APPENDIX A

Shannon & Wilson, Inc., *Geotechnical Engineering Report – Riverfront Interceptor Sewer Lift Station and Force Main*, Albany, OR. April 2019

SUBMITTED TO:
West Yost Associates, Inc.
4949 Meadows Road, #125
Lake Oswego, Oregon 97035

BY:
Shannon & Wilson, Inc.
3990 Collins Way, Suite 100
Lake Oswego, OR 97035

503-210-4750
www.shannonwilson.com

GEOTECHNICAL ENGINEERING REPORT

Riverfront Interceptor Sewer Lift Station and Force Main

ALBANY, OREGON

April 2019

Shannon & Wilson No: 100623

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West Yost Associates, Inc.
4949 Meadows Road, #125
Lake Oswego, Oregon 97035

Attn: Mr. Matt Hewitt

RE: GEOTECHNICAL ENGINEERING REPORT, RIVERFRONT INTERCEPTOR SEWER LIFT STATION AND FORCE MAIN, ALBANY, OREGON

Shannon & Wilson participated in this project as a subconsultant to West Yost Associates, Inc. (West Yost). Our scope of services was specified in Task Order Number 8 executed on September 4, 2018.

Kevin Wood

Kevin Wood, PE
Senior Engineer

77330PE
OREGON
Jan. 10, 2006
ELLIOTT CARL MECHAM
EXPIRES: 4/11/2020

Jerry Jacksha

Jerry Jacksha, PE, GE
Senior Associate | Geotechnical Engineer

Elliott Mecham, PE
Associate

LIK/KJW/ECM/JLJ:/mmm/las

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1 INTRODUCTION

1.1 General

This geotechnical engineering report (GER) presents a summary of our literature research, field explorations, and laboratory test data compiled to support design and construction of the City of Albany Riverfront Interceptor Sewer Lift Station and Force Main. The interceptor sewer lift station and force main starts near the intersection of NE Montgomery Street and NE Water Avenue and runs to the intersection of Front Avenue NE and NE Davidson Street. The Vicinity Map, Figure 1, shows the general location of the proposed project. The City of Albany is the project owner and West Yost Associates (West Yost) is leading the project design. Shannon & Wilson, Inc. (Shannon & Wilson), is providing geotechnical engineering services for the project under a subconsultant agreement with West Yost.

1.2 Project Understanding

We understand that the project will include rerouting the existing gravity sewer pipeline north from its current alignment along NE Water Avenue into a new lift station and constructing a new force main leaving the lift station. The proposed lift station and start of the force main are shown on Figure 2, Site Plan.

The existing gravity flow sewer pipeline is proposed to be rerouted through a diversion structure and manhole into the westside of the proposed lift station. The lift station is proposed to be constructed near the Dave Clark Trail between NE Water Avenue and the Willamette River northeast of the intersection of NE Water Avenue and NE Montgomery Street.

We understand that the proposed lift station will include construction of a wet well, valve vault, and electrical (control) building. The wet well and valve vault are proposed to have a footprint of approximately 21 feet by 34 feet, and the control building is proposed to have a footprint of approximately 20 feet by 10 feet and is proposed to be approximately 67 feet west of the lift station.

Additionally, we understand that the base of the new wet well for the lift station will be constructed approximately 31 to 32 feet below the ground surface (bgs). The valve vault is proposed to be founded at a shallower depth than the wet well, approximately 8 feet bgs, resulting in a cantilevered configuration.

The proposed force main alignment is east along NE Water Avenue, turning north along NE Geary Street, then turning east at Front Avenue NE and ending at the intersection of Front Avenue and Davidson Street. The force main construction includes approximately 7,000 lineal feet of the new 21-inch-diameter force main.

The new force main is generally shallow (i.e. less than 10 feet bgs) but will include manholes that will extend up to approximately 20 feet bgs. We understand that much of the proposed pipeline construction will be performed using open cut trenching. This includes at least two open cut crossings of the existing rail adjacent to the portion of the alignment along NE Water Avenue.

1.3 Scope of Work

Shannon & Wilson's services were conducted in accordance with the Scope of Work defined in Task Order No. 8, which is a task order included in the Task Order agreement between West Yost Associates, Inc, and Shannon & Wilson Inc., dated May 12, 2008. The scope of services includes the following outline of activities, assessments, and recommendations:

- Review available existing information and visit the site to observe existing site conditions, geologic hazards, and site access for the field explorations; and mark proposed exploration locations;
- Explore the subsurface conditions with three geotechnical borings and collect soil samples;
- Install standpipe piezometers in each of the boreholes and perform hydraulic conductivity testing at one location;
- Conduct laboratory testing on selected soil samples to characterize soils and develop soil properties for evaluation;
- Prepare this Geotechnical Engineering Report including the following recommendations and construction considerations:
  - Evaluate Seismic Design Parameters;
  - Evaluate the stability of the slope directly to the north of the planned lift station;
  - Evaluate lateral earth pressures for below grade structures;
  - Evaluate the total and differential settlement of proposed lift station and manhole facilities;
  - Evaluate the potential for liquefaction-induced settlement and estimate the settlement from liquefaction, if liquefaction is predicted;
  - Provide recommendations for shallow foundations for the planned lift station including the control building, and manholes;
  - Evaluate conceptual excavation and shoring methods;

- Assessment of groundwater control, and conceptual methods to control water;
- Assessment of subgrade preparation, pipe bedding, trench backfill, and cut and fill slope requirements;
- Estimation of soil modulus (E′) in the pipe zone; and
- Provide recommendations for suitable structural backfill.

2 GEOLOGIC AND SEISMIC SETTING

2.1 Regional Geology

The project site lies in the Willamette Valley physiographic province. Today, the Willamette Valley is a broad alluvial plain bounded by the Columbia River on the north, the Coast Range on the west, and the Cascade Range on the east. Before it was a terrestrial valley, the region was a broad continental shelf, extending westward from the proto-Cascades into the ocean (Orr and others, 2000).

Around 50 million years ago, an oceanic island chain slowly collided with the coastline as the oceanic crust that carried it was subducting beneath the North American tectonic plate. This accreted island chain ultimately formed the Coast Range and the western boundary of the present-day Willamette Valley

Over its long history, the Willamette Valley region has collected vast amounts of sediment. Prior to becoming a true terrestrial valley, thousands of feet of Western Cascade sediments were deposited in the region in a marine setting (Orr and others, 2000). Once formation of the valley was complete and the sea retreated from the region, around 24 million years ago, terrestrial sediments began to collect, forming thick sequences of channel and overbank deposits.

More recently, the landscape was impacted by a series of glacial outburst floods. 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 glacial ice dam that created the lake would periodically fail, leading to 40 or more repetitive outburst floods at intervals of decades (Allen and others, 2009). Floodwaters washed across the Idaho panhandle, through eastern Washington, and through the Columbia River Gorge.

When the floodwater emerged from the western end of the gorge, it spread out over the Portland Basin and up the Willamette Valley, depositing a tremendous load of sediment

(Allen and others, 2009). In the southern Willamette Valley, the Missoula Flood sediments consist mostly of silt and clay and are referred to in many publications as Willamette Silt.

2.2 Site Geology

Surficial geologic units in the vicinity of the project site have been mapped by McClaughry and others (2010). According to mapping by McClaughry and others (2010), the project site is underlain by Reworked Willamette Silt, which is Missoula Flood sediment (predominantly silt and clay) that has been remobilized and deposited by local alluvial activity during the Holocene Epoch (within the last 10,000 years).

Beneath these small layers of Willamette Silt, the project site is mapped by McClaughry (2010) as alluvial terrace and fan deposits, also known as Linn Gravel. The Linn Gravel is an upper Pleistocene (0.01 to 1.8 million-year-old) stratified gravel and sand deposit that may be slightly older than or coeval with the Willamette Silt.

According to Wiley (2006), local alluvial activity during the Holocene also remobilized and deposited portions of the Linn Gravel in the project area, hence the term Reworked Linn Gravel.

Local topographic highs in the area are mapped as mixed source marine sedimentary rocks, which generally consists of middle Eocene (11.6 to 16.0 million-year-old) siliciclastic and volcaniclastic sandstone and siltstone which were deposited in marine basins of varying depth and geography. Wiley (2006) referred to this formation as Yamhill formation. Based on the mapping, the Yamhill Formation likely underlies the Reworked Willamette Silt and Reworked Linn Gravel throughout the project area.

For the purposes of this report, we refer to the Reworked Willamette Silt more generally as Fine-Grained Alluvium. We refer to the Linn Gravel and Reworked Linn Gravel more generally as Sand and Gravel Alluvium, and we retain the term Yamhill Formation for the underlying bedrock unit. In the time since these materials were deposited, portions of the site have been graded, and variable thicknesses of fill have been placed during the course of development.

3 SUBSURFACE EXPLORATIONS

The field exploration program for the riverfront sewer interceptor and force main included three geotechnical borings, designated B-1 through B-3, and one vacuum excavation designated Vac-1. Borings B-1 and B-2 were drilled on September 6 and September 7, 2018.

The excavation with the vacuum truck was performed on September 26 and 27, 2018. Finally, Boring B-3 was drilled on September 27, 2018.

Borings B-1, B-2, and B-3 were finished with 2-inch-diameter observation wells installed to depths of 29, 50, and 29 feet, respectively, to allow for ongoing groundwater level measurements. Measurements are presented in Section 5.2, Groundwater.

Details of the field explorations, including techniques used to advance and sample the borings and install the observation wells, are presented in Appendix A, Field Explorations. The purpose of the vacuum excavation was to determine the location and depth of the existing sewer pipe. As this was the case, and because it is impossible to collect representative samples from a vacuum truck, we did not collect samples or log the soils extracted. However, we did notice that soils extracted from the vacuum excavation appeared to be similar to those sampled in nearby borings B-2 and B-3.

4 LABORATORY TESTING

The samples we obtained during our field explorations were transported to our laboratory for further examination. We then selected representative samples for laboratory tests. The laboratory testing program included moisture content tests, Atterberg limits, grain size analyses, and laboratory testing for corrosion. Moisture contents, Atterberg limits, and grain size analyses tests were performed by Shannon & Wilson in accordance with applicable ASTM International (ASTM) standard test procedures. Results of the laboratory tests and brief descriptions of the test procedures are presented in Appendix B, Laboratory Test Results. Results are also presented graphically on the boring logs in Appendix A.

Laboratory testing for corrosion was subcontracted to TestAmerica Laboratories, Inc., and included testing for pH, Resistivity, Redox, Sulfides, Chlorides, and Sulfates. Results from the corrosivity testing are attached to Appendix B.

5 SUBSURFACE CONDITIONS

5.1 Geotechnical Units

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

- **Fill**: Medium Dense to Dense, Silty and Poorly-Graded Gravel (GP, GM); moist; angular to rounded gravel; fine to course sand; low plasticity fines; few pockets of fines; few pockets of charcoal.
- **Fine-Grained Missoula Flood Deposits**: Very stiff to hard, gray to dark gray, Lean Clay (CL); fine to medium sand; medium plasticity fines.
- **Reworked Linn Gravel**: Medium Dense to Very Dense, brown, tan, and red brown, Silty Gravel with Sand (GM) to Silty Sand with Gravel (SM); subangular to rounded gravel; fine to coarse sand; slight to moderate iron-oxide staining; trace highly weathered gravel clasts.
- **Yamhill Formation**: Medium stiff to Hard, Fat Clay (CH); trace fine to medium sand, high plasticity; trace fine organics.

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

5.2 Groundwater

Groundwater levels were not noted during drilling since the borings were drilled using a mud rotary drilling technique. This technique can make the depth to groundwater difficult to discern during drilling, due to the introduction of drilling fluids into the borehole to flush the drill cuttings to the surface.

However, groundwater wells consisting of 2-inch-diameter standpipe piezometers were installed in each of the borings B-1 through B-3. Each of the wells were developed prior to recording groundwater levels. During and after development of wells, groundwater levels at the project site were measured in the observation wells by Shannon & Wilson. The groundwater level measurements from the three wells installed at the site are presented in Exhibit 5.1, Groundwater Level Measurements in Observation Wells (below).

At the time of the development of the piezometer at B-3, no water flowed back into the well after draining it the first time. The water level at piezometer B-2 was at 32 feet, which is 3 feet below the bottom of the well at B-3. This led us to believe that perched water is not present at the location of these two wells at this time of year.

Exhibit 5-1: Groundwater Level Measurements in Observation Wells

Piezometer Reading Date	Piezometer B-1 Depth to Water (ft)	Piezometer B-2 Depth to Water (ft)	Piezometer B-3 Depth to Water (ft)
9/12/18	19	38	Not Installed
10/1/18	18	32	Dry
1/9/19	Dry	30	27

Groundwater levels should be expected to vary with changes in topography, precipitation, and the level of the Willamette River. Generally, groundwater highs occur at the end of the wet season in late spring or early summer, and groundwater lows occur towards the end of the dry season in the early to mid-fall.

5.2.1 Hydraulic Conductivity Testing

As part of our scope, we performed a hydraulic conductivity test through the well installed in boring B-01. The hydraulic conductivity test consisted of slug testing performed on September 12, 2018. The slug test provides an estimate of hydraulic conductivity for the water-bearing zones screened by the well. Results and a detailed discussion of the hydraulic conductivity data collection and analysis are presented in Appendix C.

6 SEISMIC GROUND MOTIONS AND GEOLOGIC HAZARD EVALUATION

We understand that the City has requested seismic design criteria in accordance with the American Society of Civil Engineer's (ASCE) Minimum Design Loads for Buildings and Other Structures, 2016 Edition (ASCE 7-16), which is based on earthquake ground motions with a 2,475-year return period. We also evaluated liquefaction triggering and liquefaction induced settlement for 475-year return period ground motions.

6.1 Seismic Ground Motions

ASCE 7-16 requires that geotechnical hazard analyses (liquefaction, specifically) be performed for Maximum Considered Earthquake Geometric Mean (MCEG) ground motions and adjusted for site class effects. Specifically, the peak ground acceleration used in the liquefaction-related hazard analyses, PGAM is defined as the following:

- PGAM = FPGA x PGA (ASCE 7-16 equation 11.8-1)

where:

- PGAM = MCEG peak ground acceleration adjusted for site class effects

- PGA = MCEG peak ground acceleration of site class B/C boundary conditions
- FPGA = Site coefficient from ASCE 7-16 Table 11.8-1

For this project, we calculated a PGAM of 0.46g using a PGA of 0.38g and an FPGA of 1.22. PGA is shown in ASCE 7-16 Figure 22-9 and is derived from the most recent U.S. Geological Survey (USGS) National Seismic Hazard Mapping Project ground motion hazard analyses results by Petersen and others (2014). FPGA is a function of site class and PGA as indicated in ASCE 7-16 Table 11.8-1. The SPT N-value resistances measured in the borings correspond to Site Class D. Seismic design parameters based on the recommended Site Class D are presented in Exhibit 6-1.

Exhibit 6-1: USGS Code-Based MCE and Seismic Design Parameters for Site Class D

Seismic Parameters	Value
MCE Peak Bedrock Acceleration (PGA)	0.379g
MCE Bedrock Spectral Acceleration, 0.2 second period (SS)	0.807g
MCE Bedrock Spectral Acceleration, 1.0 second period (S1)	0.423g
Short-Period Site Factor, Fa	1.177
Long-Period Site Factor, Fv	1.877
Soil MCE Spectral Acceleration, 0.2 second period, Site Class D (SMS)	0.95g
Soil MCE Spectral Acceleration, 1.0 second period, Site Class D (SM1)	0.794g
Soil Peak Ground Acceleration (PGAM)	0.463g
Soil Design Spectral Acceleration, 0.2 second period, Site Class D (SDS)	0.633g
Soil Design Spectral Acceleration, 1.0 second period, Site Class D (SD1)	0.529g

Note:

PGA stands for Peak Ground Acceleration, which corresponds to spectral acceleration at zero second.

Because the maximum earthquake magnitudes for sources vary significantly, we used a probabilistically-determined mean moment magnitude of 8.2 for ground motions with a 2,475-year return period for analyses requiring magnitude (i.e. liquefaction).

6.2 Liquefaction

Liquefaction is a phenomenon in which excess pore water pressure in loose to medium dense, saturated, nonplastic to low plasticity silts and granular soils develops during ground shaking. The increase in excess pore pressure may result in a reduction of soil shear strength and a quicksand-like condition.

Important factors in evaluating a soil’s susceptibility to liquefaction include relative density, the fines content (percent of soil by weight smaller than 0.075 millimeter, passing the No. 200 sieve), and the plasticity characteristics of the fines. Relative density can be estimated from SPT N-values that were performed for this project. We performed laboratory Atterberg limits testing to evaluate the plasticity of the site soils.

6.2.1 Screening

We conducted a preliminary screening for liquefiable soils based on the Bray and Sancio (2006) criteria, which suggests that soils with plasticity indices (PI values shown in Appendix B) below 12 with a natural moisture content greater than 0.85 times the liquid limit are potentially liquefiable and using the Boulanger and Idriss (2006) method, which provides recommendations that the fine-grained soils with plasticity index greater than seven would not be liquefiable.

Based on review of the explorations and laboratory testing, our screening indicates that the fill, fine grained deposits, and gravel alluvium deposits have plasticity indices less than 12 and are susceptible to liquefaction according to this Bray and Sancio (2006) soil plasticity criteria; however, these materials are above the water table and, therefore, are not considered liquefiable. The clay soils below the water table have plasticity indices much higher than 12 and are, therefore, considered non-liquefiable. It is our opinion that the potential for liquefaction to occur at this site is low.

6.3 Lateral Spreading

Lateral spreading hazards can exist in areas with mild slopes adjacent to a much steeper slope or vertical face. Lateral spreading failure can occur if soil liquefaction develops during a seismic event and the ground acceleration (inertial force) briefly surpasses the yield acceleration (shear strength) of the liquefied soil. This can cause both the liquefied soil and an overlying non-liquefied crust of soil to displace laterally down mild slopes or towards an embankment face. The displacements are cumulative and permanent in nature.

The proposed interceptor is located about 40 feet from the sloping banks of the Willamette River. However, due to the low potential for liquefaction to occur at this site, it is our opinion that there is also a low risk of lateral spread towards the Willamette River at the location of the proposed lift station.

6.4 Slope Stability

We performed slope stability analysis at one cross-section that runs from the railroad, below the planned lift station to the Willamette River, based on available topographic information provided by the City of Albany from their own database, our subsurface explorations, and laboratory testing. The section at the west end of the alignment near the existing lift station and near boring B-2 is designated Section A-A', as shown on Figure 2, Site and Exploration Plan.

6.4.1 Approach

Slope stability is influenced by various factors, including the following: (1) the geometry of the soil mass and subsurface materials; (2) the weight of soil materials overlying a potential failure surface; (3) the shear strength of soils and/or rock along a potential failure surface; and (4) the hydrostatic pressure (groundwater levels) present within the soil mass and along a potential failure surface.

The stability of a slope can be expressed in terms of a factor of safety, which is defined as the ratio of resisting forces to driving forces. At equilibrium, the factor of safety is equal to 1.0, and the driving forces are balanced by the resisting forces. Slope movement is predicted when the driving forces exceed the resisting forces, i.e., the factor of safety is less than 1.0.

An increase in the factor of safety greater than 1.0, whether by increasing the resisting forces or decreasing the driving forces, reflects a corresponding increase in the stability of the mass. The actual factor of safety may differ from the calculated factor of safety, due to variations or uncertainty in the soil strength, subsurface geometry, potential failure surface location and orientation, groundwater level, and other factors that are not completely known.

Shannon & Wilson performed slope stability analyses using the computer program SLOPE/W, Version 9.1.0.16306 (Geo Slope International, 2018). The Morgenstern-Price method was used for rotational and irregular surface failure mechanisms. We utilized information from the closest explorations and laboratory testing to estimate material strength and unit weight parameters for the geologic units assumed to underlie the slope. Specifically, strength correlations based on SPT N-values were used. The soil properties for the geotechnical units defined in each analysis are included on the respective slope stability figure (Figures 3 and 4).

The slope stability at the cross-section was evaluated for static and seismic conditions. Post seismic conditions (liquefied soil) were not considered as we do not predict liquefaction occurring at this site. See discussions of these various conditions below and Exhibit 6-2 for tabulations of the results of our slope stability analysis.

6.4.1.1 Static

For slopes adjacent to essential facilities, a minimum factor of safety of 1.5 is recommended for the static condition.

6.4.1.2 Seismic

A minimum factor of safety of 1.1 is recommended for the seismic case. Shannon & Wilson performed pseudo-static analyses to evaluate the seismic slope stability using a horizontal seismic coefficient of 0.232, which is equal to one-half of the PGAM. If the factor of safety of the critical failure surface near the planned structure was less than 1.1, potential displacements were estimated by following the procedures in the National Cooperative Highway Research Program (NCHRP) document NCHRP 611 (NCHRP, 2008).

6.4.2 Results of the Slope Stability Analyses

We evaluated the stability of the proposed lift station at the cross-sections for static and seismic conditions (see Figures 3 and 4). Based on our analysis, the proposed lift station location satisfies the minimum slope stability factor of safety requirements for the cross-section in the static and seismic conditions.

