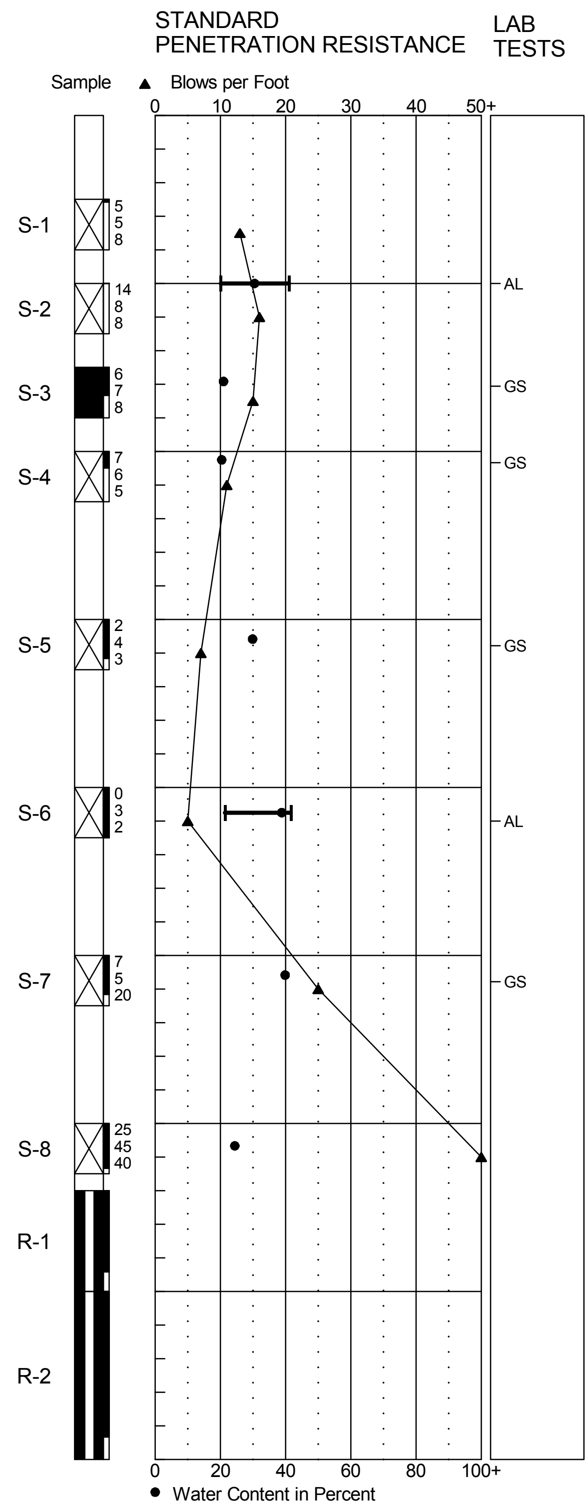
Boring Log B-2

Location:
Approximate Ground Surface Elevation (feet): 34
Horizontal Datum: N/A
Vertical Datum: N/A

Drill Equipment: CME 850/Mud Rotary
Hammer Type: Autohammer
Hole Diameter: 3 7/8 inches
Logged By: A. Jones   Reviewed By:

USCS Class	Soil Descriptions	Depth in Feet
GW	ASPHALT CONCRETE (4-inch thick)	0
	BASE AGGREGATE (26-inch thick)	
CL	Very stiff, moist, brown, sandy LEAN CLAY, fine to coarse sand, occasional gravel, low plasticity, laminated, scattered roots. (Fill)	5
SM/ML	Loose to medium dense, moist, brown, silty SAND to sandy SILT, fine sand, homogeneous. (Alluvium)	10, 15
CL	Medium stiff, moist to wet, sandy LEAN CLAY, occasional fine angular gravel, fine to coarse sand, low toughness, slow dilatancy, scattered roots and iron oxide staining. (Residual Soil)	20
SM	Medium to very dense, moist, brown, silty SAND, fine to coarse sand, occasional fine angular basalt gravel, weathered phenocrysts.	25, 30
	BASALT, dark gray, predominantly decomposed, extremely soft to very soft (R0 to R1), highly fractured, silty infill with iron oxide staining on fractured surfaces. (Basalt of Waverly Heights)	35
	Open joints, 90° to core axis, stepped, rough joint surface, 0.5cm clay fill	
	BASALT, gray to dark gray, slightly weathered, hard (R4), moderate to close joint spacing, some vesicles.	40



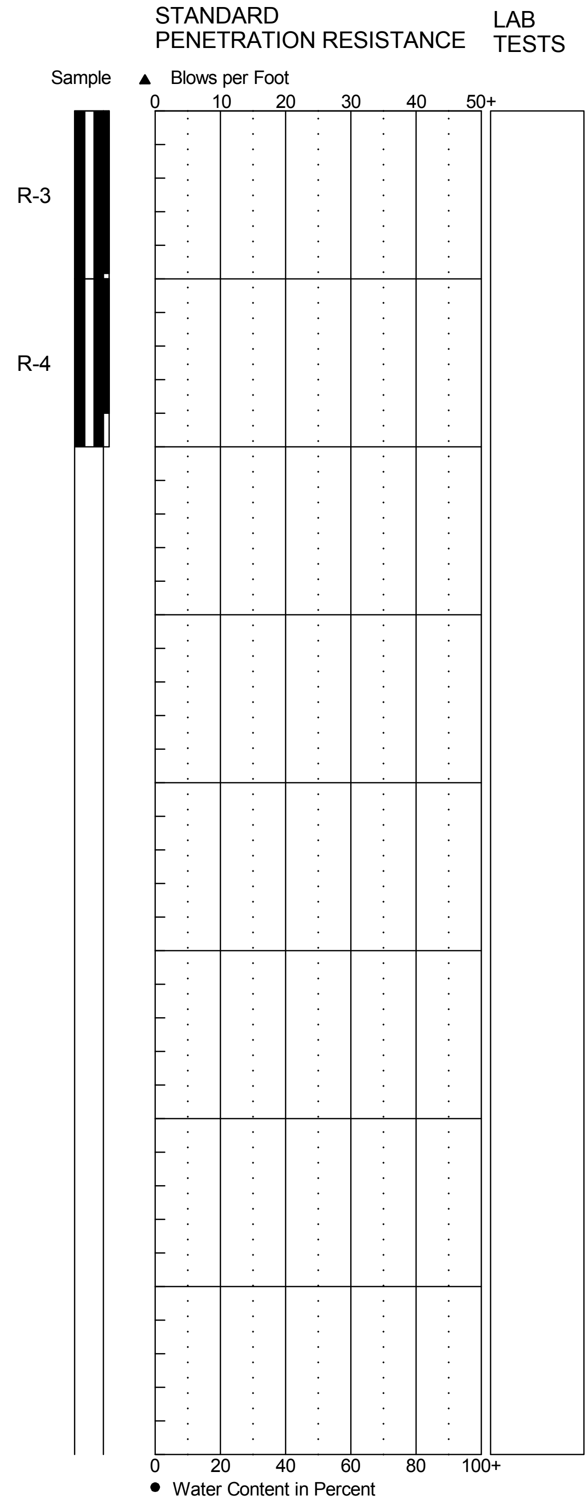
1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

Boring Log B-2

Location:
Approximate Ground Surface Elevation (feet): 34
Horizontal Datum: N/A
Vertical Datum: N/A

Drill Equipment: CME 850/Mud Rotary
Hammer Type: Autohammer
Hole Diameter: 3 7/8 inches
Logged By: A. Jones Reviewed By:

USCS Class	Graphic Log	Soil Descriptions	Depth in Feet
		Open joints, 0° to core axis, undulating, rough surface, 0.5cm clay fill	40
		BASALT, gray to dark gray, slightly weathered, hard (R4), moderate to close joint spacing, some vesicles. (cont'd)	
		Open joints, 90° to core axis, stepped, rough joint surface	
		Open joints, 90° and 45° to core axis, stepped, slightly rough joint surface, silty sand/clay fill 1-2-inch thick	45
		Open joints, 80° to core axis, stepped, slightly rough joint surface, clay fill 1.5-inch thick	
		Bottom of Boring at 50.0 Feet. Started 03/15/16. Completed 03/15/16.	50
			55
			60
			65
			70
			75
			80



1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual.
3. USCS designations are based on visual manual classification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling (ATD) or for date specified. Level may vary with time.

HARTCROWSER

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Figure A-4 2/2

HARTCROWSER

SUMMARY OF LABORATORY RESULTS

CLIENT City of Milwaukie

PROJECT NAME Riverfront Park Bridge Foundation Design

PROJECT NUMBER 154-038-004

PROJECT LOCATION Milwaukie, Oregon

Borehole	Depth	Liquid Limit	Plastic Limit	Plasticity Index	Maximum Size (mm)	%<#200 Sieve	Class-ification	Water Content (%)	Dry Density (pcf)	Remarks
B-1	4.5				76.2	72	ML	33.9		
B-1	7.5						SM	26.5		
B-1	10.0						SM	25.9		
B-1	15.0	47	41	6			SM	18.6		
B-1	20.0						SM	24.5		
B-1	25.0						SP-SM	26.8		
B-2	5.0	41	20	21			CL	30.5		
B-2	7.5				0.15	63	CL	21.0		
B-2	10.0				0.15	51	ML	20.4		
B-2	15.0				0.15	47	SM	29.9		
B-2	20.0	42	21	21			CL	38.8		
B-2	25.0				0.15	45	SM	39.8		
B-2	30.0						SM	24.4		

LAB SUMMARY - F:\GINT\OREGON_LIBRARY.GLB - 4/13/16 15:12 - F:\NOTEBOOKS\154038004_RIVERFRONT PARK BRIDGE FOUNDATION DESIGN\FIELD DATA\PERM_GINT\154038004.GPJ