Near the crest of the slope (i.e. closer than 10 feet), the critical factor of safety is approximately 1.3 and 0.95 for the static and seismic conditions, respectively. However, the lift station is set back approximately 30 feet from the crest of the slope. The slope stability results at a distance of approximately 30 feet from the crest of the slope are summarized in Exhibit 6-2.

Exhibit 6-2: Summary of Lift Station Slope Stability Results

Stability Section	Condition	Factor of Safety
A-A'	Static	1.82
	Seismic	1.13

The bank of the Willamette River is densely vegetated between the proposed lift station and the river. We did not observe erosion occurring directly adjacent to the planned project site. However, it should be noted that if riverbank erosion occurs, the overall factor of safety against failure decreases and the factors of safety presented in Exhibit 6-2 may not be representative.

We recommend the civil design team consider the risk of riverbank erosion. If erosion becomes an issue adjacent to the project site in the future, the shallow vault and control building are the portions of the overall structure most at risk. Mitigation of this risk could consist of deep foundation elements (i.e. micropiles) that are connected (tied) into the shallow foundation elements recommended in Section 7.4.

6.5 Fault Rupture

According to the quaternary faults and folds database, the Owl Creek fault is the closest fault to the site and is mapped 5.5 miles from the proposed alignment. Also, the slip-rate is less than 0.2 mm/year. Therefore, it is our opinion that the potential for a hazard posed by ground surface fault rupture at the site is low.

7 BURIED PIPELINE AND LIFT STATION DESIGN RECOMMENDATIONS

7.1 Modulus of Soil Reaction for Flexible Pipe

The modulus of soil reaction, E′, for flexible pipeline design, characterizes the stiffness of the pipe zone backfill placed at the sides of buried flexible pipelines. E′ is an empirical parameter (Spangler's Iowa formula) that is dependent on the deflection and the pressure developed at the spring line of the pipe. Variables also depend on the depth of the pipe, the type and density of the backfill, the thickness of compacted pipe zone backfill between the pipe and the trench wall, and the type of native soil. Shannon & Wilson understands this "composite" E′ that considers the variables described above, will be developed by the West Yost design team.

Based on Table 6 from the U.S. Department of the Interior Bureau of Reclamation Manual 25, 2nd Edition (U.S. Bureau of Reclamation, 2013), and the relative consistency (density) of the soils encountered in the field explorations, Shannon & Wilson recommends an E′ value of 1,500 psi for the native Linn Gravel or in situ reworked Linn Gravel fill. This value should be used to determine a composite E′ based on the variables described above.

At two locations, we understand that the pipe will be installed beneath the existing rail using open-cut, shored trenches. Therefore, we want to recommend that additional loading due to the railroad live load and dead load, if applicable based on rail operations, be incorporated by the West Yost design team when calculating pipe deflections.

7.2 Bedding Pipe Zone and Trench Backfill

7.2.1 Bedding

The pipe bedding zone in the trench should be constructed with imported, well-graded, clean crushed rock material suitable for compaction and allowing for flexible joints. The on-site excavation spoils will be predominantly silty gravel, and silty sand, with fines being non-plastic to low-plasticity silts that are not suitable for use as bedding material. The bedding material should consist of imported, 3/4-inch minus crushed aggregate, as specified in Oregon Standard Specification for Construction (OSSC 2018), Item 00405.12, Bedding.

Provided that the subgrade soil is competent and is not disturbed by the excavation equipment, the minimum thickness of granular bedding below the invert of the pipeline should be a minimum of 4 inches per City of Albany requirements for pipes less than 27 inches in diameter. In areas where wet, weak, or disturbed subgrade conditions are encountered, the required subgrade stabilization (subgrade overexcavation/replacement) will likely result in thicker pipe bedding.

It is anticipated that the subgrade soils will contain gravel and cobbles up to at least 12 inches in diameter. As such, over-excavation may need to extend more than 4 inches below the planned pipe bedding depth in localized areas. The pipe bedding should consist of crushed aggregate, with less than 5 percent by dry weight passing a U.S. Standard No. 200 Sieve, and it should meet OSSC 2018 00405.14 (Class B Backfill).

Pipe zone compaction should be at least 90 percent of maximum density, as determined by a proctor, conforming to ASTM D1557. Where testing is not possible due to the proximity of the pipe or trench walls, materials must be compacted to a firm and unyielding state as determined by the engineer or the engineer's representative.

Based on groundwater levels from the installed piezometers, we do not believe groundwater will be encountered during the construction of the force main. However, groundwater levels could fluctuate, and should either groundwater or perched water be encountered, or if water enters the trench from subsurface water traveling along other buried pipelines in the area, we recommend installation of a crushed rock drainage layer at least 12 inches thick. The drainage layer should be installed below the pipe bedding to facilitate sump pumping within the trench. It should be constructed with open, free-draining crushed rock materials with a 1-1/2- to 3/4-inch gradation, conforming to Oregon Standard Specifications for Construction (OSSC 2018, 00430.11).

The crushed rock for the working mat/drainage system should also have less than 2 percent by weight passing the No. 200 wet sieve; and 90 percent of particles by weight retained on the U.S. No. 4 sieve should have at least two fractured faces. In areas where the drainage rock described above is used, the material may also serve as the pipe bedding, depending on requirements of pipe material and joints.

7.2.2 Pipe Zone

For the pipe zone material, bedding material specified in OSSC 2018, Item 00405.13, should be used for flexible pipes. Pipe zone materials should extend at least 12 inches above the top of the pipe, per City of Albany code, or more as determined by the manufacturer. Pipe zone compaction should be at least 90 percent of maximum dry density, as determined by a proctor, conforming to ASTM D1557.

7.2.3 Trench Backfill

Above the pipe zone, the pipelines and buried structures can be backfilled with select native material. The gravel alluvium soils encountered at the site during our subsurface investigation program are generally suitable for placement as trench backfill during warm, dry weather when moisture contents can be maintained by air drying and/or addition of water. The moisture content of the near-surface soils can be expected to vary depending on the time of year and recent weather conditions.

Select native backfill consisting of the gravel alluvium in non-settlement-sensitive areas should be compacted to a minimum of 90 percent of maximum density, as determined by a proctor conforming to ASTM D1557. Where testing is difficult, or not possible, the trench backfill material must be compacted to a firm and unyielding state, as determined by the engineer or the engineer’s representative. The material must be inspected by the geotechnical engineer of record before reuse. Select native backfill material must not be placed within 18 inches of the ground surface.

In locations where trench backfill is placed in settlement-sensitive areas and for the final 18-inches below roadway or structural elements, we recommend the use of 3/4-inch minus crushed aggregate, with less than 5 percent by dry weight passing a U.S. Standard No. 200 Sieve, and it should meet OSSC 2018 00405.14 (Class B Backfill). The backfill above the pipe zone should be compacted to 92 percent of the maximum dry density as determined by ASTM D1557. Along NE Water Avenue, where the pipe backfill is being placed near the Burlington Northern Santa Fe (BNSF) tracks, the pipe backfill requirements will be controlled by the joint BNSF (Burlington Northern Santa Fe) and UP (Union Pacific) Guidelines for Temporary Shoring (2004), which specifies that backfill be compacted to 95 percent of the maximum dry density as determined by ASTM D1557.

7.3 Lateral Earth Pressures

Due to the limited working space, we recommend that the temporary shoring used to support the excavation for the proposed lift station, which includes the wet well and valve vault, consist of drilled-in and grouted soldier piles with steel sheet lagging and internal bracing. Further discussion of the recommended temporary shoring for the lift station facility can be found in Section 8.4.1.

The lateral earth pressures for temporary braced shoring are shown on Figure 5. In our analysis for temporary lift station shoring, we assumed that the temporary braced shoring will be against the native soil with a level backfill surface, with one or multiple levels of bracing, and will be designed with active earth pressure conditions.

We anticipate that the lateral earth pressures on the permanent manholes and the lift station will be from native soil outside the shoring system and imported crushed rock or gravel backfill against the concrete walls; sand should not be used as backfill around the structures. We also anticipate the structures will be designed for at-rest conditions.

The lateral earth pressures on embedded walls for manholes and the embedded portions of the lift station (i.e. wet well and valve vault) are shown on Figure 6. In our analysis for permanent embedded structures, we assumed that the walls will be designed as non-yielding walls with a level backfill surface.

Figures 5 and 6 present the typical earth pressure distribution based on the surcharge load from a train parallel to the proposed shoring element or permanent embedded structure. The live load surcharge is calculated by taking the weight of the train (80,000 lb) and dividing it by the distance between axels (5 feet for a Cooper E80) and the length of the rail ties (typically about 8 to 9 feet).

Lateral earth pressures for temporary shoring systems along the pipeline should be developed by the Contactor's professional engineer licensed in the State of Oregon in accordance with joint BNSF (Burlington Northern Santa Fe) and UP (Union Pacific) Guidelines for Temporary Shoring (2004). The Contractor's engineer should base the lateral earth pressures for temporary shoring on a sufficient number of borings along the trench excavation to determine with a reasonable degree of certainty, the subsurface conditions as described in AREMA 8.5.2 (2018). Geotechnical explorations were performed near the beginning of the alignment near the location of the lift station. No explorations were performed along the remaining approximately 4,880 feet of the proposed pipeline alignment along Water Avenue and parallel to railroad alignment. In our opinion, the Contractor's engineer should consider performing a minimum of an additional four explorations, approximately evenly spaced and to a minimum depth of 15 feet below bottom of trench. The shoring system must be designed such that horizontal deflection of the shoring system and top of rail elevation do not exceed the deflection criteria outlined in the BNSF & UP Guidelines for Temporary Shoring. These Deflection Criteria from the Guidelines are reproduced below for reference as Exhibit 7-3: Deflection Criteria. There are other guidelines related to shoring, monitoring and construction contingency plans for corrective action not described in this report that also should be followed by the Contractor.

Exhibit 7-3: Deflection Criteria

Horizontal distance from shoring to track C/L measured at a right angle from the track	Maximum horizontal movement of shoring system	Maximum acceptable horizontal or vertical movement of rail
12'<S<18'	3/8"	¼"
18'<S<24'	1/2"	¼"

7.4 Foundation Recommendations

7.4.1 Manhole Foundations

Based on our estimates of the depth of material, manholes may be placed on crushed rock over firm native gravel alluvium. The footprint of the over-excavation should extend a minimum of 6 inches outside the edge of the structure and 6 inches below the structure subgrade. The over-excavated material should be replaced with an engineered 3/4-inch minus crushed rock fill consisting of imported crushed rock. With this subgrade preparation and crushed rock layer, a subgrade modulus of 200 pci may be used for foundations.

If the recommended crushed rock fills are constructed as described above, the proposed manholes can be supported on conventional shallow foundations founded on the crushed rock mat with a net allowable bearing capacity of 4,000 psf. A total static settlement of less than 1/2-inch and a differential settlement on the order of 50 percent of the total settlement are estimated, with the proposed structures supported on the crushed rock layer. Our settlement estimate assumes that no disturbance to the foundation soil subgrade would be permitted during excavation and fill placement.

7.4.2 Wet Well Foundation

Based on the information received from West Yost via email on October 8, 2018, the top of the proposed slab for the lift station wet well will be approximately 31 feet below the existing ground surface at an approximate elevation of 173 feet. Assuming a concrete slab (mat) thickness of 1 foot and a combined thickness of an additional 1.5 feet for leveling course (6 inches) and drainage layer (12 inches), the resulting excavation would be about 32.5 feet below existing grade, which is about an elevation of 171.5 feet.

The footprint of the over excavation should extend 1 foot outside the edge of the wet well and 1.5 feet below the structure subgrade. The over-excavated material should be replaced with 6 inches of an engineered 3/4-inch minus crushed rock fill underlain by 12 inches of an engineered free-draining, crushed rock fill underlain by a layer of non-woven geotextile fabric. With this subgrade preparation and crushed rock layer, a subgrade modulus of 150 pci may be used for foundations.

If the recommended crushed rock fills are constructed as described above, the proposed wet well for the lift station can be supported on conventional shallow foundations founded on the crushed rock mat drainage/working mat with a net allowable bearing capacity of 3,000 psf.

A total static settlement of less than 1 inch and an estimated differential settlement less than 1/2-inch is estimated for the wet well supported on the crushed rock layer. Our settlement estimate assumes that no disturbance to the foundation soil subgrade would be permitted during excavation and fill placement.

7.4.3 Valve Vault Foundation

We recommend that the valve vault adjacent to the north side of the wet well be encompassed by the temporary shoring used to construct the wet well. If the valve vault is encompassed by the temporary shoring for the wet well construction, then we would anticipate that the valve vault would be founded on compacted crushed rock backfill.

With this subgrade preparation and crushed rock layer, a subgrade modulus of 150 pci may be used for foundations. If the valve vault is founded on compacted crushed rock fill, then a net allowable bearing capacity of 3,000 psf can be used for design. A total static settlement of less than 1 inch and an estimated differential settlement of less than 1/2-inch is estimated for the valve vault constructed on the crushed rock backfill layers.

7.4.4 Control Building Foundation

We anticipate that the control building will be constructed on typical shallow continuous footings with a minimum footing depth of approximately 18 inches and an interior slab-on-grade. Also, if the control building is directly adjacent to the eastside of the wet well, then a portion of the control building footprint will be over the crushed rock backfill placed during construction of the wet well and valve vault and a portion would be on in situ material. However, based on our explorations, the shallow (i.e. less than 5 to 7 feet) in situ material at this site is undocumented fill. Therefore, we recommend overexcavating approximately 2 to 3 feet into the native in situ material and replacing with compacted crushed rock backfill.

If the control building subgrade is overexcavated 2 to 3 feet, then a net allowable bearing pressure of 2,000 psf can be used for design. With this subgrade preparation and crushed rock layer, a subgrade modulus of 100 pci may be used for design. A total static settlement of less than 1 inch and an estimated differential settlement of less than 1/2-inch is estimated for the control building founded on the crushed rock backfill. Our settlement estimate assumes that no disturbance to the foundation soil subgrade would be permitted during excavation and fill placement.

8 CONSTRUCTION CONSIDERATIONS

8.1 Groundwater Control

As discussed in "Section 5.2 Groundwater," groundwater in boring B-1 located near the intersection of NE Davidson Street and Front Avenue NE was measured at 19 feet below ground surface on September 12, 2018, and 18 feet below ground surface on October 1, 2018. The depth to groundwater is below depth to the pipe invert of approximately 13 feet below ground surface (El 193.5 feet) and within 1 foot of the depth to the bottom of the manhole, which is approximately 19 feet below ground surface (El 198.89) during fall of 2018.

We anticipate that dewatering of any perched water along the pipeline can be performed using well filtered sumps. At the manhole near NE Davidson Street and Front Avenue NE, dewatering of depths of up to 3 to 4 feet can also be performed using well filtered sumps. If construction of the manhole is performed during a period of extended wet weather and more than 4 feet of drawdown is required, dewatering systems external to the trench such as vacuum extraction well points or deep wells should be used.

At boring B-2, groundwater was observed at a depth of approximately 32 feet below the ground surface in October 2018, and no ground water was observed in the well installed at the interface of the upper gravels and stiff clays, at a depth of 29 feet below ground surface in boring B-2.

While no perched water was observed on top of the stiff clay during our exploration, perched water may be present after periods of extended rainfall. The total depth of excavation for the lift station will be approximately 33.5 feet.

Due to the presence of fine-grained soils (and due to several laboratory tests exhibiting high plasticity) we anticipate that groundwater, perched water, and seepage within the deep excavation could be controlled with pumping from localized, well-constructed, filtered sumps, provided the bottom of trench excavation and permanent structure excavations are less than an estimated 4 feet below the groundwater level.

Due to the presence of the fine-grained soils at the base of the lift station wet well, excavations of less than 4 feet below the groundwater table can be made without the need for dewatering systems (external to the trench) such as vacuum extraction well points or deep wells.

8.2 Wet Weather Construction

Excavation and construction operations may expose the on-site soils that are sensitive to inclement weather conditions. The stability of exposed soils may rapidly deteriorate due to a change in moisture content (i.e. wetting or drying) or the action of heavy or repeated construction traffic. Accordingly, excavations should be adequately protected from the elements and from the action of repetitive or heavy construction loadings.

8.3 Temporary Pipeline Excavation Stability

Most of the pipeline excavation will be performed along an existing city street running parallel and adjacent to an existing railroad. Shoring along this section must be used to mitigate the risk of ground movement adjacent to the railroad, pavements, utilities, and other settlement sensitive structures. The shoring system selection, design, installation, monitoring and any corrective actions needed should be the responsibility of the Contractor.

Along the railroad, the shoring must conform to the requirements outlined in the BNSF (Burlington Northern Santa Fe) and UP (Union Pacific) Guidelines for Temporary Shoring, including being capable of limiting the deflections to the requirements presented in the above Exhibit 7-3, Deflection Criteria, and capable of penetrating gravels and cobbles known to exist in the subsurface of the project area. Trench boxes for shoring are not allowed under the BNSF and UP Guidelines for Temporary Shoring. Systems such as sheet piles, Slide Rail, and Shore-Trac may not be capable of penetrating soil formations containing cobbles.

Considering the criteria from the BNSF and UP shoring guidelines mentioned above and for other excavations adjacent to existing buried facilities, we recommend utilizing positive, laterally restrained shoring systems to provide full-time lateral support to the trench sidewalls during the trench excavation, pipe installation, backfilling, and compaction of the trench pipeline and backfill materials.

In addition to the above requirements, all excavations and shoring requirements should be in accordance with OSHA and state and local requirements. Based on the subsurface conditions in the project area, the soil encountered in the excavations could be classified as OSHA Type C soil. The Contractor should be aware of, and familiar with, applicable local, state, and federal safety regulations, including the current OSHA Excavation and Trench

Safety Standards. Site safety generally is the sole responsibility of the Contractor, who also is solely responsible for the means, methods, and sequencing of construction operations.

We are providing the above information and opinions solely as a service to our client. Under no circumstances should the information provided and opinions expressed above be interpreted to mean that Shannon & Wilson is assuming responsibility for construction site safety or the Contractor's activities; such responsibility is not being implied and should not be inferred.

8.4 Lift Station and Sewer Tie-In Excavation and Backfill

The shoring system selection, design, installation, monitoring and any corrective actions should be the responsibility of the Contractor. Due to the proximity of the proposed lift station excavations to the Willamette River to the north, privately owned property to the east, and the railroad and NE Water Avenue to the south, an excavation per OSHA requirements for a type C soil with a temporary slope of 1.5H:1V would become unfeasible due to the approximate 33.5-foot excavation depth. Therefore, for the shoring system consideration, we recommend positive, laterally restrained shoring systems to provide full-time lateral support to the excavation sidewalls, designed by the Contractor, such as a temporary cased, drilled-in and socketed soldier pile and steel sheet lagging system with interior bracing.

As previously mentioned, we recommend that the shoring system surround the entire wet well and valve vault excavations. Lateral earth pressures for a typical braced excavation are shown on Figure 5; however, the final earth pressure design should be the responsibility of the Contractor's design engineer. Note, "driven" soldier piles and sheet piles should not be used based on the subsurface conditions at the lift station site and the close proximity to the riverbank slope, as described below.

8.4.1 Temporary Shoring for the Lift station and Manhole Tie-In

Temporary shoring systems for relatively deep excavations, such as those required for the lift station, typically consist of tieback walls, deadman walls, interlocked steel sheet pile, or interior brace cantilever walls. However, a sheet pile system will likely have significant difficulty penetrating the very dense gravels and very stiff clayey soils, like those found in the sand and gravel alluvium and in the Yamhill formation, to sufficient depth to support an excavation. Further, vibrations generated during installation and retrieval of sheet piles may cause the adjacent railroad embankment to become unstable. Likewise, we recommend against allowing "driven" soldier piles as described below.

Tiebacks and deadman anchors act the same way in that they are elements that connect to the driven piles and provide support to the temporary shoring by adding a tensile load against the pile from pull resistance in the soil.

In order for these kinds of systems to work, there needs to be sufficient length of soil for the tieback or deadman in order for the elements to provide enough tensile strength. Due to the close proximity to the Willamette River, we do not anticipate there is an adequate amount of soil for providing that strength.

Therefore, instead of sheet piles or soldier piles with tiebacks, we recommend the use of a full-height drilled-in soldier piles and lagging system, with steel sheets as lagging and with interior bracing for support. We also recommend that an outer temporary steel casing be used to install the drilled-in soldier piles.

In our opinion, this system is only feasible to support the full height of this deep excavation if steel sheet lagging is used instead of timber lagging which is sometimes used. Due to the granular nature of the soil, raveling and soil loss is anticipated with installation of conventional wood lagging, which requires excavation to stand unsupported while each piece of lagging is installed.

The advantage of steel sheet lagging is they remain in contact with the soils they are retaining while they are pushed down between the flanges of the soldier piles into the base of the excavation. We recommend that steel sheeting also be considered where the new sewer will tie into the existing sewer and two new manholes will be constructed to redirect the flow provided the contract is capable of advancing the soldier piles and sheeting to the top of ground surface at the end of each day. Any shoring system implemented near the two manholes will be required to be advanced below the ground surface each night, such that train traffic can advance over the shoring without interference from any above grade elements of the temporary shoring.