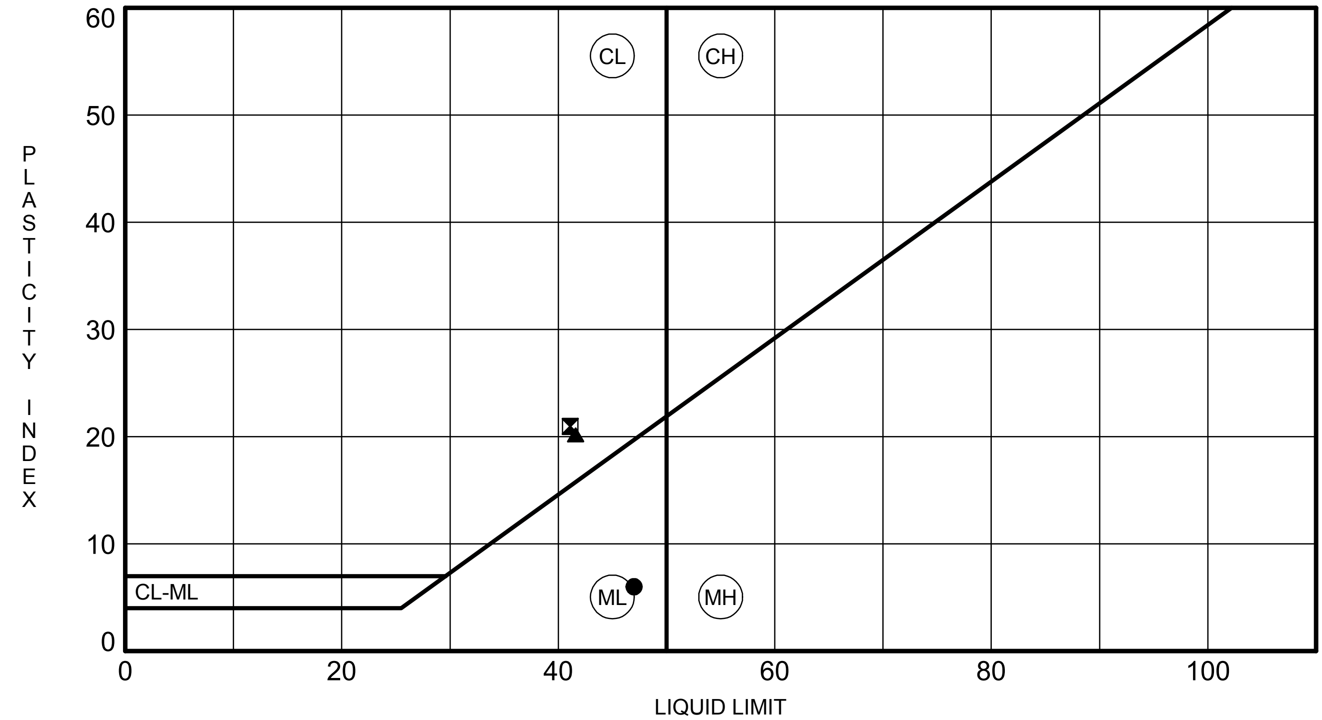
ATTERBERG LIMITS RESULTS

CLIENT City of Milwaukie

PROJECT NAME Riverfront Park Bridge Foundation Design

PROJECT NUMBER 154-038-004

PROJECT LOCATION Milwaukie, Oregon



	BOREHOLE	DEPTH	LL	PL	PI	Fines	Classification
●	**B-1**	**15.0**	**47**	**41**	**6**		**Silty SAND to SAND with silt**
⧗	**B-2**	**5.0**	**41**	**20**	**21**		**sandy LEAN CLAY, occasional gravel**
▲	**B-2**	**20.0**	**42**	**21**	**21**		**sandy LEAN CLAY**

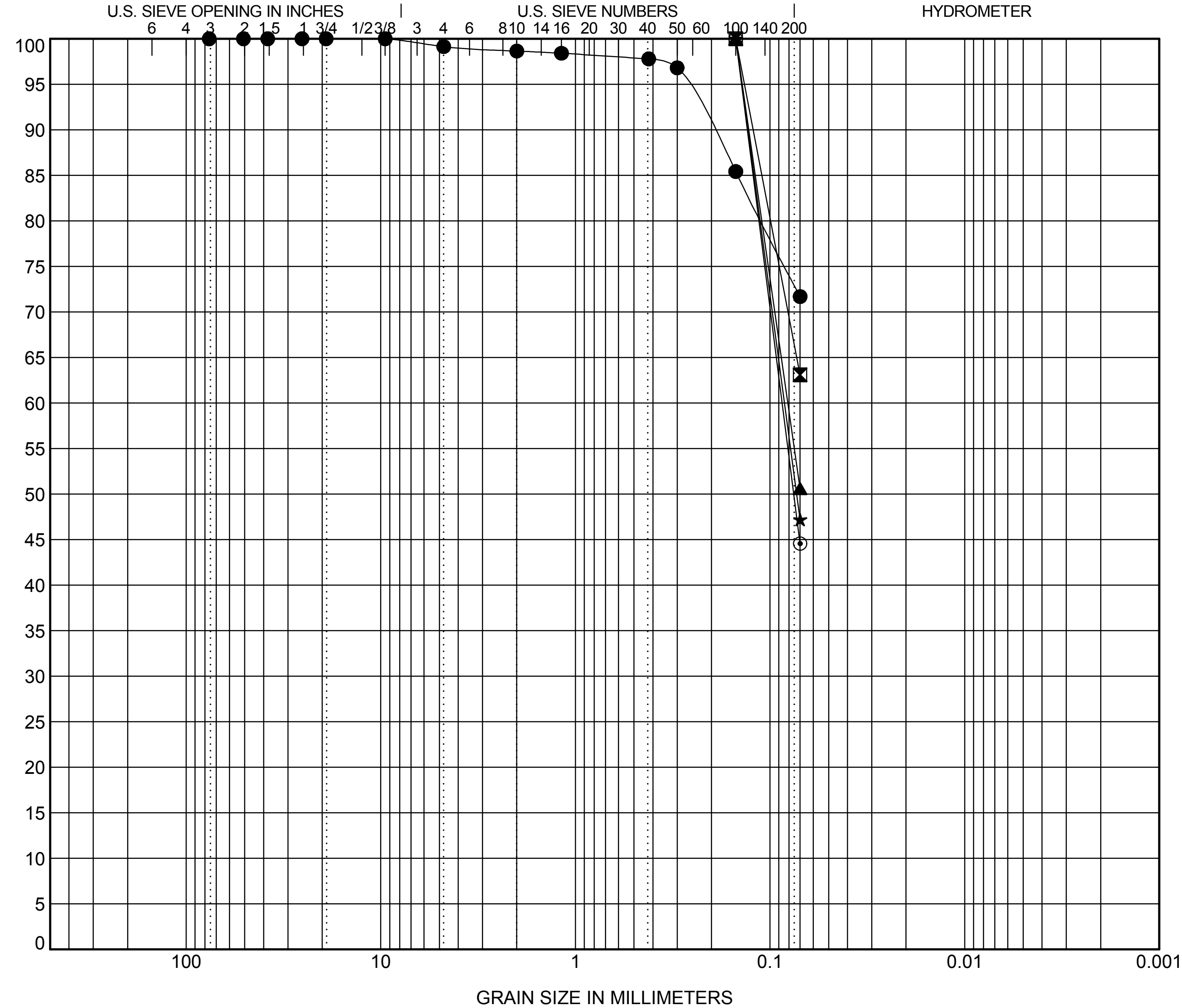
ATTERBERG LIMITS - F:\GINT\OREGON_LIBRARY.GLB - 4/13/16 15:14 - F:\NOTEBOOKS\154038004_RIVERFRONT PARK BRIDGE FOUNDATION DESIGN\FIELD DATA\PERM_GINT\154038004.GPJ

GRAIN SIZE DISTRIBUTION

HARTCROWSER

CLIENT City of Milwaukie **PROJECT NAME** Riverfront Park Bridge Foundation Design

PROJECT NUMBER 154-038-004 **PROJECT LOCATION** Milwaukie, Oregon



BOREHOLE		DEPTH	Classification	LL	PL	PI	Cc	Cu
●	**B-1**	**4.5**	**SILT with fine sand, occasional gravel**					
⊠	**B-2**	**7.5**	**sandy LEAN CLAY, occasional gravel**					
▲	**B-2**	**10.0**	**sandy SILT**					
★	**B-2**	**15.0**	**silty SAND**					
⊙	**B-2**	**25.0**	**silty SAND, occasional gravel**					

BOREHOLE		DEPTH	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**B-1**	**4.5**	**76.2**				**0.9**	**27.4**	**71.7**	
⊠	**B-2**	**7.5**	**0.15**				**0.0**	**36.9**	**63.1**	
▲	**B-2**	**10.0**	**0.15**	**0.081**			**0.0**	**49.4**	**50.6**	
★	**B-2**	**15.0**	**0.15**	**0.084**			**0.0**	**52.8**	**47.2**	
⊙	**B-2**	**25.0**	**0.15**	**0.087**			**0.0**	**55.4**	**44.6**	

GRAIN SIZE - F:\GINT\OREGON_LIBRARY.GLB - 4/13/16 15:14 - F:\NOTEBOOKS\154038004_RIVERFRONT PARK BRIDGE FOUNDATION DESIGN\FIELD DATA\PERM_GINT\154038004.GPJ