8.4.2 Soldier Piles and Steel Sheeting Shoring Preliminary Design Values

A soldier pile wall is a construction technique that uses vertical steel piles with lagging between piles to retain soil. In some cases, soldier piles (H-piles) are driven or vibrated in at regular intervals along the excavation perimeter. However, due to the presence of dense gravels, we recommend the piles be drilled in with temporary outer casings.

For pile backfilling, the portion of the pile below the excavation (supporting zone) should be backfilled with high strength concrete up to the same elevation as the planned excavation depth, then above the excavation where the steel sheets are installed, the piles are backfilled with very low strength grout that allows the steel sheets to be installed. The design and detailed means and methods of this soldier pile system should be the responsibility of the Contractor.

A soldier pile shoring system should be designed using the typical lateral earth pressures provided on Figure 5; however, the final earth pressure design should be the responsibility of the Contractor's design engineer. The lateral earth pressures presented on Figure 5 are unfactored. Based on our experience, settlement on the order of 1 inch can be expected adjacent to braced shoring for walls up to 25 feet tall. We anticipate that the settlement will become negligible at a distance of approximately 25 feet from the wall. The risk of settlement can be mitigated by maintaining constant contact between the steel sheeting and the soils retained behind the gravel excavation.

Structural design of the soldier piles should consider the lateral earth pressures discussed above. The piles can derive the vertical- and lateral-load-carrying support from the underlying dense gravel and very stiff clay. We recommend an allowable skin friction of 1 kips per foot between the concrete and surrounding gravel and very stiff clay (Yamhill Formation) and an allowable end-bearing pressure of 10 kips per foot on the gravel and very stiff clay. In addition, we recommend that the grout or concrete at the tip of the pile have sufficient strength to withstand the imposed loads. These values should be verified by the Contractor's structural engineer designing the shoring. Concrete backfill should be placed using tremie pipe methods.

8.4.3 Soldier Piles and Steel Sheeting Installation Considerations

We anticipate there will be some difficult pile drilling conditions in the very dense gravel. After installation of the soldier piles, we recommend prompt and careful installation of lagging to maintain the integrity of the excavation, particularly in areas of raveling granular soil. Due to the close proximity of the adjacent railroad, and the embankment of the Willamette River, we recommend against any unsupported exposed excavation faces during wall construction.

Until the inside soldier pile system bracing struts are installed and loaded, the system will need to develop lateral capacity from embedment of the soldier piles. The design should consider deformations for the cantilever condition of the shoring prior to the installation of bracing. A total allowable passive pressure of 375 pounds per cubic foot (pcf) can be used for embedment into gravel, applied over 3 pile diameters.

8.4.4 Shoring System Backfill and Abandonment

As mentioned in Section 7.3, the backfill between the soldier pile system and the buried structures should be compacted, imported crushed rock or gravel; sand should not be used as backfill material. This material is needed primarily to support the foundation systems for the shallow valve vault and the at-grade control building adjacent to the lift station wet well; however, this type of material also reduces the lateral pressures on the buried structures.

We recommend the use of 1-1/2-inch minus crushed rock or gravel, with less than 7 percent by dry weight passing a U.S. Standard No. 200 Sieve, and it should meet the OSSC 2018 02640 Shoulder Aggregate requirements, except that sand shall not be allowed. The backfill should be compacted to a minimum of 95 percent of the maximum dry density, as determined by ASTM D1557. Since hand-compaction equipment will likely be used in this confined space, we recommend maximum loose lifts of 6 to 8 inches will be required to obtain the minimum compaction requirements.

As the backfilling proceeds, we anticipate the solder pile system steel sheet lagging will be extracted in stages. However, the steel sheets should always remain at least a minimum of 5 feet below the surface of the compacted backfill surface. We anticipate the soldier piles and the internal bracing will be abandoned in place due to the requirement to maintain lateral support as the sheets are being removed and backfill is being placed. We recommend the top 4 feet of the soldier piles below the final grade be cut off and the lateral bracing also remain 4 feet below the final grade.

8.5 Erosion Control

Erosion of the soil at the site will occur as exposed surfaces are disturbed due to construction activities and exposure to climatic conditions. Excavated surfaces should be protected by a weather-resistant cover or erosion-control product, if left exposed. Temporary erosion and runoff control measures should be in place prior to and during construction. Erosion-control measures should remain in place and be maintained by the Contractor until disturbed areas are stabilized. The expected erosion control work consists of furnishing, installing, maintaining, removing, and disposing of water sediments and should be executed in accordance with OSSC, Section 00280.

9 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, Shannon & Wilson should be advised at once so that these conditions can be reviewed, and the recommendations reconsidered, 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, it is recommended that Shannon & Wilson review this report to determine the applicability of the conclusions and recommendations.

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. Shannon & Wilson makes no other warranty, either express or implied. These conclusions and recommendations were based on Shannon & Wilson's 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 West Yost and the City of Albany for the Riverfront Interceptor Sewer Lift Station and Force Main. This report contains interpretations and conclusions and recommendations for West Yost and the City of Albany, it should be provided to the Contractors for their information or reference only and not as a basis of Contractor bidding, and evaluation of differing conditions during construction. Also, since this report contains interpretations and conclusions, it should not be construed as a warranty of subsurface conditions.

The scope of Shannon & Wilson's 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.

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

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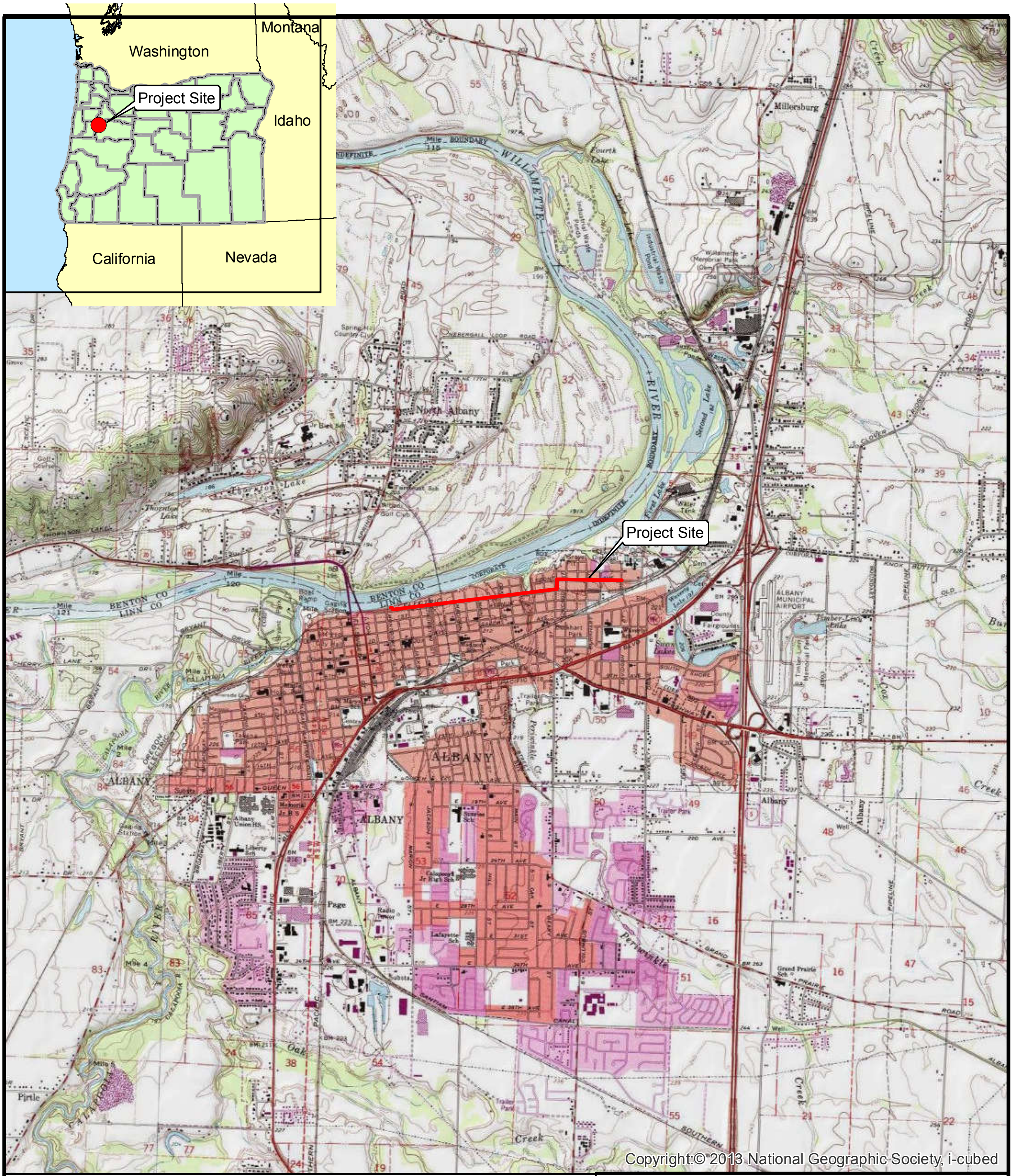

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Albany Riverfront Interceptor
Sewer Pump Station and Force Main
Albany, Oregon

VICINITY MAP

April 2019

100623

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FIG. 1

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LEGEND

⌖	Surveyed Location of Boring
△	Surveyed Location of Vacuum Excavation Exploration
—	Proposed Site Layout and Pipeline Alignment
-·-·-·-	Surveyed Contours

NOTES

1. Proposed alignment, site layout and surveyed contours provided by West Yost Associates, drawing x-51914-1818Site.dwg, obtained March 11, 2019.

0 10 20 40
Feet

Albany Riverfront Interceptor
Sewer Pump Station and Force Main
Albany, Oregon

SITE AND EXPLORATION PLAN

April 2019 100623

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FIG. 2
Page 1 of 2

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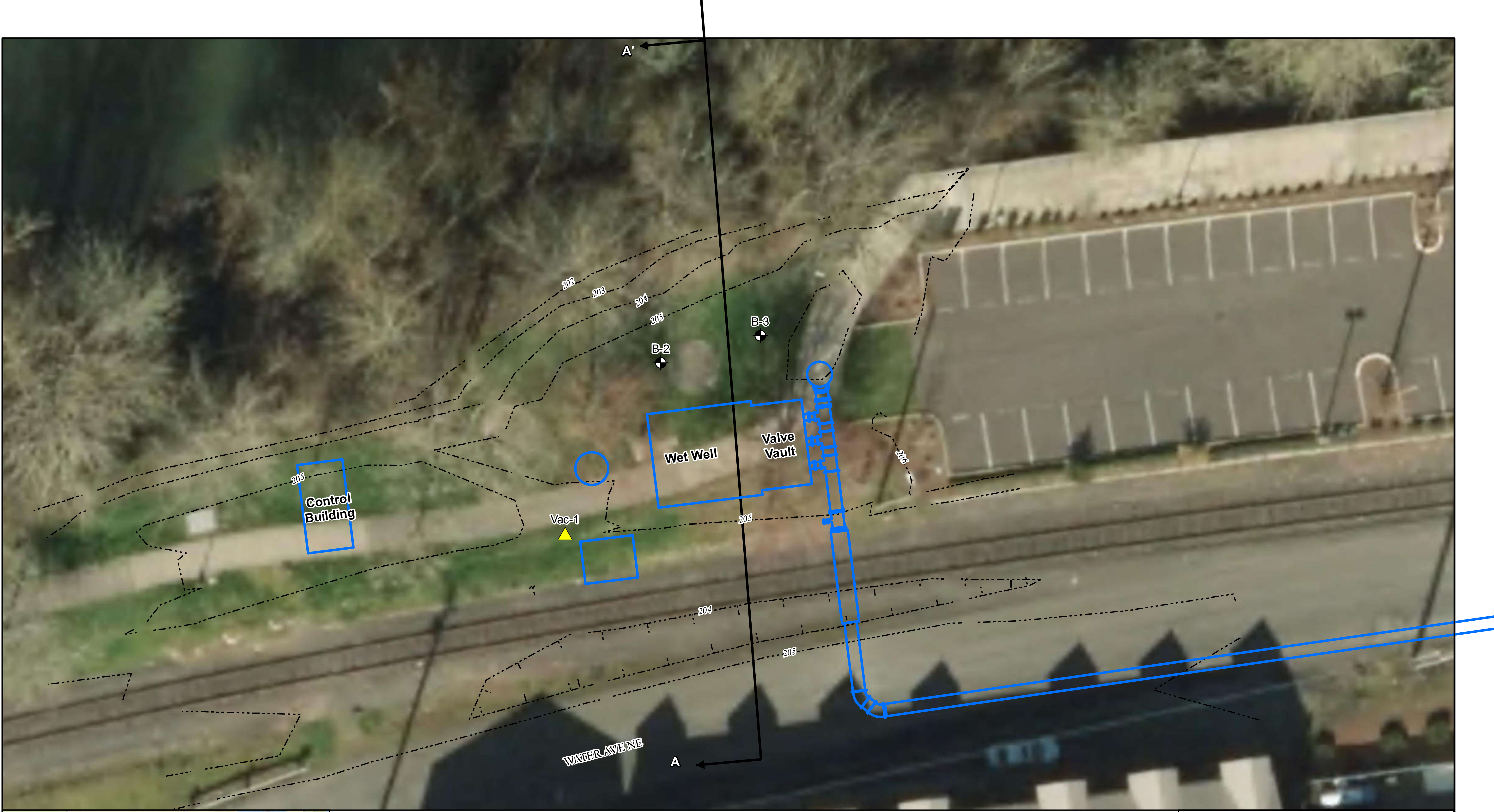

LEGEND

⊕	Surveyed Location of Boring
▲	Surveyed Location of Vacuum Excavation Exploration
—	Proposed Site Layout and Pipeline Alignment
- - · - -	Surveyed Contours

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0 10 20 40 Feet

N E S W

Albany Riverfront Interceptor
Sewer Pump Station and Force Main
Albany, Oregon

SITE AND EXPLORATION PLAN

April 2019 100623

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FIG. 2
Page 2 of 2

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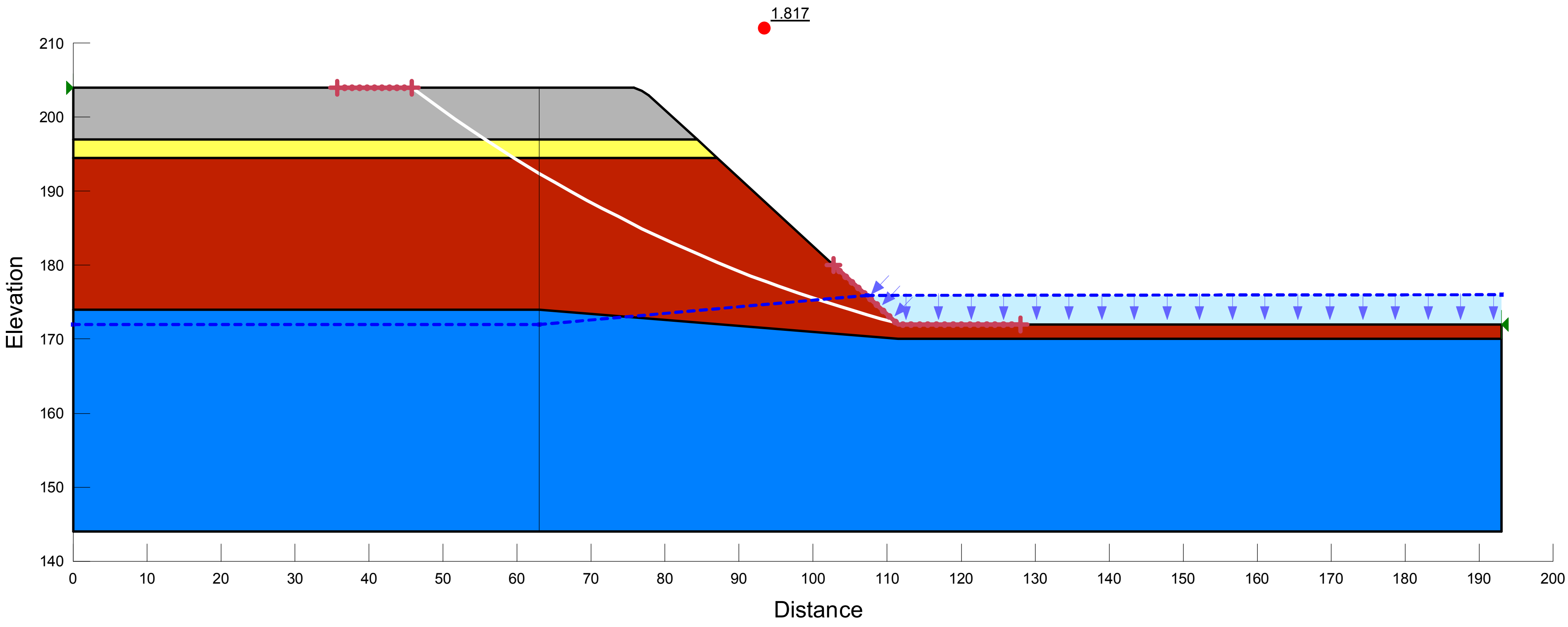

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)	Piezometric Line
⬜	Fill	Mohr-Coulomb	130	0	32	0	1
🟨	Fine Grained Missoula Deposits	Mohr-Coulomb	105	2,000	0	0	1
🟥	Linn Gravel	Mohr-Coulomb	135	0	38	0	1
🟦	Yamhill Formation	Mohr-Coulomb	115	2,300	0	0	1

Albany Riverfront Interceptor
Sewer Pump Station and Force Main
Albany, Oregon

STATIC SLOPE STABILITY ANALYSIS RESULTS

April 2019
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FIG. 3

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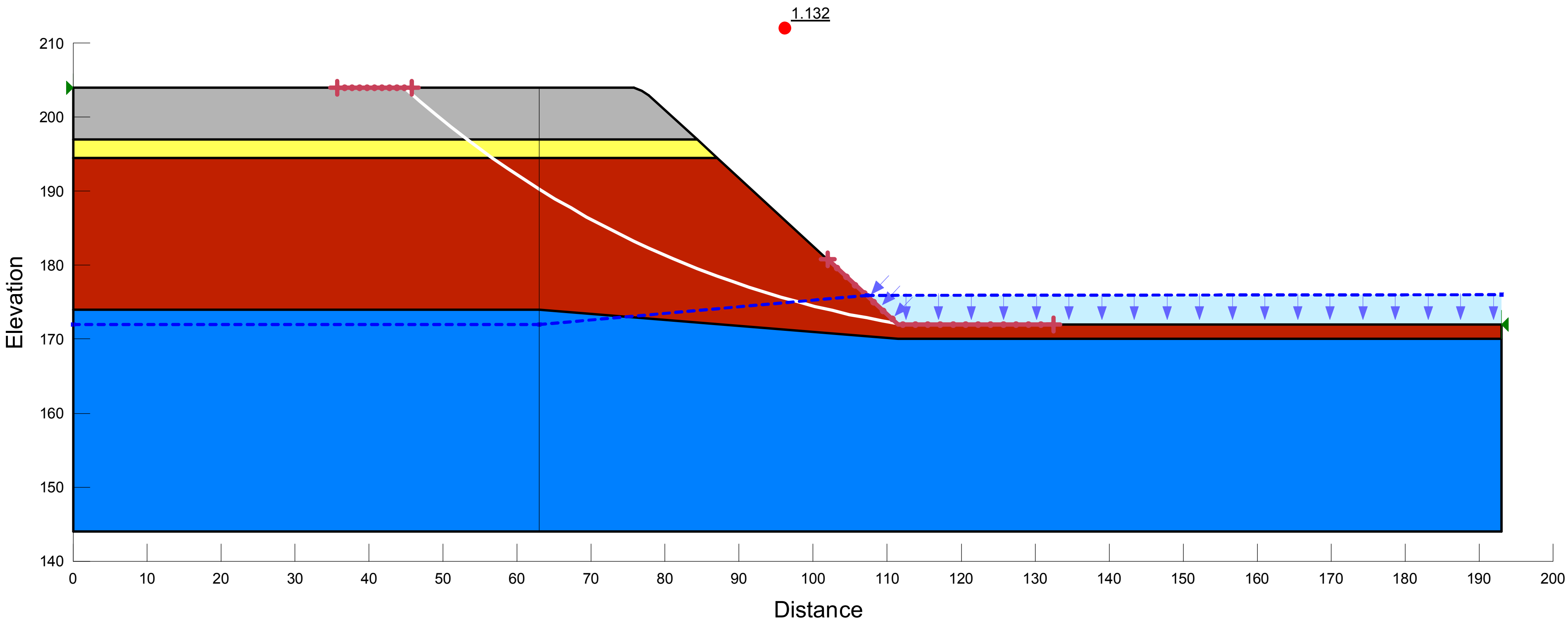

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Phi-B (°)	Piezometric Line
⬜	Fill	Mohr-Coulomb	130	0	32	0	1
🟨	Fine Grained Missoula Deposits	Mohr-Coulomb	105	2,000	0	0	1
🟥	Linn Gravel	Mohr-Coulomb	135	0	38	0	1
🟦	Yamhill Formation	Mohr-Coulomb	115	2,300	0	0	1

Albany Riverfront Interceptor
Sewer Pump Station and Force Main
Albany, Oregon

SEISMIC SLOPE STABILITY ANALYSIS RESULTS

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FIG. 4

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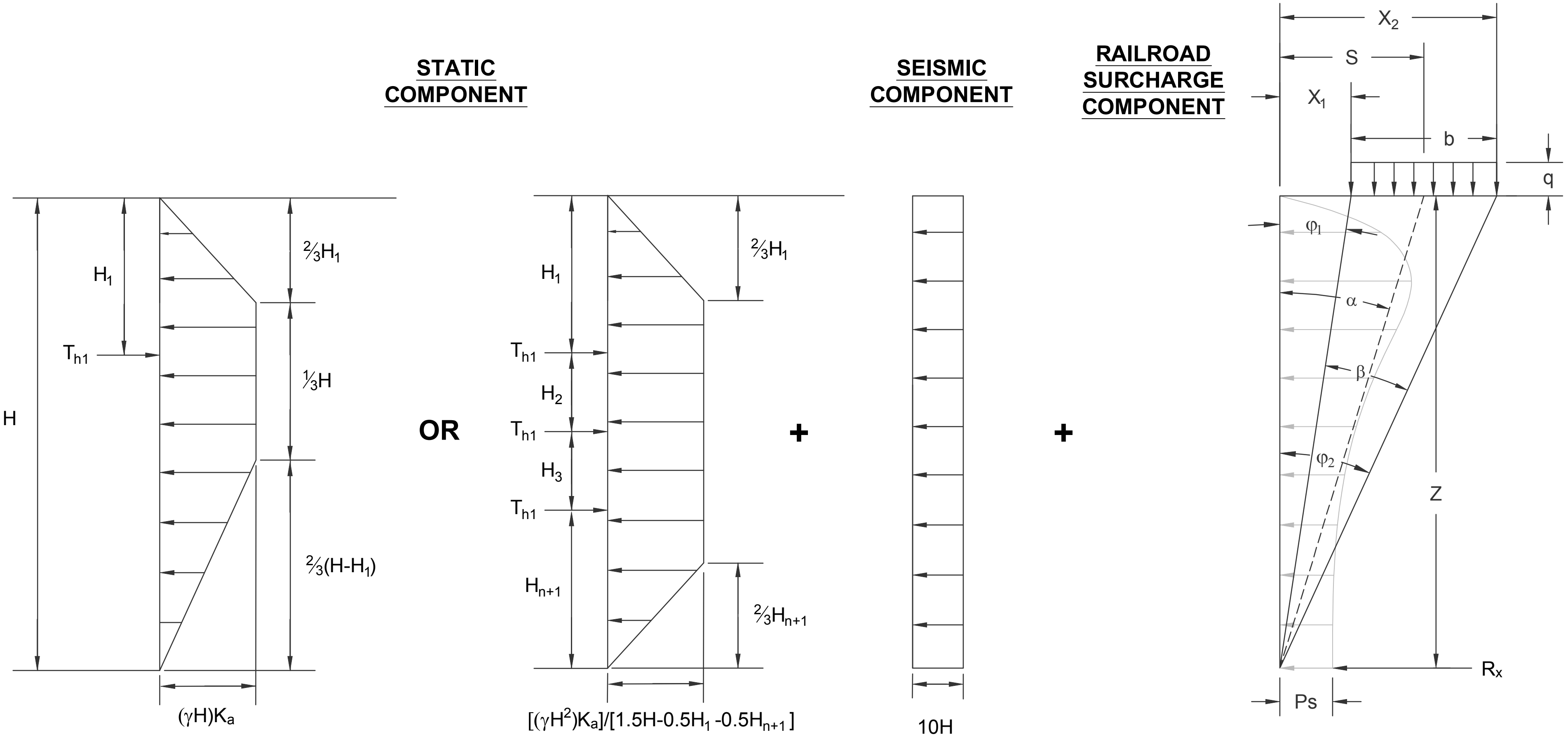

Single Row of Bracing

Multiple Rows of Bracing

Active Condition

$Ps = [2q/\pi](\beta\text{-}\sin\beta\cos2\alpha)$

$R_x = 2qH\beta/\pi$

$Z = [H^2\beta\text{-}bH+X_2^2(\pi/2\text{-}\varphi_2)\text{-}X_1^2(\pi/2\text{-}\varphi_1)]/2H\beta$

NOT TO SCALE

MATERIAL PROPERTIES				
Material ID	Material Name	Unit Weight, γ (pcf)	Friction Angle, ϕ (°)	Active Ka
1	Linn Gravels	133	34	0.28

NOTES

1. Surcharge due to the adjacent railroad is shown and should be included. Live load surcharge for a Cooper E80 can be approximated at q = 1,880 psf/ft of rail. Other surcharge due to construction vehicle traffic should also be included and the appropriate values should be selected by the structural engineer.
2. Groundwater assumed to be below the bottom of the excavation.

Albany Riverfront Interceptor
Sewer Pump Station and Force Main
Albany, Oregon

LATERAL EARTH PRESSURE DISTRIBUTION ON LIFT STATION TEMPORARY BRACED SHORING

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FIG. 5

FIG. 5

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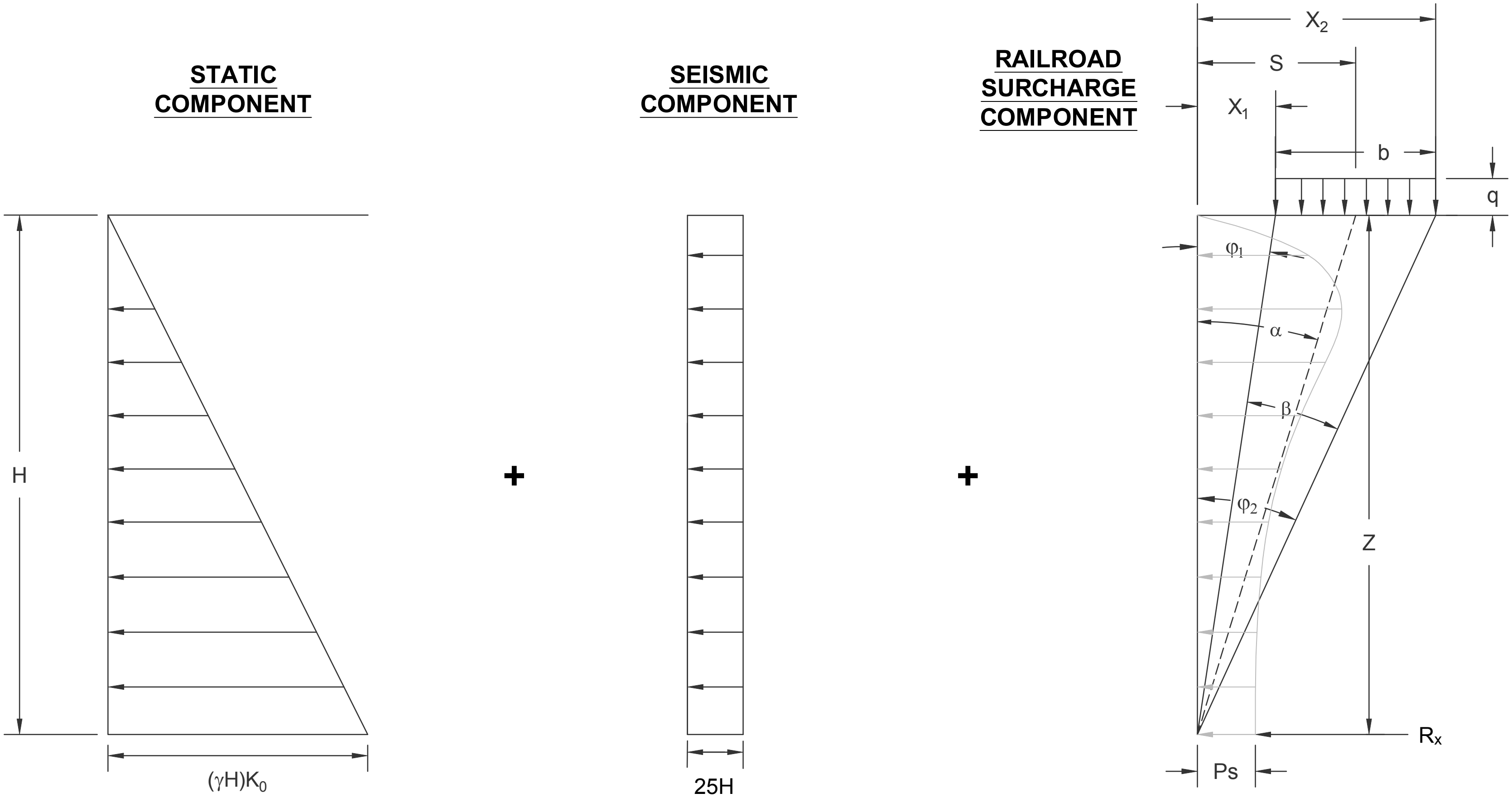

At-Rest Condition

NOT TO SCALE

$P_s = [2q/\pi](\beta - \sin\beta\cos2\alpha)$

$R_x = 2qH\beta/\pi$

$Z = [H^2\beta - bH + X_2^2(\pi/2-\varphi_2) - X_1^2(\pi/2-\varphi_1)]/2H\beta$

MATERIAL PROPERTIES

Material ID	Material Name	Unit Weight, γ (pcf)	Friction Angle, ϕ (°)	At-rest K_o
1	Structural Fill	135	36	0.41
2	Linn Gravels	133	34	0.44

NOTES

1. Surcharge due to the adjacent railroad is shown and should be included. Live load surcharge due to a Cooper E80 can be approximated at q = 1,880 psf/ft of rail. Other surcharge due to construction vehicle traffic should also be included and the appropriate values should be selected by the structural engineer.
2. Structural fill assumed to be drained imported crushed rock or gravel backfill material. Sand backfill should not be used
3. Groundwater assumed to be below the bottom of the excavation.

Albany Riverfront Interceptor
Sewer Pump Station and Force Main
Albany, Oregon

LATERAL EARTH PRESSURE DISTRIBUTION ON PERMANENT EMBEDDED STRUCTURES

April 2019 100623

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 6

FIG. 6

APPENDIX A: FIELD EXPLORATIONS

Appendix A

Field Explorations

CONTENTS

Figures

APPENDIX A

A.1 GENERAL

The field exploration program for the riverfront sewer interceptor and force main project included three geotechnical borings, designated B-1 through B-3. Completed borehole locations were measured in the field relative to existing site features. Approximate elevations (NAVD 88) were estimated from the technical memorandum provided by West Yost dated July 21, 2015. Approximate boring locations are shown on the Site and Exploration Plan, Figure 2. This appendix describes the techniques used to advance and sample the borings and presents logs of the materials encountered, along with borehole installation and backfill details.

A.2 DRILLING

Borings B-1 and B-2 were drilled on September 6 and September 7, 2018. Boring B-3 was drilled on September 27, 2018. All three were drilled using a truck-mounted CME-55 drill rig provided and operated by Western States Soil Conservation, Inc. (Western States), of Hubbard, Oregon. The borings were drilled to a total depth of 31.5 feet, 61.5 feet and 31.5 feet for borings B-1, B-2, and B-3 respectively. Shannon & Wilson geology staff were on site during drilling to locate the borings, observe drilling, collect samples, and maintain logs of the materials encountered.

A.2.1 Disturbed Sampling

Disturbed samples were collected in the borings, typically at 2.5- to 5-foot-depth intervals, using a standard 2-inch outside diameter (O.D.) split spoon sampler in conjunction with Standard Penetration Testing. In a Standard Penetration Test (SPT), ASTM D1586, the sampler is driven 18 inches into the soil using a 140-pound hammer dropped 30 inches. The number of blows required to drive the sampler the last 12 inches is defined as the standard penetration resistance, or N-value. The SPT N-value provides a measure of in situ relative density of cohesionless soils (silt, sand, and gravel), and the consistency of cohesive soils (silt and clay). All disturbed samples were visually identified and described in the field, sealed to retain moisture, and returned to our laboratory for additional examination 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 hammers. Based on information we received from Western States, the energy transfer efficiency of the hammer of the CME-55 truck rig used on site averaged 83.2

percent when measured in January 2018. All N-values presented in this report are in blows per foot, as counted in the field. No corrections of any kind have been applied.

A.3 BOREHOLE INSTALLATIONS AND ABANDONMENT

A.3.1 Observation Well

Observation wells were installed to depths of 29 feet in boring B-1, 50 feet in boring B-2, and 29 feet in boring B-3 to allow for ongoing groundwater level measurements. The wells were constructed using 2-inch-diameter, schedule 40 polyvinyl chloride (PVC) pipe. The bottom 10 feet of pipe are machine slotted (screened) to allow groundwater to enter. The annulus around the screened section of pipe is backfilled with a sand filter pack. The annulus around the solid PVC pipe above is backfilled with bentonite chips. The well is protected at the surface with a flush-mount monument set in concrete. Well construction details and measured water levels are shown on the Logs of Borings B-1, B-2, and B-3 on Figures A2, A3 and A4, respectively.

A.3.2 Borehole Abandonment

Borings B-1, B-2, and B-3 were backfilled in accordance with Oregon Department of Water Resources regulations, using bentonite chips and matching surface material. Hand augers were backfilled with excavated material.

A.4 MATERIAL DESCRIPTIONS

Soil samples were described and identified visually in the field in general accordance with ASTM D2488, Standard Practice for Description and Identification of Soils (Visual-Manual Procedure). The specific terminology used is defined in the Soil Description and Log Key, Figure A1. Consistency, color, relative moisture, degree of plasticity, peculiar odors, and other distinguishing characteristics of the samples were noted.

Once transported to the Shannon & Wilson laboratory, the samples were re-examined, various classification tests were performed, and the field descriptions and identifications were modified, where necessary. Shannon & Wilson refined the visual-manual soil descriptions and identifications based on the results of the laboratory tests, using elements of the Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System), ASTM D2487. However, ASTM D2487 was not followed in full because it requires that a suite of tests be performed to fully classify a single sample.

A.5 LOGS OF BORINGS

Summary logs of borings are presented in Figures A2 through A4. Material descriptions and interfaces on the logs are interpretive, and actual changes may be gradual. The left-hand portion of the boring logs provides description, identification, and geotechnical unit designation for the materials encountered in the boring. The right-hand portion of the boring logs shows a graphic log, sample locations and designations, well installation details, groundwater information, graphical representation of N-values, natural water contents, Atterberg limits, fines content, and sample recovery.

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

S&W INORGANIC SOIL CONSTITUENT DEFINITIONS

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

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

MOISTURE CONTENT TERMS

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

STANDARD PENETRATION TEST (SPT) SPECIFICATIONS

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

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

PARTICLE SIZE DEFINITIONS

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

RELATIVE DENSITY / CONSISTENCY

COHESIONLESS SOILS		COHESIVE SOILS	
N, SPT, BLOWS/FT.	RELATIVE DENSITY	N, SPT, BLOWS/FT.	RELATIVE CONSISTENCY
< 4	Very loose	< 2	Very soft
4 - 10	Loose	2 - 4	Soft
10 - 30	Medium dense	4 - 8	Medium stiff
30 - 50	Dense	8 - 15	Stiff
> 50	Very dense	15 - 30	Very stiff
		> 30	Hard

WELL AND BACKFILL SYMBOLS

▧	Bentonite Cement Grout	▒	Surface Cement Seal
▨	Bentonite Grout	■	Asphalt or Cap
▩	Bentonite Chips	▧	Slough
░	Silica Sand	◫	Inclinometer or Non-perforated Casing
⁘	Gravel	◫	Vibrating Wire Piezometer
▤	Perforated or Screened Casing		

PERCENTAGES TERMS [1, 2]

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

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

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

City of Albany Riverfront Interceptor Sewer
Pump Station and Force Main
Albany, Oregon

SOIL DESCRIPTION AND LOG KEY

April 2019
100623-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A1
Sheet 1 of 3

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

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

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

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

NOTES

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

City of Albany Riverfront Interceptor Sewer
Pump Station and Force Main
Albany, Oregon

SOIL DESCRIPTION AND LOG KEY

April 2019 100623-001

SHANNON & WILSON, INC.
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FIG. A1
Sheet 2 of 3

GRADATION TERMS

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

CEMENTATION TERMS[1]

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

PLASTICITY[2]

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

ADDITIONAL TERMS

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

PARTICLE ANGULARITY AND SHAPE TERMS[1]

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

ACRONYMS AND ABBREVIATIONS

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

STRUCTURE TERMS[1]

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

City of Albany Riverfront Interceptor Sewer
Pump Station and Force Main
Albany, Oregon

SOIL DESCRIPTION AND LOG KEY

April 2019 100623-001

SHANNON & WILSON, INC.
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FIG. A1
Sheet 3 of 3

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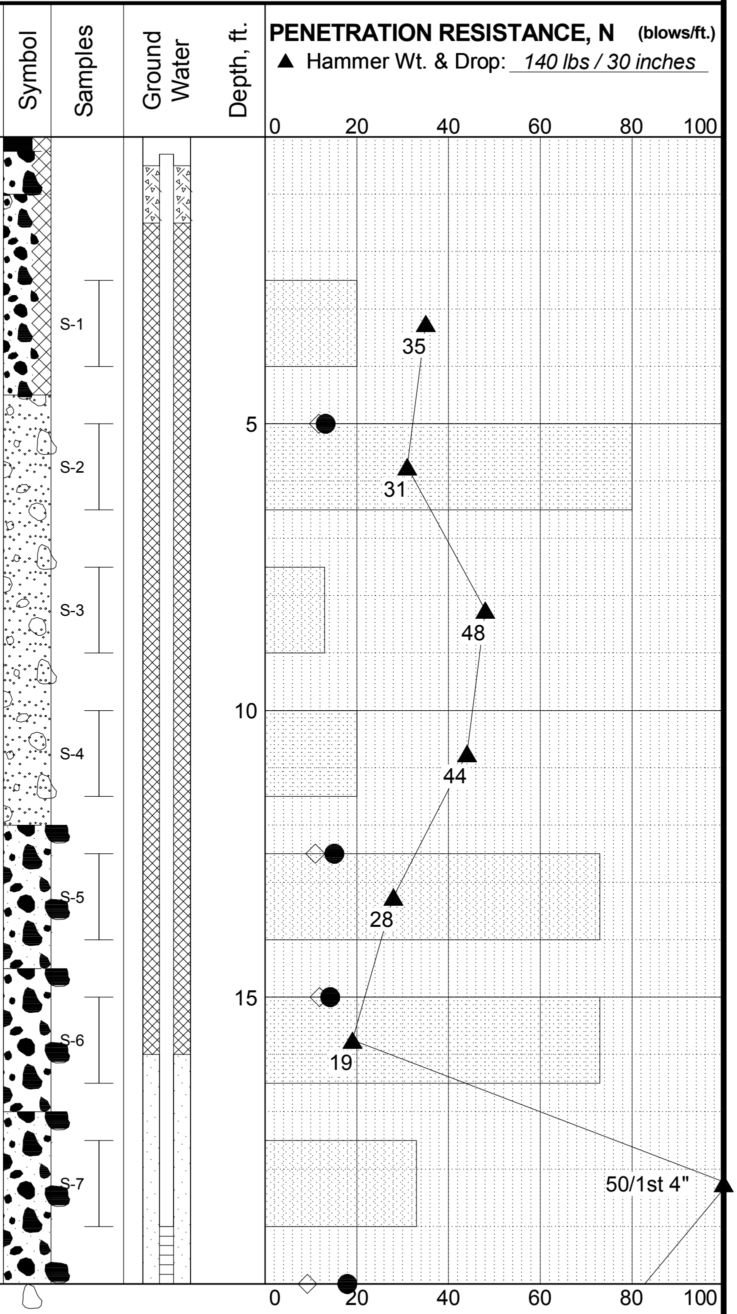
[2] Adapted, with permission, from ASTM D2488 - 09a Standard Practice for Description and Identification of Soils (Visual-Manual Procedure), copyright ASTM International, 100 Barr Harbor Drive, West Conshohocken, PA 19428. A copy of the complete standard may be obtained from ASTM International, www.astm.org.