Riverfront Park Bridge Foundation Design
Milwaukie, Oregon

Core - Boring HC-1 from 32.5 to 47.5 feet

154-038-004

04/16

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Figure

A-8

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Riverfront Park Bridge Foundation Design
Milwaukie, Oregon

Core - Boring HC-1 from 32 to 50 feet

154-038-004 | 04/16

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Figure

A-9

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Columbia West Engineering, Inc

UNCONFINED COMPRESSION REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Milwaukie Bridge Repair Oregon	Hart Crowser Jim Alders, PE 300 W. 15th Street, Suite 302 Vancouver, Washington 98660	11187O	S16-165
		REPORT DATE: 03/23/16	FIELD ID: n/a
		DATE SAMPLED: unknown	SAMPLED BY: client rep.

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	TEST PROCEDURE
basalt cored specimen	B-2, depth = 38 to 39 feet	ASTM D7012 Method C

LABORATORY TEST DATA

LABORATORY EQUIPMENT

Testmark Ind., CM-2500-LD, SN:18302

TEST SPECIMEN PREPARATION

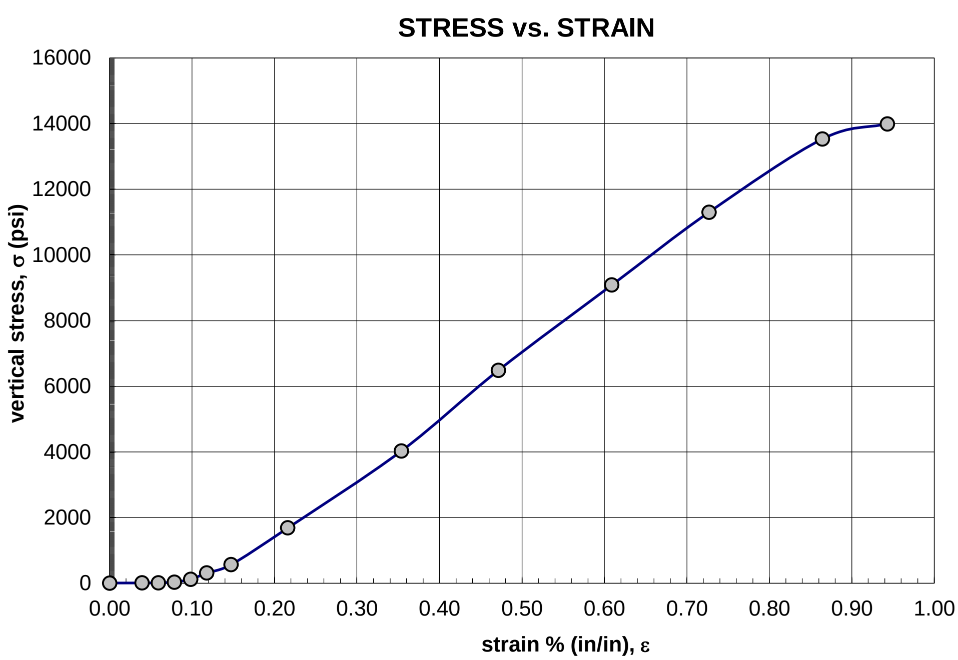
ASTM D4543; ends sawed; Procedures S1; approximate seating load = 30 lbs; as-received moisture condition

TEST DATA

unconfined compressive strength at peak (psi) =	13994.6
compressive strength at failure (psi) =	13994.6
time until failure (min) =	3.12
average strain rate until failure (in/min) =	0.015
vertical deformation at failure (in) =	0.048
percent strain at failure =	0.943

SAMPLE DATA

sample mass (g) =	1019.4
sample height (in) =	5.09
sample diameter (in) =	2.40
height-to-diameter ratio =	2.12
initial dry density (pcf) =	162.7
moisture content =	3.6%



FAILURE SKETCH OR PHOTO

NOTES

Specimen sampled by client representative and delivered to the Columbia West laboratory on March 22, 2016 for subsequent testing. Ends of the specimen were capped with sulfur mortar per ASTM C617 on March 22, 2016. Specimen was sealed in plastic wrap and stored in the temperature controlled laboratory until the time of test.