Total Depth:	*31.5 ft.*	Northing:	*~ 366,139 ft.*	Drilling Method:	*Mud Rotary*	Hole Diam.:	*5 in.*
Top Elevation:	*~ 215 ft.*	Easting:	*~ 7,531,650 ft.*	Drilling Company:	*Western States*	Rod Type:	*NWJ*
Vert. Datum:	*NAVD88*	Station:	*~*	Drill Rig Equipment:	*CME-75 truck*	Hammer Type:	*Automatic*
Horiz. Datum:	*OR SPCS*	Offset:	*~*	Other Comments:	*Hammer Efficiency = 81.4%*		

SOIL DESCRIPTION *Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.*	*Elev.* Depth (ft.)
Asphalt Concrete; 3 inches thick	*214.8* 0.3
Base Aggregate	*214.0* 1.0
Dense, brown and gray, *Poorly Graded Gravel with Sand (GP)*; moist; fine to coarse, angular to rounded gravel; fine to course sand. **FILL**	*210.5* 4.5
Dense, brown, gray, and tan, *Well-Graded Gravel with Silt and Sand (GW-GM) to Well-Graded Sand with Silt and Gravel (SW-SM)*; moist; fine to coarse, subrounded to rounded gravel; fine to course sand; nonplastic to low plasticity fines; slight iron-oxide staining. **REWORKED LINN GRAVEL**	*203.0* 12.0
Medium dense, brown, gray, and tan, *Poorly Graded Gravel with Silt and Sand (GP-GM)*; moist; fine to coarse, subrounded to rounded gravel; fine to course sand; nonplastic fines; slight to moderate iron-oxide staining; trace decomposed gravel clasts.	*200.5* 14.5
Medium dense, brown, gray, and tan, *Poorly Graded Gravel with Clay and Sand (GP-GC)*; moist; fine to coarse, subrounded to rounded gravel; fine to course sand; medium plasticity fines; moderate iron-oxide staining.	*198.0* 17.0
Very dense, brown, gray, and red-brown, *Well-Graded Gravel with Silt and Sand (GW)*.	



CONTINUED NEXT SHEET

LEGEND

⌶ Standard Penetration Test

▽ Groundwater Level on Date Shown

▨ Recovery (%)

◇ % Fines (<0.075mm)

● % Water Content

Plastic Limit ├──┤ Liquid Limit

NOTES

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

City of Albany Riverfront Interceptor Sewer
Pump Station and Force Main
Albany, Oregon

LOG OF BORING B-1

April 2019 100623-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A2
Sheet 1 of 2

REV 3

MASTER_LOG_E 100623-001.GPJ SW2013LIBRARYPDX.GLB SHANWIL_PDX.GDT 4/5/19

Log: CKS Rev: AAJH Typ: KTR

Total Depth:	*31.5 ft.*	Northing:	*~ 366,139 ft.*	Drilling Method:	*Mud Rotary*	Hole Diam.:	*5 in.*
Top Elevation:	*~ 215 ft.*	Easting:	*~ 7,531,650 ft.*	Drilling Company:	*Western States*	Rod Type:	*NWJ*
Vert. Datum:	*NAVD88*	Station:	*~*	Drill Rig Equipment:	*CME-75 truck*	Hammer Type:	*Automatic*
Horiz. Datum:	*OR SPCS*	Offset:	*~*	Other Comments:	*Hammer Efficiency = 81.4%*		

SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.

Continued:

Very dense, brown, gray, and red-brown, *Well-Graded Gravel with Silt and Sand (GW-GM)*; moist; fine to coarse, subrounded to rounded gravel; fine to coarse sand; nonplastic fines; slight to moderate iron-oxide staining; trace decomposed gravel clasts.

LINN GRAVEL

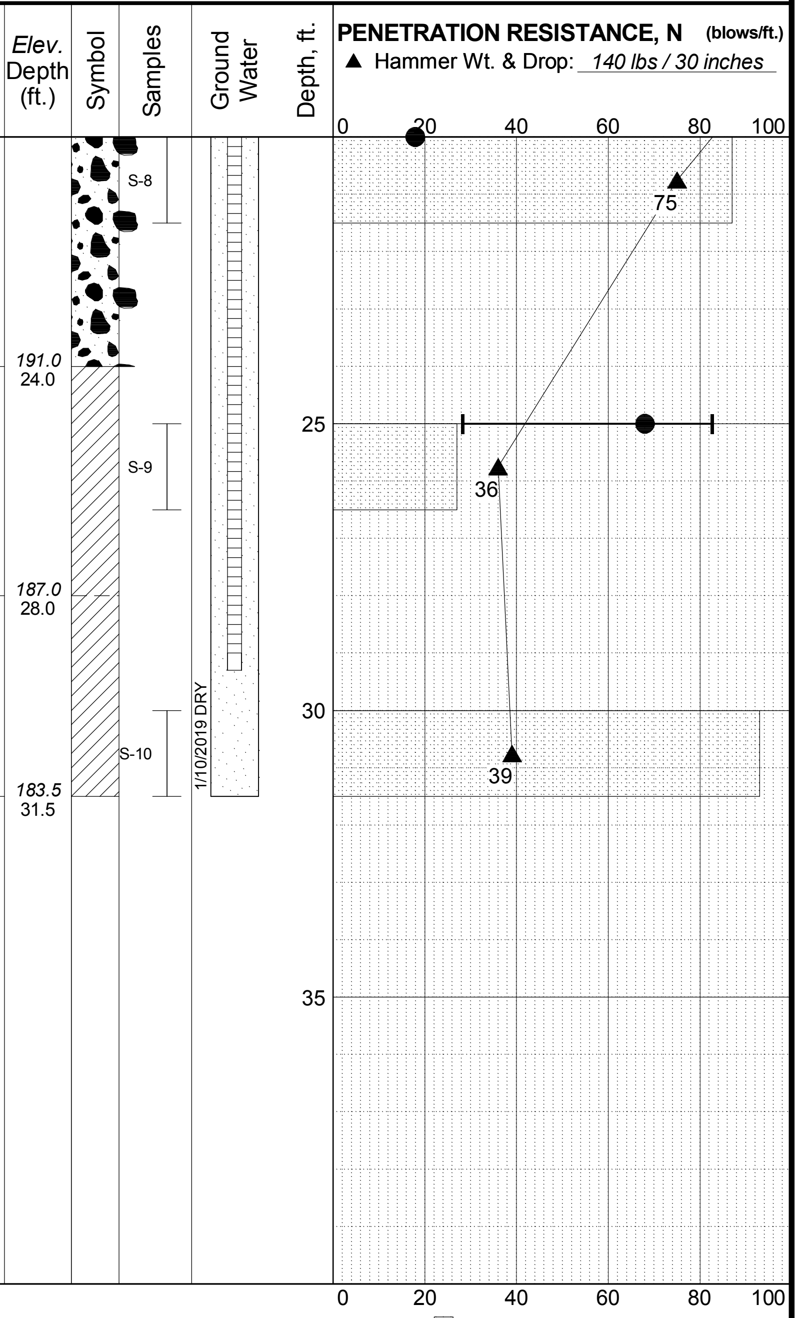
Slight mud loss at 17 feet

Hard, dark brown and gray mottled, *Fat Clay (CH)*; wet; trace fine to medium sand; high plasticity; trace fine organics.

YAMHILL FORMATION

Hard, dark brown, gray, and black mottled, *Fat Clay (CH)*; moist to wet; trace fine sand; high plasticity.

Completed: August 21, 2018



▨ Recovery (%)

◇ % Fines (<0.075mm)

● % Water Content

Plastic Limit ├──┤ Liquid Limit

LEGEND

⌶ Standard Penetration Test

⊻ Groundwater Level on Date Shown

NOTES

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

City of Albany Riverfront Interceptor Sewer
Pump Station and Force Main
Albany, Oregon

LOG OF BORING B-1

April 2019 100623-001

SHANNON & WILSON, INC.
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FIG. A2
Sheet 2 of 2

REV 3

MASTER_LOG_E 100623-001.GPJ SW2013LIBRARYPDX.GLB SHANWIL_PDX.GDT 4/5/19

Log: CKS Rev: AAJH Typ: KTR

Total Depth:	61.5 ft.	Northing:	~ 365,293 ft.	Drilling Method:	Mud Rotary	Hole Diam.:	5 in.
Top Elevation:	~ 205 ft.	Easting:	~ 7,524,890 ft.	Drilling Company:	Western States	Rod Type:	NWJ
Vert. Datum:	NAVD88	Station:	~	Drill Rig Equipment:	CME-75 truck	Hammer Type:	Automatic
Horiz. Datum:	OR SPCS	Offset:	~	Other Comments:	Hammer Efficiency = 81.4%		

SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.

Elev. Depth (ft.)

Dark brown, *Silt (ML)*; moist; trace fine to coarse, angular to subrounded gravel; fine to coarse sand; nonplastic to low plasticity.

204.0
1.0

Medium dense, brown and gray, *Silty Gravel with Sand (GM)*; moist; fine to coarse, angular to rounded gravel; fine to course sand; low plasticity fines; few pockets of gray *Lean Clay (CL)*; few pockets of charcoal.

FILL

198.0
7.0

Very stiff, gray and orange-brown mottled, *Lean Clay with Sand (CL)*; moist; fine to medium sand; medium plasticity; stratified with few interbeds of *Silty Sand (SM)*.

FINE-GRAINED MISSOULA FLOOD DEPOSITS

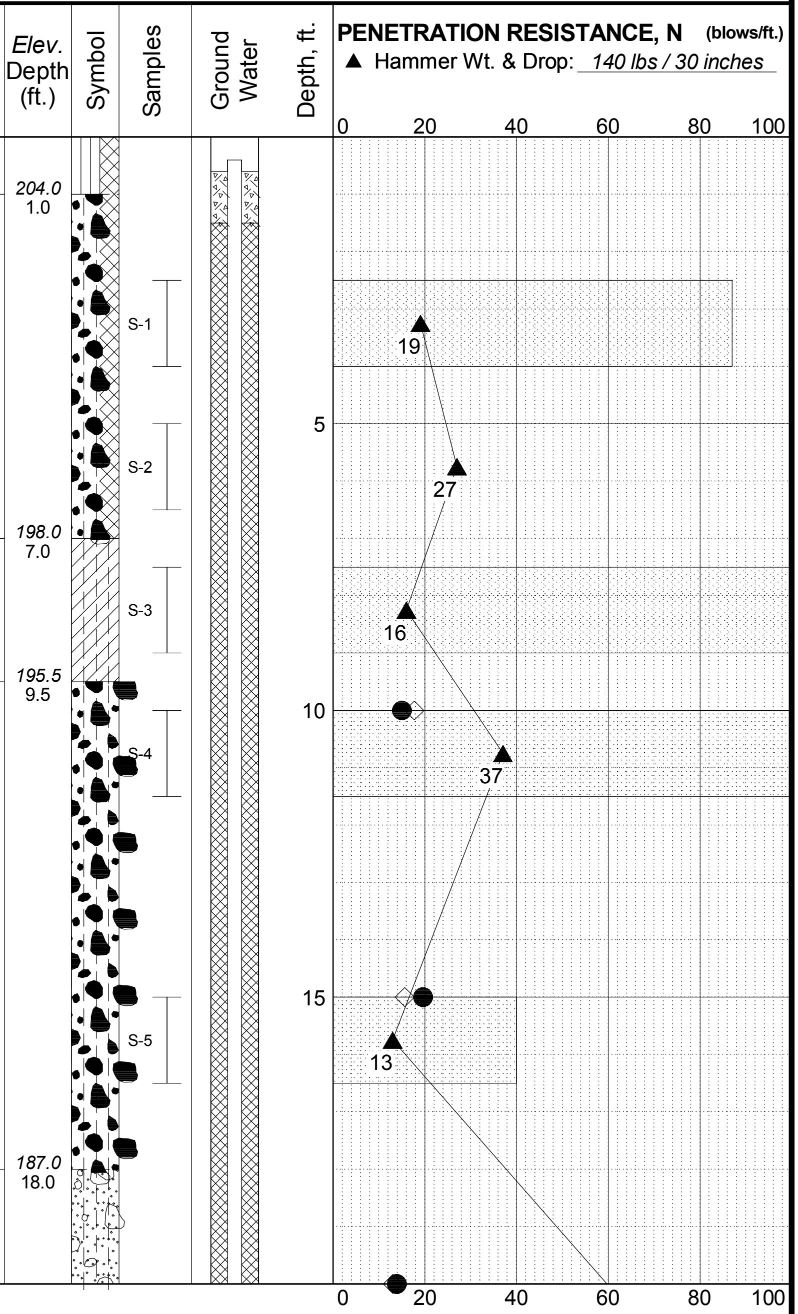
195.5
9.5

Medium dense to dense, brown, red-brown, and orange-brown mottled, *Silty Gravel with Sand (GM)*; moist; fine to coarse, subrounded to rounded gravel; fine to course sand; low plasticity fines; slight iron-oxide staining.

REWORKED LINN GRAVEL

187.0
18.0

Very dense, brown, tan, and red-brown, *Silty Gravel with Sand (GM) to Silty Sand with Gravel (SM)*.



CONTINUED NEXT SHEET

LEGEND

⌶ Standard Penetration Test

▽ Groundwater Level on Date Shown

▨ Recovery (%)

◇ % Fines (<0.075mm)

● % Water Content

Plastic Limit ├──┤ Liquid Limit

NOTES

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

City of Albany Riverfront Interceptor Sewer
Pump Station and Force Main
Albany, Oregon

LOG OF BORING B-2

April 2019 100623-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A3
Sheet 1 of 4

REV 3

MASTER_LOG_E 100623-001.GPJ SW2013LIBRARYPDX.GLB SHANWIL_PDX.GDT 4/5/19 Log: CKS Rev: AAJH Typ: KTR

Total Depth:	61.5 ft.	Northing:	~ 365,293 ft.	Drilling Method:	Mud Rotary	Hole Diam.:	5 in.
Top Elevation:	~ 205 ft.	Easting:	~ 7,524,890 ft.	Drilling Company:	Western States	Rod Type:	NWJ
Vert. Datum:	NAVD88	Station:	~	Drill Rig Equipment:	CME-75 truck	Hammer Type:	Automatic
Horiz. Datum:	OR SPCS	Offset:	~	Other Comments:	Hammer Efficiency = 81.4%		

SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.

Elev. Depth (ft.)

Continued:

Very dense, brown, tan, and red-brown, *Silty Gravel with Sand (GM) to Silty Sand with Gravel (SM)*; moist; fine to coarse, subangular to rounded gravel; fine to coarse sand; low plasticity fines; slight to moderate iron-oxide staining; trace highly weathered gravel clasts.

LINN GRAVEL

174.5
30.5

Stiff, gray, *Fat Clay (CH)*; moist; trace fine sand; high plasticity.

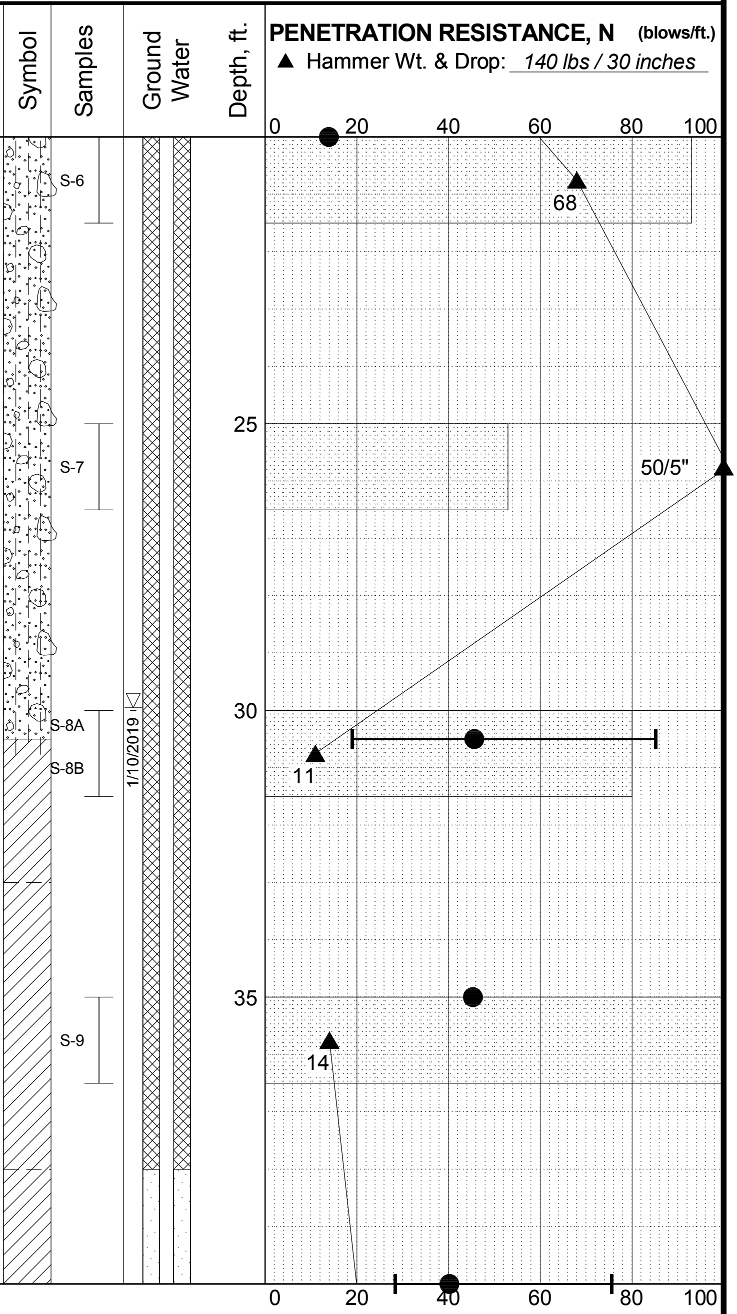
YAMHILL FORMATION

172.0
33.0

Stiff, blue-gray, *Fat Clay (CH)*; moist to wet; trace fine sand; high plasticity; faintly stratified with few dark gray interbeds; R0-R1 Mudstone.

167.0
38.0

Very stiff, blue-gray and dark gray mottled, *Fat Clay (CH)*; moist to wet; high plasticity; trace white and gray secondary mineral deposits.



CONTINUED NEXT SHEET

LEGEND

⌶ Standard Penetration Test

▽ Groundwater Level on Date Shown

⬚ Recovery (%)

◇ % Fines (<0.075mm)

● % Water Content

Plastic Limit ├──┤ Liquid Limit

NOTES

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

City of Albany Riverfront Interceptor Sewer
Pump Station and Force Main
Albany, Oregon

LOG OF BORING B-2

April 2019 100623-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A3
Sheet 2 of 4

REV 3

MASTER_LOG_E 100623-001.GPJ SW2013LIBRARYPDX.GLB SHANWIL_PDX.GDT 4/5/19

Log: CKS Rev: AAJH Typ: KTR

Total Depth: 61.5 ft.
Top Elevation: ~ 205 ft.
Vert. Datum: NAVD88
Horiz. Datum: OR SPCS

Northing: ~ 365,293 ft.
Easting: ~ 7,524,890 ft.
Station: ~
Offset: ~

Drilling Method: Mud Rotary
Drilling Company: Western States
Drill Rig Equipment: CME-75 truck
Other Comments: Hammer Efficiency = 81.4%

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

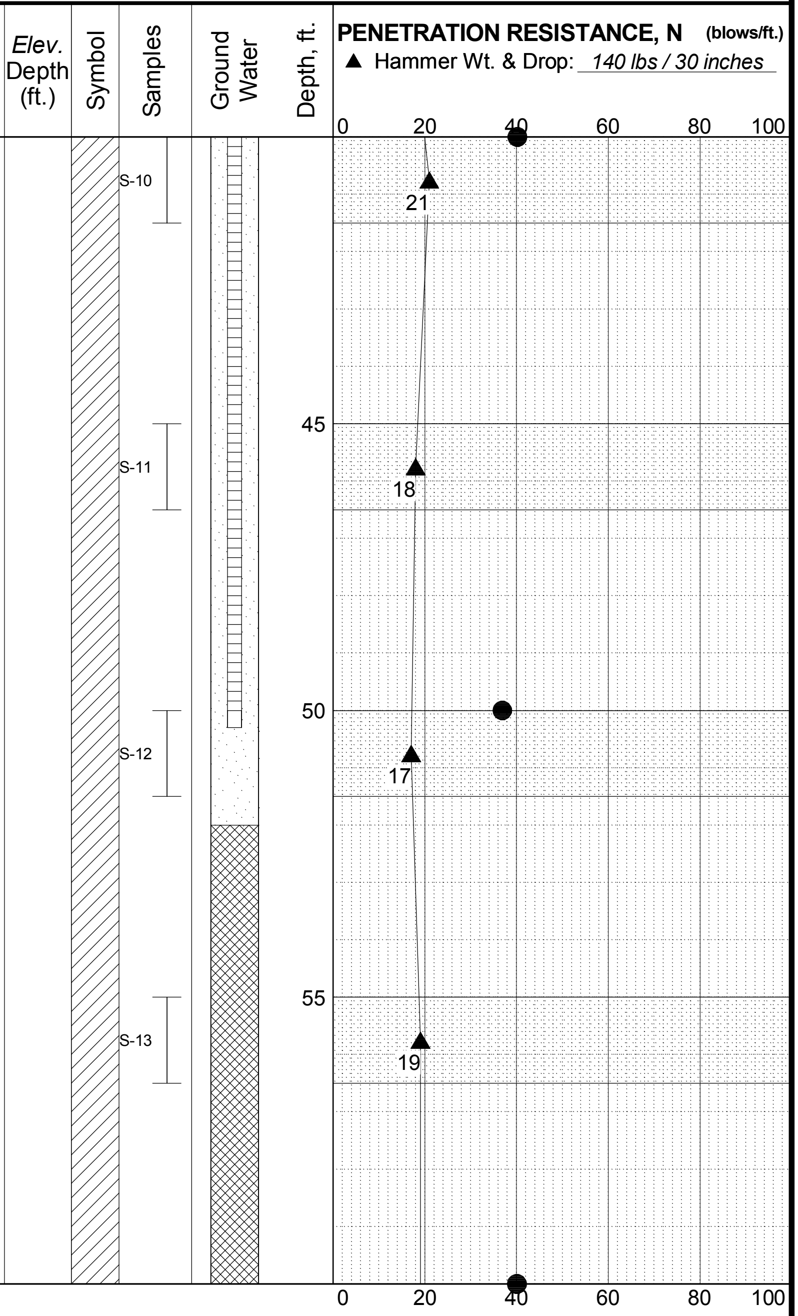
SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.

Continued:

Very stiff, blue-gray an dark gray mottled, *Fat Clay (CH)*; moist to wet; high plasticity; trace white and gray secondary mineral deposits; R0-R1 Mudstone.

YAMHILL FORMATION



CONTINUED NEXT SHEET

LEGEND

⌶ Standard Penetration Test

⊻ Groundwater Level on Date Shown

▨ Recovery (%)

◇ % Fines (<0.075mm)

● % Water Content

Plastic Limit ├──┤ Liquid Limit

NOTES

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

City of Albany Riverfront Interceptor Sewer
Pump Station and Force Main
Albany, Oregon

LOG OF BORING B-2

April 2019 100623-001

SHANNON & WILSON, INC.
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FIG. A3
Sheet 3 of 4

REV 3

MASTER_LOG_E 100623-001.GPJ SW2013LIBRARYPDX.GLB SHANWIL_PDX.GDT 4/5/19 Log: CKS Rev: AAJH Typ: KTR

Total Depth:	*61.5 ft.*	Northing:	*~ 365,293 ft.*	Drilling Method:	*Mud Rotary*	Hole Diam.:	*5 in.*
Top Elevation:	*~ 205 ft.*	Easting:	*~ 7,524,890 ft.*	Drilling Company:	*Western States*	Rod Type:	*NWJ*
Vert. Datum:	*NAVD88*	Station:	*~*	Drill Rig Equipment:	*CME-75 truck*	Hammer Type:	*Automatic*
Horiz. Datum:	*OR SPCS*	Offset:	*~*	Other Comments:	*Hammer Efficiency = 81.4%*		

SOIL DESCRIPTION

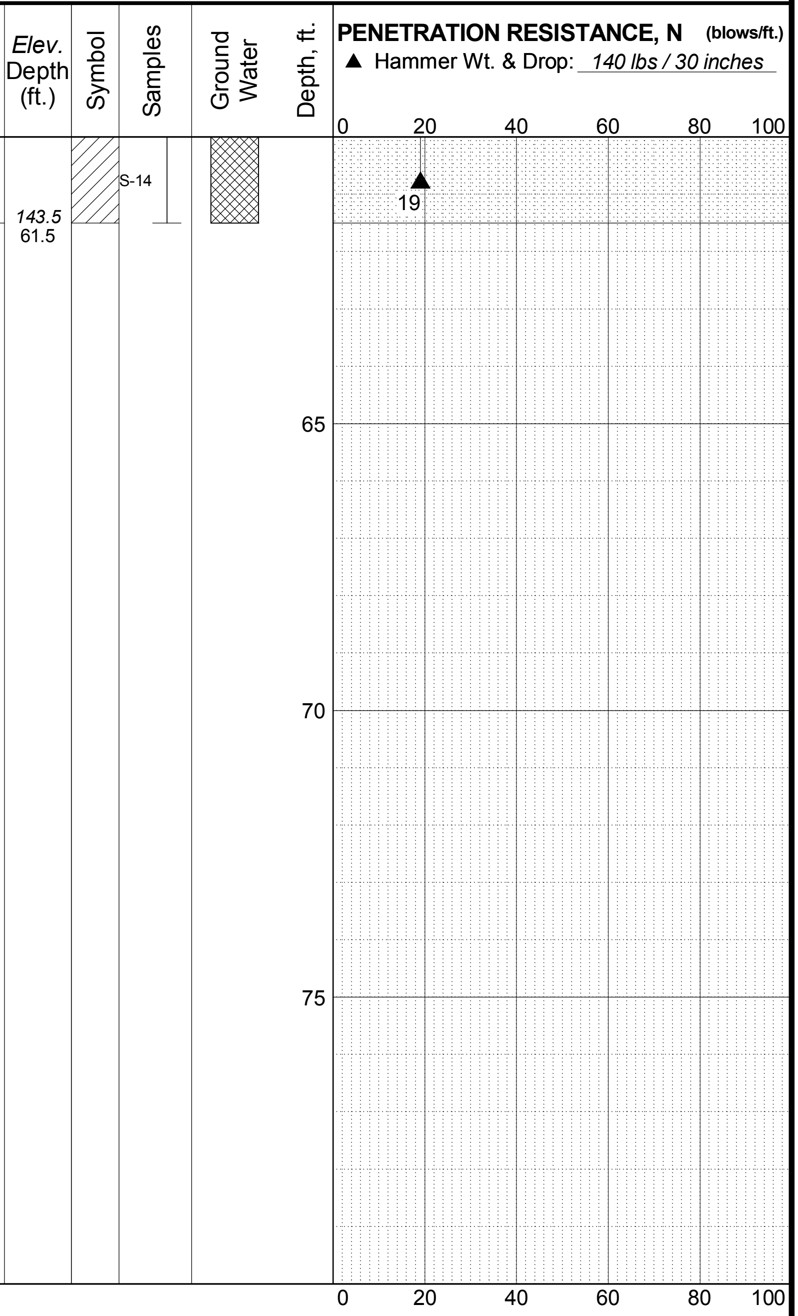
Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.

Continued:

Very stiff, blue-gray an dark gray mottled, *Fat Clay (CH)*; moist to wet; high plasticity; trace white and gray secondary mineral deposits; R0-R1 Mudstone.

YAMHILL FORMATION

Completed: August 23, 2018



▨ Recovery (%)

◇ % Fines (<0.075mm)

● % Water Content

Plastic Limit ├──┤ Liquid Limit

LEGEND

⌶ Standard Penetration Test

⊻ Groundwater Level on Date Shown

NOTES

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

City of Albany Riverfront Interceptor Sewer
Pump Station and Force Main
Albany, Oregon

LOG OF BORING B-2

April 2019 100623-001

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. A3
Sheet 4 of 4

MASTER_LOG_E 100623-001.GPJ SW2013LIBRARYPDX.GLB SHANWIL_PDX.GDT 4/5/19 *Log: CKS Rev: AAJH Typ: KTR*

Total Depth:	31.5 ft.	Northing:	~ 365,299 ft.	Drilling Method:	Mud Rotary	Hole Diam.:	5 in.
Top Elevation:	~ 205 ft.	Easting:	~ 7,524,911 ft.	Drilling Company:	Western States	Rod Type:	NWJ
Vert. Datum:	NAVD88	Station:	~	Drill Rig Equipment:	CME-75 truck	Hammer Type:	Automatic
Horiz. Datum:	OR SPCS	Offset:	~	Other Comments:	Hammer Efficiency = 81.4%		

SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.

Topsoil

Medium dense, brown, *Silty Gravel with Sand (GM)*; wet; fine to coarse, angular to subrounded gravel; fine to coarse sand; low plasticity fines.

FILL

Very stiff to hard, gray to dark gray, brown, orange, *Lean Clay (CL)*; moist; trace fine sand; medium plasticity fines.

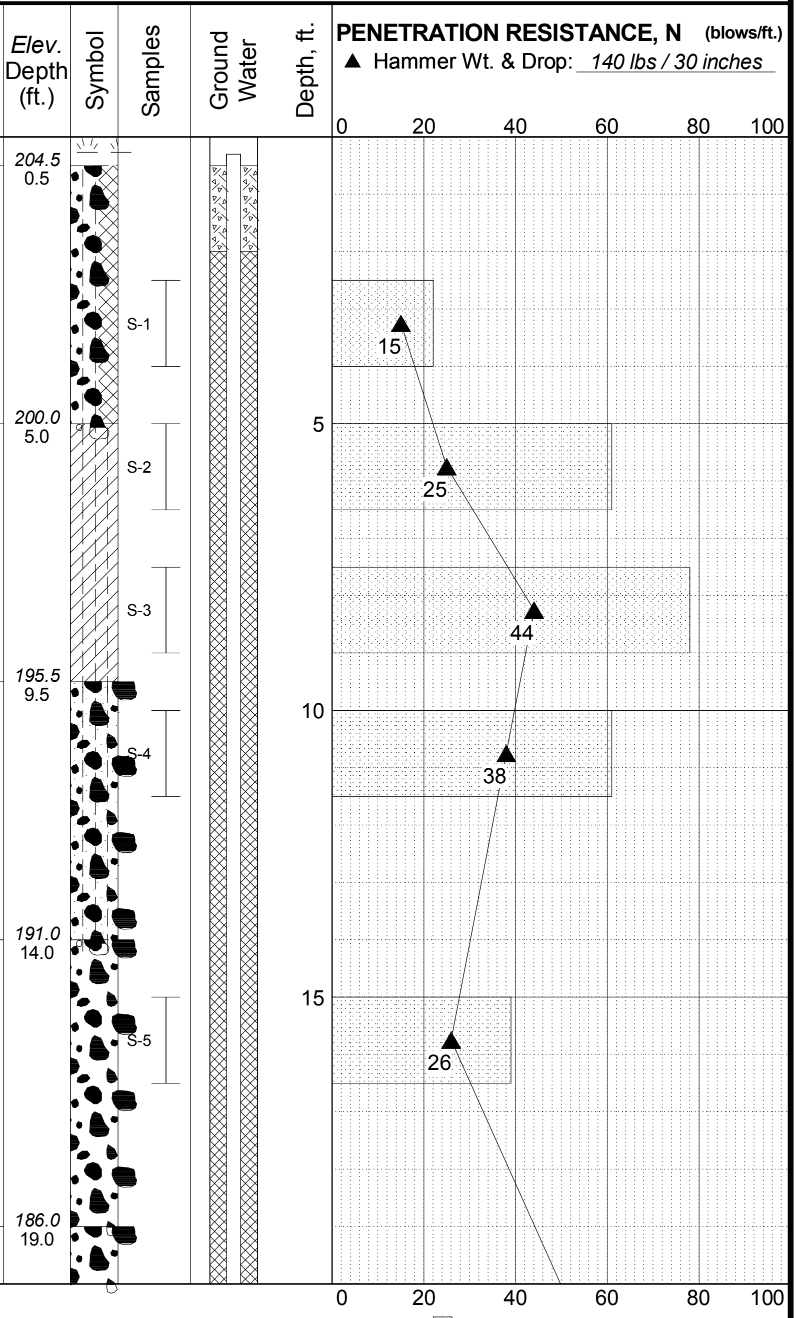
FINE-GRAINED MISSOULA FLOOD DEPOSITS

Dense, gray to brown to orange, *Silty Gravel with Sand (GM) to Silty Sand with Gravel (SM)*; moist to wet; fine to coarse, angular to subrounded gravel; fine to coarse sand; low to medium plasticity fines.

REWORKED LINN GRAVEL

Medium dense, brown to gray to tan, *Silty Gravel with Sand, with Cobbles and Boulders (GM)*; wet; trace cobbles and possible boulders; fine to coarse, angular to subrounded gravel; fine to coarse sand; low plasticity fines.

Very dense, brown to gray to tan, *Poorly Graded Gravel with Silt and Sand, with*



CONTINUED NEXT SHEET

LEGEND

⌶ Standard Penetration Test

▽ Groundwater Level on Date Shown

⬚ Recovery (%)

◇ % Fines (<0.075mm)

● % Water Content

Plastic Limit ├──┤ Liquid Limit

NOTES

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

City of Albany Riverfront Interceptor Sewer
Pump Station and Force Main
Albany, Oregon

LOG OF BORING B-3

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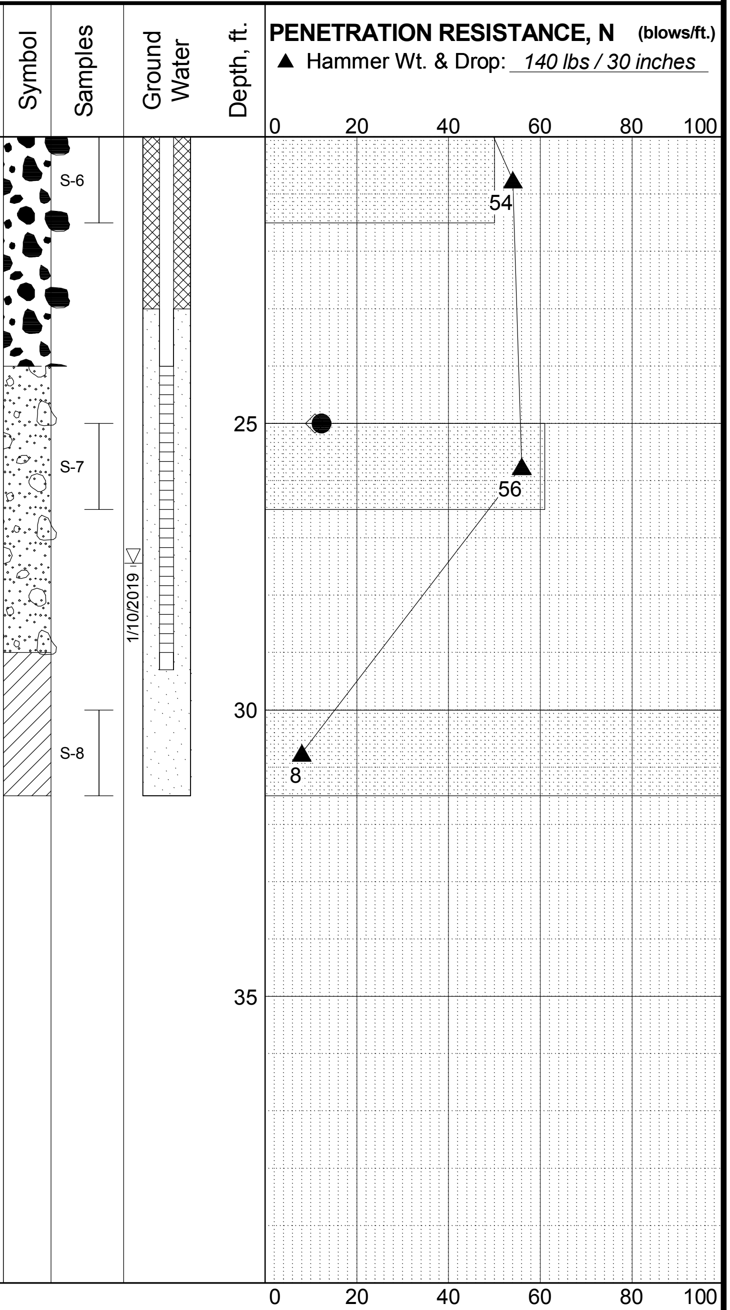
FIG. A4
Sheet 1 of 2

REV 3

Total Depth:	*31.5 ft.*	Northing:	*~ 365,299 ft.*	Drilling Method:	*Mud Rotary*	Hole Diam.:	*5 in.*
Top Elevation:	*~ 205 ft.*	Easting:	*~ 7,524,911 ft.*	Drilling Company:	*Western States*	Rod Type:	*NWJ*
Vert. Datum:	*NAVD88*	Station:	*~*	Drill Rig Equipment:	*CME-75 truck*	Hammer Type:	*Automatic*
Horiz. Datum:	*OR SPCS*	Offset:	*~*	Other Comments:	*Hammer Efficiency = 81.4%*		

SOIL DESCRIPTION *Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between soil types, and the transitions may be gradual.*	*Elev.* Depth (ft.)
Cobbles and Boulders (GP-GM); wet; trace cobbles and possible boulders; fine to coarse, angular to subrounded gravel; fine to coarse sand; nonplastic fines. **LINN GRAVEL**	
Very dense, brown to gray to tan, *Well-Graded Gravel with Silt and Sand (GW-GM) to Well-Graded Sand with Silt and Gravel (SW-SM)*; moist to wet; fine to coarse, angular to subrounded gravel; fine to coarse sand; nonplastic to low plasticity fines.	*181.0* 24.0
Medium stiff to stiff, gray, *Fat Clay (CH)*; moist; trace fine sand; high plasticity. **YAMHILL FORMATION**	*176.0* 29.0
Completed: September 27, 2018	*173.5* 31.5



LEGEND

⌶ Standard Penetration Test

⊻ Groundwater Level on Date Shown

⬚ Recovery (%)

◇ % Fines (<0.075mm)

● % Water Content

Plastic Limit ├──┤ Liquid Limit

NOTES

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

City of Albany Riverfront Interceptor Sewer
Pump Station and Force Main
Albany, Oregon

LOG OF BORING B-3

April 2019 100623-001

SHANNON & WILSON, INC.
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FIG. A4
Sheet 2 of 2

REV 3

APPENDIX B: LABORATORY TEST RESULTS

Appendix B

Laboratory Test Results

CONTENTS

Figures

APPENDIX B

B.1 GENERAL

Soil samples obtained during the field explorations were described and identified in the field in general accordance with the Standard Practice for Description and Identification of Soils (Visual-Manual Procedure), ASTM D2488. The specific terminology used is presented on Appendix A, Figure A1.

The samples were reviewed in the Shannon & Wilson laboratory. The physical characteristics of the samples were noted, and the field descriptions and identifications were modified where necessary in accordance with terminology presented in Appendix A, Figure A1.

Representative samples were selected for various laboratory tests. We refined our visual-manual soil descriptions and identifications based on the results of the laboratory tests, using elements of the Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System), ASTM D2487. The refined descriptions and identifications were then incorporated into the Logs of Borings, presented in Appendix A. Note that ASTM D2487 was not followed in full because it requires that a suite of tests be performed to fully classify a single sample.

The soil testing program included moisture content analyses, Atterberg limits tests, and particle-size analyses. The testing was performed by Shannon & Wilson in accordance with applicable ASTM standards. General testing procedures are summarized in the following paragraphs.

B.2 SOIL TESTING

B.2.1 Moisture (Natural Water) Content

Natural moisture content analyses 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 the explorations are performed and is defined as the ratio of water weight to dry soil weight, expressed as a percentage. The results of the moisture content analyses are presented graphically on the Logs of Borings in Appendix A.

B.2.2 Atterberg Limits

Atterberg limits were determined for three 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).

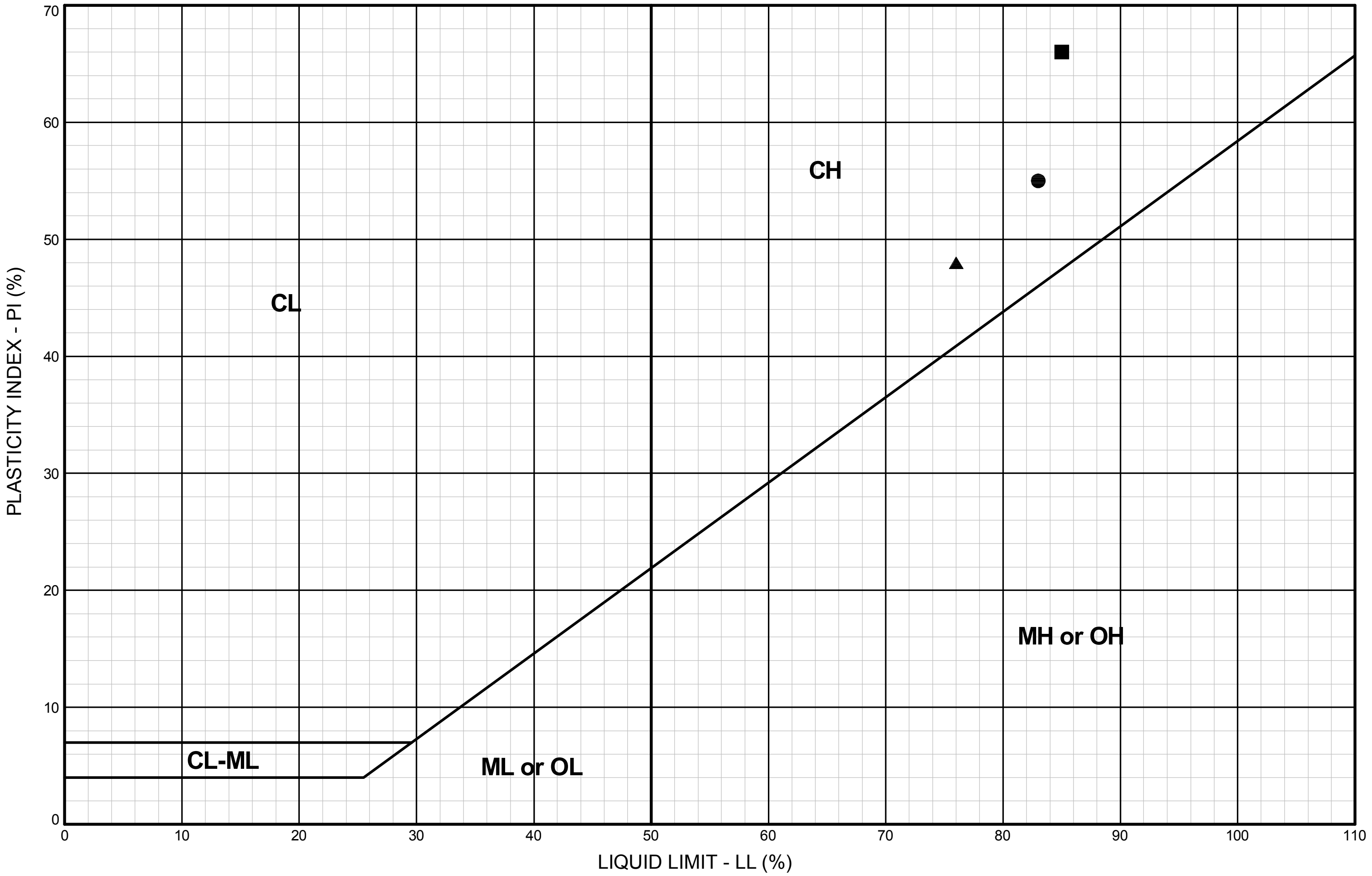
These are the maximum and minimum moisture contents at which the soil exhibits plastic behavior. A soil's plasticity index (PI) can be determined by subtracting PL from LL. The LL, PL, and PI of tested sample are presented on Figure B1, Atterberg Limits Results.