DATE TESTED	TESTED BY
03/22/16	JJC/JMR

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Columbia West Engineering, Inc

UNCONFINED COMPRESSION REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Milwaukie Bridge Repair Oregon	Hart Crowser Jim Alders, PE 300 W. 15th Street, Suite 302 Vancouver, Washington 98660	11187O	S16-166
		REPORT DATE: 03/23/16	FIELD ID: n/a
		DATE SAMPLED: unknown	SAMPLED BY: client rep.

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	TEST PROCEDURE
basalt cored specimen	B-2, depth = 44 to 45 feet	ASTM D7012 Method C

LABORATORY TEST DATA

LABORATORY EQUIPMENT

Testmark Ind., CM-2500-LD, SN:18302

TEST SPECIMEN PREPARATION

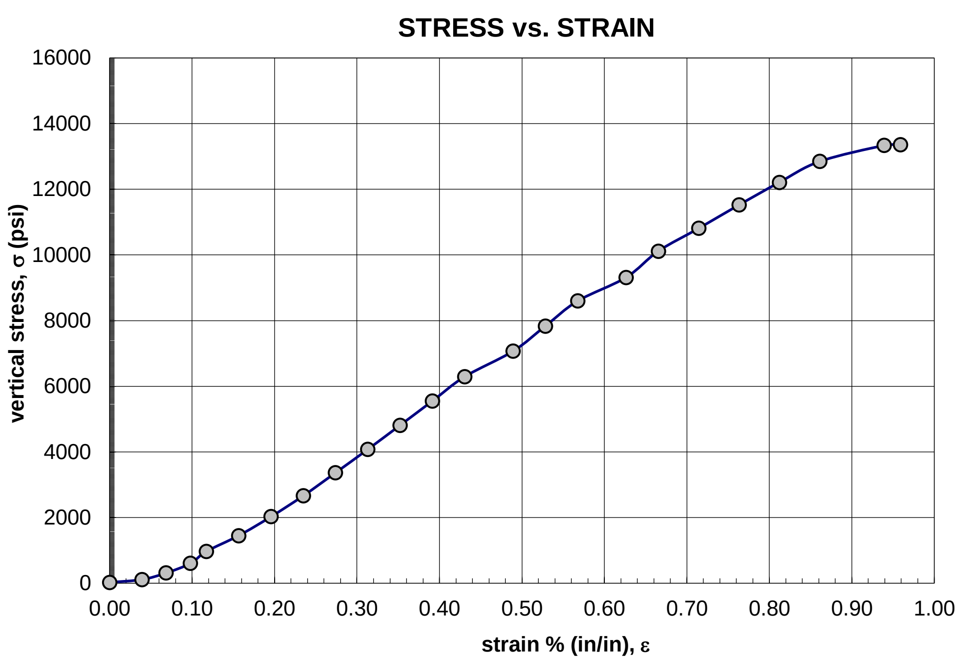
ASTM D4543; ends sawed; Procedures S1; approximate seating load = 110 lbs; as-received moisture condition

TEST DATA

unconfined compressive strength at peak (psi) =	13353.5
compressive strength at failure (psi) =	13353.5
time until failure (min) =	5.65
average strain rate until failure (in/min) =	0.009
vertical deformation at failure (in) =	0.049
percent strain at failure =	0.959

SAMPLE DATA

sample mass (g) =	1032.3
sample height (in) =	5.11
sample diameter (in) =	2.40
height-to-diameter ratio =	2.13
initial dry density (pcf) =	164.1
moisture content =	3.6%



FAILURE SKETCH OR PHOTO

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DATE TESTED	TESTED BY
03/22/16	JJC/JMR

COLUMBIA WEST ENGINEERING, INC. authorized signature