The results are also shown graphically on the Logs of Borings in Appendix A. For the purposes of soil description, Shannon & Wilson uses the term nonplastic to refer to soils with a PI less than 4, low plasticity for soils with a PI range of 4 to 10, medium plasticity for soils with a PI range of 10 to 20, high plasticity for soils with a PI greater than 20.

B.2.3 Particle-Size Analysis

Particle-size analyses were conducted on two samples to determine their grain-size distributions. Grain size distributions were determined in accordance with ASTM D6913. 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. Results of all particle-size analyses are presented on Figure B2, Grain Size Distribution. The percentage of each sample passing the No. 200 sieve is also shown graphically on the Logs of Borings in Appendix A.



NOTES

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

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

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

ATT MAIN 100623-001.GPJ SHAN WIL.GDT 4/5/19

BORING AND SAMPLE NO.	DEPTH (feet)	GROUP SYMBOL[2]	GROUP NAME[2]	LL %	PL %	PI %[3]	NAT. W.C. %	FINES %
● B-1, S-9	25.0	*CH*	*Fat Clay*	83	28	55	68	
■ B-2, S-8B	30.5	*CH*	*Fat Clay*	85	19	66	46	
▲ B-2, S-10	40.0	*CH*	*Fat Clay*	76	28	48	40	

City of Albany Riverfront Interceptor Sewer
Pump Station and Force Main
Albany, Oregon

ATTERBERG LIMITS RESULTS

April 2019
100623-001

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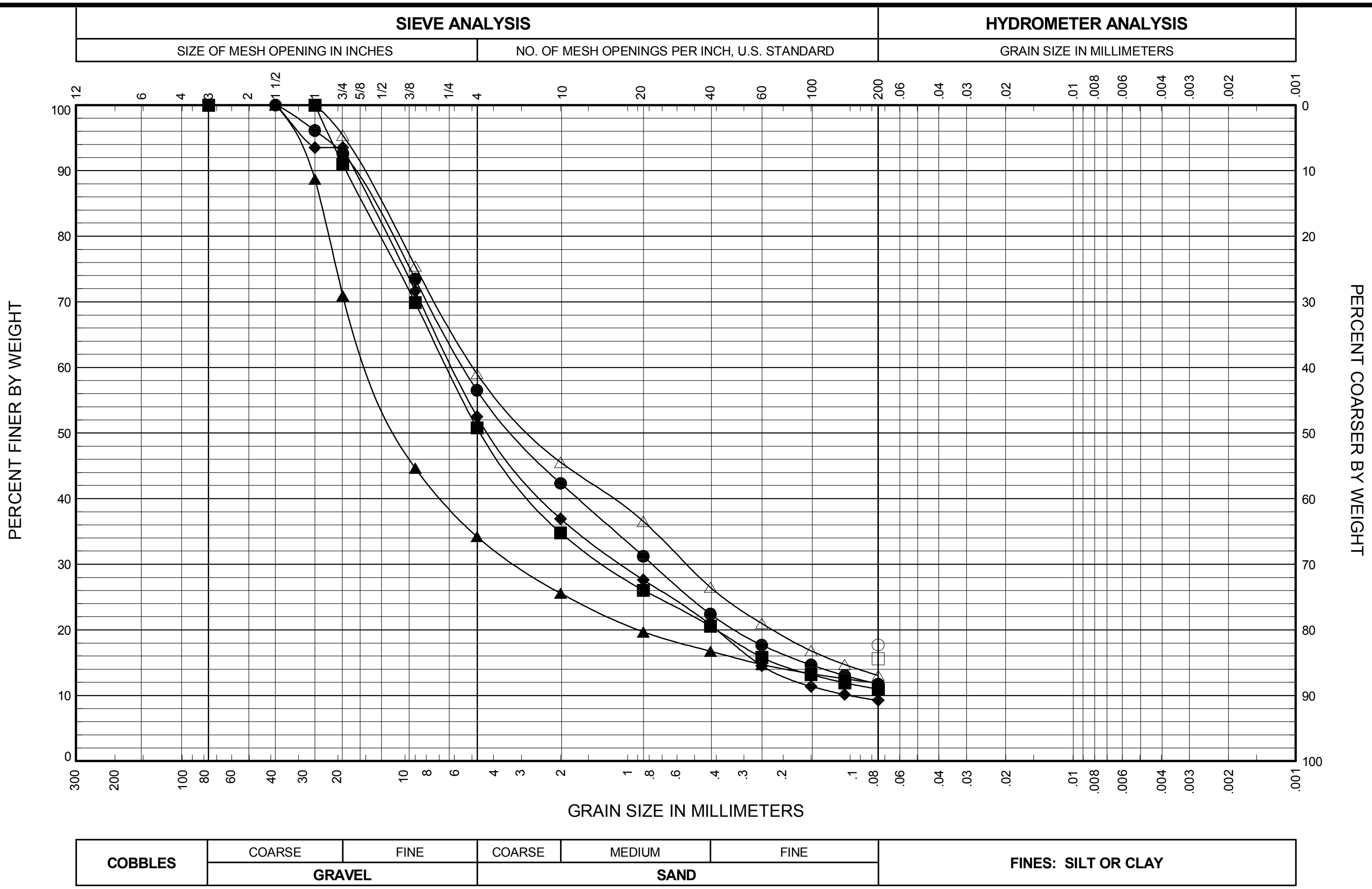
FIG. B1



BORING AND SAMPLE NO.	DEPTH (feet)	GROUP SYMBOL²	GROUP NAME²	GRAVEL %	SAND %	FINES %	NAT. W.C. %	DRY DENSITY PCF
● B-1, S-2*	5.0	*SW-SM*	*Well-Graded Sand with Silt and Gravel*	43	45	12	13	
■ B-1, S-5*	12.5	*GP-GM*	*Poorly-Graded Gravel with Silt and Sand*	49	40	11	15	
▲ B-1, S-6*	15.0	*GP-GC*	*Poorly-Graded Gravel with Clay and Sand*	66	22	12	14	
◆ B-1, S-8*	20.0	*GW-GM*	*Well-Graded Gravel with Silt and Sand*	47	43	9	18	
○ B-2, S-4*	10.0	*GM*	*Silty Gravel with Sand*	-	-	18	15	
□ B-2, S-5*	15.0	*GM*	*Silty Gravel with Sand*	-	-	16	20	
△ B-2, S-6*	20.0	*SM*	*Silty Sand with Gravel*	41	46	13	14	

City of Albany Riverfront Interceptor Sewer
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Albany, Oregon

GRAIN SIZE DISTRIBUTION

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FIG. B2
Sheet 1 of 2

FIG. B2

GSA MAIN 100623-001.GPJ SHAN WIL.GDT 4/5/19

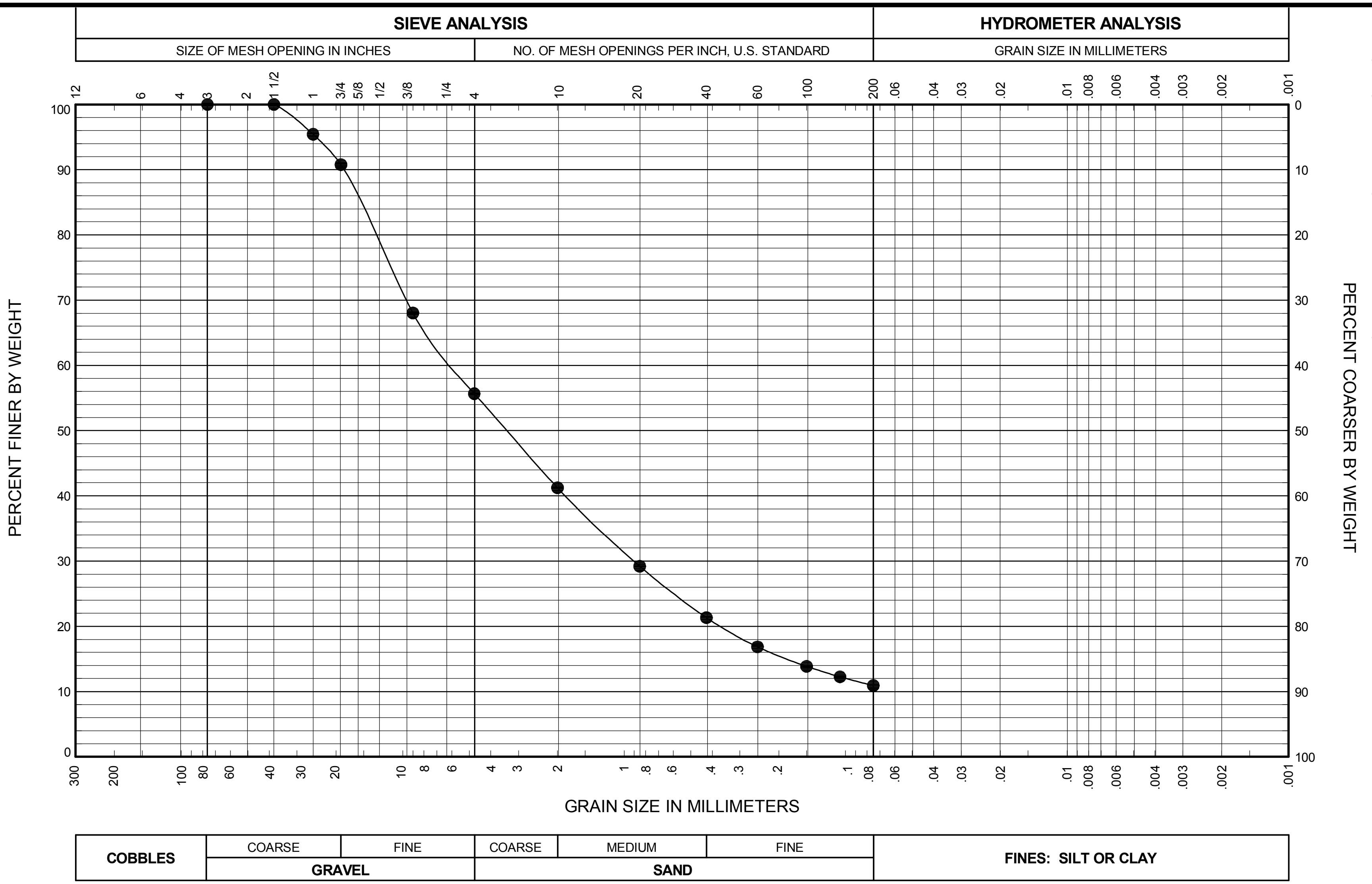
NOTES:
1) Sieve analyses were performed in general accordance with ASTM D6913, sieve with hydrometer analyses were performed in general accordance with ASTM D422, and amount finer than #200 sieve analyses were performed in general accordance with ASTM D1140 unless otherwise noted in the report.
2) Group Name and Group Symbol are in accordance with ASTM D2488 and are refined in accordance with ASTM D2487 where appropriate laboratory tests are performed.
** Sample specimen weight did not meet required minimum mass for ASTM test method.*

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.

** Sample specimen weight did not meet required minimum mass for ASTM test method.*



BORING AND SAMPLE NO.	DEPTH (feet)	GROUP SYMBOL[2]	GROUP NAME[2]	GRAVEL %	SAND %	FINES %	NAT. W.C. %	DRY DENSITY PCF
● B-3, S-7*	25.0	*SW-SM*	*Well-Graded Sand with Silt and Gravel*	44	45	11	12	

City of Albany Riverfront Interceptor Sewer
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GRAIN SIZE DISTRIBUTION

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FIG. B2
Sheet 2 of 2

FIG. B2

TestAmerica

THE LEADER IN ENVIRONMENTAL TESTING

ANALYTICAL REPORT

TestAmerica Laboratories, Inc.
TestAmerica Seattle
5755 8th Street East
Tacoma, WA 98424
Tel: (253)922-2310

TestAmerica Job ID: 580-80688-2
Client Project/Site: Shannon & Wilson, Inc

For:
Shannon & Wilson, Inc
400 N. 34th Suite 100
PO BOX 300303
Seattle, Washington 98103

Attn: Elliott Mecham

Kayse Zalmai

Authorized for release by:
10/19/2018 1:06:42 PM

Kayse Zalmai, Project Manager I
(253)922-2310
kayse.zalmai@testamericainc.com

LINKS

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www.testamericainc.com

Table of Contents

Case Narrative

Job ID: 580-80688-2

Laboratory: TestAmerica Seattle

Narrative

Job Narrative
580-80688-2

Receipt
The samples were received on 9/28/2018 2:30 PM. The temperature of the cooler at receipt was 24.2º C.

Receipt Exception
The following samples were analyzed outside of analytical holding time due to analysis being added past hold: Albany Riverfront B-2,S-3 (580-80688-1), Albany Riverfront B-3,S-6 (580-80688-2) and (580-80688-A-1-D DU).

HPLC/IC
No analytical or quality issues were noted, other than those described above or in the Definitions/Glossary page.

General Chemistry
Method(s) SM 2510B: Conductivity result was reported at a dilution and may have increased error compared to an undiluted sample. The following samples are impacted:

No additional analytical or quality issues were noted, other than those described above or in the Definitions/Glossary page.

Definitions/Glossary

Qualifiers

HPLC/IC

Qualifier	Qualifier Description
H	Sample was prepped or analyzed beyond the specified holding time

General Chemistry

Qualifier	Qualifier Description
H	Sample was prepped or analyzed beyond the specified holding time

Glossary

Abbreviation	These commonly used abbreviations may or may not be present in this report.
¤	Listed under the "D" column to designate that the result is reported on a dry weight basis
%R	Percent Recovery
CFL	Contains Free Liquid
CNF	Contains No Free Liquid
DER	Duplicate Error Ratio (normalized absolute difference)
Dil Fac	Dilution Factor
DL	Detection Limit (DoD/DOE)
DL, RA, RE, IN	Indicates a Dilution, Re-analysis, Re-extraction, or additional Initial metals/anion analysis of the sample
DLC	Decision Level Concentration (Radiochemistry)
EDL	Estimated Detection Limit (Dioxin)
LOD	Limit of Detection (DoD/DOE)
LOQ	Limit of Quantitation (DoD/DOE)
MDA	Minimum Detectable Activity (Radiochemistry)
MDC	Minimum Detectable Concentration (Radiochemistry)
MDL	Method Detection Limit
ML	Minimum Level (Dioxin)
NC	Not Calculated
ND	Not Detected at the reporting limit (or MDL or EDL if shown)
PQL	Practical Quantitation Limit
QC	Quality Control
RER	Relative Error Ratio (Radiochemistry)
RL	Reporting Limit or Requested Limit (Radiochemistry)
RPD	Relative Percent Difference, a measure of the relative difference between two points
TEF	Toxicity Equivalent Factor (Dioxin)
TEQ	Toxicity Equivalent Quotient (Dioxin)

Client Sample ID: Albany Riverfront B-2,S-3
Lab Sample ID: 580-80688-1
Date Collected: 09/10/18 12:00
Matrix: Solid
Date Received: 09/28/18 14:30

General Chemistry

Analyte	Result	Qualifier	RL		Unit	D	Prepared	Analyzed	Dil Fac
Percent Moisture	**22.7**		0.1		%			10/15/18 14:55	1
Percent Solids	**77.3**		0.1		%			10/15/18 14:55	1

General Chemistry - Soluble

Analyte	Result	Qualifier	RL		Unit	D	Prepared	Analyzed	Dil Fac
Specific Conductance	**210**	**H**	9.9		umhos/cm			10/15/18 08:05	1
Resistivity	**4800**	**H**	0.99		ohm cm			10/15/18 08:05	1
Oxidation Reduction Potential	**480**	**H**			millivolts			10/17/18 16:54	1

Client Sample Results

Client Sample ID: Albany Riverfront B-2,S-3

Lab Sample ID: 580-80688-1
Date Collected: 09/10/18 12:00
Matrix: Solid
Date Received: 09/28/18 14:30
Percent Solids: 77.3

Method: 300.0 - Anions, Ion Chromatography - Soluble

Analyte	Result	Qualifier	RL		Unit	D	Prepared	Analyzed	Dil Fac
Chloride	ND	H	6.5		mg/Kg	☼		10/13/18 04:10	1
Sulfate	**40**	**H**	6.5		mg/Kg	☼		10/13/18 04:10	1

General Chemistry

Analyte	Result	Qualifier	RL		Unit	D	Prepared	Analyzed	Dil Fac
Sulfide	ND	H	52		mg/Kg	☼	10/18/18 10:48	10/18/18 13:43	1

Client Sample ID: Albany Riverfront B-3,S-6

Lab Sample ID: 580-80688-2

Date Collected: 09/27/18 12:00

Matrix: Solid

Date Received: 09/28/18 14:30

General Chemistry

Analyte	Result	Qualifier	RL		Unit	D	Prepared	Analyzed	Dil Fac
Percent Moisture	14.6		0.1		%			10/15/18 14:55	1
Percent Solids	85.4		0.1		%			10/15/18 14:55	1

General Chemistry - Soluble

Analyte	Result	Qualifier	RL		Unit	D	Prepared	Analyzed	Dil Fac
Specific Conductance	180		9.6		umhos/cm			10/15/18 08:05	1
Resistivity	5600		0.96		ohm cm			10/15/18 08:05	1
Oxidation Reduction Potential	510	H			millivolts			10/17/18 16:54	1

Client Sample Results

Client Sample ID: Albany Riverfront B-3,S-6

Lab Sample ID: 580-80688-2

Date Collected: 09/27/18 12:00
Matrix: Solid
Date Received: 09/28/18 14:30
Percent Solids: 85.4

Method: 300.0 - Anions, Ion Chromatography - Soluble

Analyte	Result	Qualifier	RL	Unit	D	Prepared	Analyzed	Dil Fac
Chloride	ND		5.9	mg/Kg	☼		10/13/18 04:25	1
Sulfate	**30**		5.9	mg/Kg	☼		10/13/18 04:25	1

General Chemistry

Analyte	Result	Qualifier	RL	Unit	D	Prepared	Analyzed	Dil Fac
Sulfide	ND	H	47	mg/Kg	☼	10/18/18 10:48	10/18/18 13:43	1

QC Sample Results

Client: Shannon & Wilson, Inc
Project/Site: Shannon & Wilson, Inc

TestAmerica Job ID: 580-80688-2

Method: 300.0 - Anions, Ion Chromatography

Lab Sample ID: MB 440-504801/1-A
Matrix: Solid
Analysis Batch: 504635

Client Sample ID: Method Blank
Prep Type: Soluble

Analyte	MB Result	MB Qualifier	RL		Unit	D	Prepared	Analyzed	Dil Fac
Chloride	ND		5.0		mg/Kg			10/13/18 01:51	1
Sulfate	ND		5.0		mg/Kg			10/13/18 01:51	1

Lab Sample ID: LCS 440-504801/2-A
Matrix: Solid
Analysis Batch: 504635

Client Sample ID: Lab Control Sample
Prep Type: Soluble

Analyte			Spike Added	LCS Result	LCS Qualifier	Unit	D	%Rec	%Rec. Limits		
Chloride			50.0	46.7		mg/Kg		93	90 - 110		
Sulfate			50.0	48.8		mg/Kg		98	90 - 110		

Method: 9034 - Sulfide, Acid soluble and Insoluble (Titrimetric)

Lab Sample ID: MB 440-505993/1-A
Matrix: Solid
Analysis Batch: 506042

Client Sample ID: Method Blank
Prep Type: Total/NA
Prep Batch: 505993

Analyte	MB Result	MB Qualifier	RL		Unit	D	Prepared	Analyzed	Dil Fac
Sulfide	ND		40		mg/Kg		10/18/18 10:48	10/18/18 13:43	1

Lab Sample ID: LCS 440-505993/2-A
Matrix: Solid
Analysis Batch: 506042

Client Sample ID: Lab Control Sample
Prep Type: Total/NA
Prep Batch: 505993

Analyte			Spike Added	LCS Result	LCS Qualifier	Unit	D	%Rec	%Rec. Limits		
Sulfide			79.4	71.4		mg/Kg		90	80 - 120		

Lab Sample ID: LCSD 440-505993/3-A
Matrix: Solid
Analysis Batch: 506042

Client Sample ID: Lab Control Sample Dup
Prep Type: Total/NA
Prep Batch: 505993

Analyte			Spike Added	LCSD Result	LCSD Qualifier	Unit	D	%Rec	%Rec. Limits	RPD	RPD Limit
Sulfide			78.6	62.9		mg/Kg		80	80 - 120	13	20

Lab Sample ID: 580-80688-1 MS
Matrix: Solid
Analysis Batch: 506042

Client Sample ID: Albany Riverfront B-2,S-3
Prep Type: Total/NA
Prep Batch: 505993

Analyte	Sample Result	Sample Qualifier	Spike Added	MS Result	MS Qualifier	Unit	D	%Rec	%Rec. Limits		
Sulfide	ND	H	104	93.2		mg/Kg	☼	90	70 - 130		

Lab Sample ID: 580-80688-1 MSD
Matrix: Solid
Analysis Batch: 506042

Client Sample ID: Albany Riverfront B-2,S-3
Prep Type: Total/NA
Prep Batch: 505993

Analyte	Sample Result	Sample Qualifier	Spike Added	MSD Result	MSD Qualifier	Unit	D	%Rec	%Rec. Limits	RPD	RPD Limit
Sulfide	ND	H	102	92.1		mg/Kg	☼	90	70 - 130	1	30

QC Sample Results

Method: SM 2510B - Conductivity, Specific Conductance

Lab Sample ID: MB 440-505028/2-A
Matrix: Solid
Analysis Batch: 505154

Client Sample ID: Method Blank
Prep Type: Soluble

Analyte	MB Result	MB Qualifier	RL		Unit	D	Prepared	Analyzed	Dil Fac
Specific Conductance	ND		1.0		umhos/cm			10/15/18 08:05	1

Lab Sample ID: LCS 440-505028/3-A
Matrix: Solid
Analysis Batch: 505154

Client Sample ID: Lab Control Sample
Prep Type: Soluble

Analyte			Spike Added	LCS Result	LCS Qualifier	Unit	D	%Rec	%Rec. Limits		
Specific Conductance			953	948		umhos/cm		99	90 - 110		

Lab Sample ID: 580-80688-1 DU
Matrix: Solid
Analysis Batch: 505154

Client Sample ID: Albany Riverfront B-2,S-3
Prep Type: Soluble

Analyte	Sample Result	Sample Qualifier		DU Result	DU Qualifier	Unit	D			RPD	RPD Limit
Specific Conductance	210	H		203		umhos/cm				3	20
Resistivity	4800	H		4920		ohm cm				3	20

Method: SM 2580B - Reduction-Oxidation (REDOX) Potential

Lab Sample ID: 580-80688-2 DU
Matrix: Solid
Analysis Batch: 505837

Client Sample ID: Albany Riverfront B-3,S-6
Prep Type: Soluble

Analyte	Sample Result	Sample Qualifier		DU Result	DU Qualifier	Unit	D			RPD	RPD Limit
Oxidation Reduction Potential	510	H		508		millivolts				0.4	5

Lab Chronicle

Client Sample ID: Albany Riverfront B-2,S-3

Lab Sample ID: 580-80688-1
Date Collected: 09/10/18 12:00
Matrix: Solid
Date Received: 09/28/18 14:30

Prep Type	Batch Type	Batch Method	Run	Dilution Factor	Batch Number	Prepared or Analyzed	Analyst	Lab
Total/NA	Analysis	Moisture		1	505308	10/15/18 14:55	KM	TAL IRV
Soluble	Leach	DI Leach			505028	10/14/18 08:50	XL	TAL IRV
Soluble	Analysis	SM 2510B		1	505154	10/15/18 08:05	XL	TAL IRV
Soluble	Leach	DI Leach			505368	10/15/18 17:57	CMM	TAL IRV
Soluble	Analysis	SM 2580B		1	505837	10/17/18 16:54	ST	TAL IRV

Client Sample ID: Albany Riverfront B-2,S-3

Lab Sample ID: 580-80688-1
Date Collected: 09/10/18 12:00
Matrix: Solid
Date Received: 09/28/18 14:30
Percent Solids: 77.3

Prep Type	Batch Type	Batch Method	Run	Dilution Factor	Batch Number	Prepared or Analyzed	Analyst	Lab
Soluble	Leach	DI Leach			504801	10/12/18 20:41	HTL	TAL IRV
Soluble	Analysis	300.0		1	504635	10/13/18 04:10	NTN	TAL IRV
Total/NA	Prep	9030B			505993	10/18/18 10:48	KMY	TAL IRV
Total/NA	Analysis	9034		1	506042	10/18/18 13:43	KMY	TAL IRV

Client Sample ID: Albany Riverfront B-3,S-6

Lab Sample ID: 580-80688-2
Date Collected: 09/27/18 12:00
Matrix: Solid
Date Received: 09/28/18 14:30

Prep Type	Batch Type	Batch Method	Run	Dilution Factor	Batch Number	Prepared or Analyzed	Analyst	Lab
Total/NA	Analysis	Moisture		1	505308	10/15/18 14:55	KM	TAL IRV
Soluble	Leach	DI Leach			505028	10/14/18 08:50	XL	TAL IRV
Soluble	Analysis	SM 2510B		1	505154	10/15/18 08:05	XL	TAL IRV
Soluble	Leach	DI Leach			505368	10/15/18 17:57	CMM	TAL IRV
Soluble	Analysis	SM 2580B		1	505837	10/17/18 16:54	ST	TAL IRV

Client Sample ID: Albany Riverfront B-3,S-6

Lab Sample ID: 580-80688-2
Date Collected: 09/27/18 12:00
Matrix: Solid
Date Received: 09/28/18 14:30
Percent Solids: 85.4

Prep Type	Batch Type	Batch Method	Run	Dilution Factor	Batch Number	Prepared or Analyzed	Analyst	Lab
Soluble	Leach	DI Leach			504801	10/12/18 20:41	HTL	TAL IRV
Soluble	Analysis	300.0		1	504635	10/13/18 04:25	NTN	TAL IRV
Total/NA	Prep	9030B			505993	10/18/18 10:48	KMY	TAL IRV
Total/NA	Analysis	9034		1	506042	10/18/18 13:43	KMY	TAL IRV

Laboratory References:

TAL IRV = TestAmerica Irvine, 17461 Derian Ave, Suite 100, Irvine, CA 92614-5817, TEL (949)261-1022

Accreditation/Certification Summary

Client: Shannon & Wilson, Inc
Project/Site: Shannon & Wilson, Inc

TestAmerica Job ID: 580-80688-2

Laboratory: TestAmerica Seattle

All accreditations/certifications held by this laboratory are listed. Not all accreditations/certifications are applicable to this report.

Authority	Program	EPA Region	Identification Number	Expiration Date
Alaska (UST)	State Program	10	17-024	01-19-19
ANAB	DoD ELAP		L2236	01-19-19
ANAB	ISO/IEC 17025		L2236	01-19-19
California	State Program	9	2901	11-05-18
Montana (UST)	State Program	8	N/A	04-30-20
Nevada	State Program	9	WA000502019-1	07-31-19
Oregon	NELAP	10	WA100007	11-05-18
US Fish & Wildlife	Federal		LE058448-0	07-31-19
USDA	Federal		P330-14-00126	02-10-20
Washington	State Program	10	C553	02-17-19

Laboratory: TestAmerica Irvine

All accreditations/certifications held by this laboratory are listed. Not all accreditations/certifications are applicable to this report.

Authority	Program	EPA Region	Identification Number	Expiration Date
Alaska	State Program	10	CA01531	06-30-19
Arizona	State Program	9	AZ0671	10-14-18 *
California	LA Cty Sanitation Districts	9	10256	06-30-19
California	State Program	9	CA ELAP 2706	06-30-19
Guam	State Program	9	Cert. No. 17-003R	01-23-19
Hawaii	State Program	9	N/A	01-29-19
Kansas	NELAP	7	E-10420	07-31-19
Nevada	State Program	9	CA015312018-1	07-31-19
New Mexico	State Program	6	N/A	01-29-19
Oregon	NELAP	10	4028	01-29-19
US Fish & Wildlife	Federal		058448	07-31-19
USDA	Federal		P330-15-00184	07-09-21
Washington	State Program	10	C900	09-03-18 *

* Accreditation/Certification renewal pending - accreditation/certification considered valid.

Sample Summary

Client: Shannon & Wilson, Inc
Project/Site: Shannon & Wilson, Inc

TestAmerica Job ID: 580-80688-2

Lab Sample ID	Client Sample ID	Matrix	Collected	Received
580-80688-1	Albany Riverfront B-2,S-3	Solid	09/10/18 12:00	09/28/18 14:30
580-80688-2	Albany Riverfront B-3,S-6	Solid	09/27/18 12:00	09/28/18 14:30

Login Sample Receipt Checklist

Client: Shannon & Wilson, Inc Job Number: 580-80688-2

Login Number: 80688 **List Source: TestAmerica Seattle**
List Number: 1
Creator: O'Connell, Jason I

Question	Answer	Comment
Radioactivity wasn't checked or is </= background as measured by a survey meter.	N/A	Lab does not accept radioactive samples.
The cooler's custody seal, if present, is intact.	True	
Sample custody seals, if present, are intact.	True	
The cooler or samples do not appear to have been compromised or tampered with.	True	
Samples were received on ice.	False	No Coolant
Cooler Temperature is acceptable.	False	Cooler temperature outside required temperature criteria.
Cooler Temperature is recorded.	True	
COC is present.	True	
COC is filled out in ink and legible.	True	
COC is filled out with all pertinent information.	True	
Is the Field Sampler's name present on COC?	True	
There are no discrepancies between the containers received and the COC.	True	
Samples are received within Holding Time (excluding tests with immediate HTs)	True	
Sample containers have legible labels.	True	
Containers are not broken or leaking.	True	
Sample collection date/times are provided.	True	
Appropriate sample containers are used.	True	
Sample bottles are completely filled.	True	
Sample Preservation Verified.	N/A	
There is sufficient vol. for all requested analyses, incl. any requested MS/MSDs	True	
Containers requiring zero headspace have no headspace or bubble is <6mm (1/4").	N/A	
Multiphasic samples are not present.	True	
Samples do not require splitting or compositing.	True	
Residual Chlorine Checked.	N/A	

Login Sample Receipt Checklist

Client: Shannon & Wilson, Inc

Job Number: 580-80688-2

Login Number: 80688
List Number: 2
Creator: Ornelas, Olga

List Source: TestAmerica Irvine
List Creation: 10/02/18 11:51 AM

Question	Answer	Comment
Radioactivity wasn't checked or is </= background as measured by a survey meter.	True	
The cooler's custody seal, if present, is intact.	True	
Sample custody seals, if present, are intact.	N/A	Not Present
The cooler or samples do not appear to have been compromised or tampered with.	True	
Samples were received on ice.	True	
Cooler Temperature is acceptable.	True	
Cooler Temperature is recorded.	True	
COC is present.	True	
COC is filled out in ink and legible.	True	
COC is filled out with all pertinent information.	True	
Is the Field Sampler's name present on COC?	N/A	Received project as a subcontract.
There are no discrepancies between the containers received and the COC.	True	
Samples are received within Holding Time (excluding tests with immediate HTs)	True	
Sample containers have legible labels.	True	
Containers are not broken or leaking.	True	
Sample collection date/times are provided.	True	
Appropriate sample containers are used.	True	
Sample bottles are completely filled.	True	
Sample Preservation Verified.	N/A	
There is sufficient vol. for all requested analyses, incl. any requested MS/MSDs	True	
Containers requiring zero headspace have no headspace or bubble is <6mm (1/4").	True	
Multiphasic samples are not present.	True	
Samples do not require splitting or compositing.	True	
Residual Chlorine Checked.	N/A	

Appendix C

Hydraulic Conductivity (SLUG) Test Results

CONTENTS

Tables

Figures

APPENDIX C

C.1 HYDRAULIC CONDUCTIVITY (SLUG) TESTING

Hydraulic conductivity is a parameter used in many equations that describe the flow of groundwater. Expressed in units of length over time, hydraulic conductivity is essentially the distance water will travel through a soil over a given time, under a 1 horizontal to 1 vertical (1H:1V) hydraulic gradient. A form of hydraulic conductivity testing, commonly referred to as "slug testing," was performed in observation well B-1 on September 12, 2018, to provide an estimate of hydraulic conductivity for the water-bearing zones screened by the well. Construction details for the tested wells are presented on Figure A2 in Appendix A. This appendix describes well development and slug testing procedures and presents estimated hydraulic conductivities based on the test data.

C.1.1 Well Development

After the observation well was installed, Shannon & Wilson developed it by working a surge block system up and down the screened section. The surge block system contains a cylindrical block that is slightly narrower than the well casing diameter. The block is attached to a high-density polyethylene (HDPE) tube with a check-valve at the bottom. By moving the assembly up and down in the well screen interval, water is surged in and out of the filter pack while water is simultaneously removed through the tubing.

After surging the well, Shannon & Wilson further purged several well-volumes of water using a down-hole pump. This process of surging and pumping helps to improve the consistency of the communication between the well and the aquifer, making it more reliable for aquifer testing. After well development, sufficient time was allowed for the groundwater to return to the static level before testing.

C.1.2 Data Collection

Shannon & Wilson used a hand-held electronic water-level indicator to measure the static water level prior to the start of the first slug test. For each slug test, an electronic datalogger/transducer (Solinst Levelogger®) was placed down the test well, several feet below the static groundwater level.

With the datalogger recording data at specified time intervals, Shannon & Wilson displaced a known volume of water in the well by fully submerging a dimensionally measured, sand-filled polyvinyl chloride (PVC) pipe (slug) suspended from an eye bolt and nylon line. Then the water level in the well was allowed to fall back to the static level, with the recovery curve being recorded by the datalogger. This is the falling head slug test.

Following recovery of the water level back to the static level, Shannon & Wilson initiated a rising head slug test by rapidly removing the slug from the well. The water level in the well was allowed to rise back to the static level, with the recovery curve again being recorded by the datalogger. To ensure that the datalogger was working properly and that the test was proceeding as intended, Shannon & Wilson occasionally collected manual measurements during the tests using the hand-held electronic water-level indicator.

At least three falling and three rising head slug tests were performed in observation wells. Typically, one set of slug tests was performed using an 8-foot-long slug, but in boring B-1, two sets were performed with an 8-foot slug, and one set was performed using a 4-foot-long slug.

C.1.3 Data Analysis

The data collected show the induced changes in the water level and subsequent return to the static level over time. Hydraulic conductivities were estimated from the data using the method of Bouwer and Rice (1976). In the Bouwer and Rice method, slug test data is plotted on a semi-log plot, with normalized change in head on a logarithmically scaled y-axis and time on an x-axis. A straight line interpretation is fit to the data.

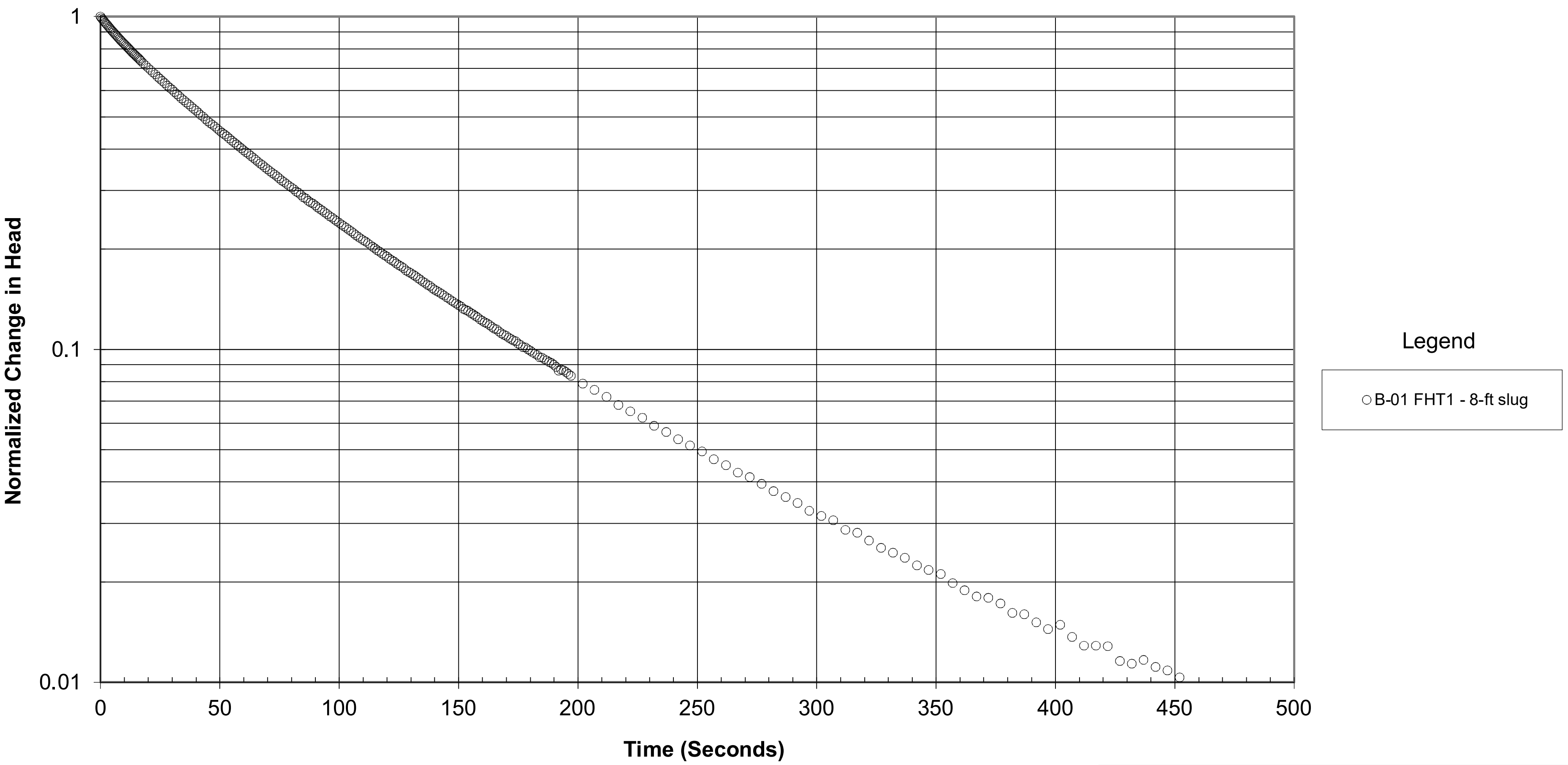
Hydraulic conductivity is then estimated from an equation with inputs that include the slope of the fit line and various well parameters. Semi-log plots of normalized change in head versus time are presented in Figures C1 through C6. Shannon & Wilson estimates of hydraulic conductivity for each test, based on calculations after Bouwer and Rice (1976), are summarized in Table C1, Hydraulic Conductivity (Slug) Test Results. These values are overall estimates for materials screened by the well. The hydraulic conductivities of individual strata penetrated by the well screen will vary depending on factors such as grain size distribution, grain shape, and fines content.

In boring B-1, a well screen 10-feet in length was installed from approximately 19.0 feet to 29.0 feet. The screened interval lies within two distinct lithological units with the upper 5 feet lying within the Linn Gravel (GW-GM), and the lower 5 feet lying within the Yamhill Formation (CH). The hydraulic conductivity is estimated to be significantly different in the two lithological units, so we performed two analyses for boring B-1 using a saturated screen interval of 10 feet and a saturated screen interval of 5 feet, and presented both sets of data in Table C1.

TABLE C1: HYDRAULIC CONDUCTIVITY (SLUG) TEST RESULTS

Observation Well	Test Type and Designation	Slug Length (feet)	Estimated Hydraulic Conductivity (feet/day)	Saturated Screen Interval Used for Analysis (feet)	Average Hydraulic Conductivity (feet/day)
B-01	FHT1	8	1.35	10	1.42
B-01	FHT2	8	1.40	10	
B-01	FHT3	4	1.35	10	
B-01	RHT1	8	1.45	10	
B-01	RHT2	8	1.40	10	
B-01	RHT3	4	1.48	10	
B-01	FHT1	8	2.14	5	2.25
B-01	FHT2	8	2.22	5	
B-01	FHT3	4	2.14	5	
B-01	RHT1	8	2.30	5	
B-01	RHT2	8	2.22	5	
B-01	RHT3	4	2.25	5	

FHT = Falling Head Test
RHT = Rising Head Test



Notes:
1. FHT = falling head test; RHT = rising head test

Albany Riverfront Interceptor
Sewer Pump Station and Force Main
Albany, Oregon

OBSERVATION WELL B-01
FALLING HEAD SLUG TEST 1

December 2018 100623

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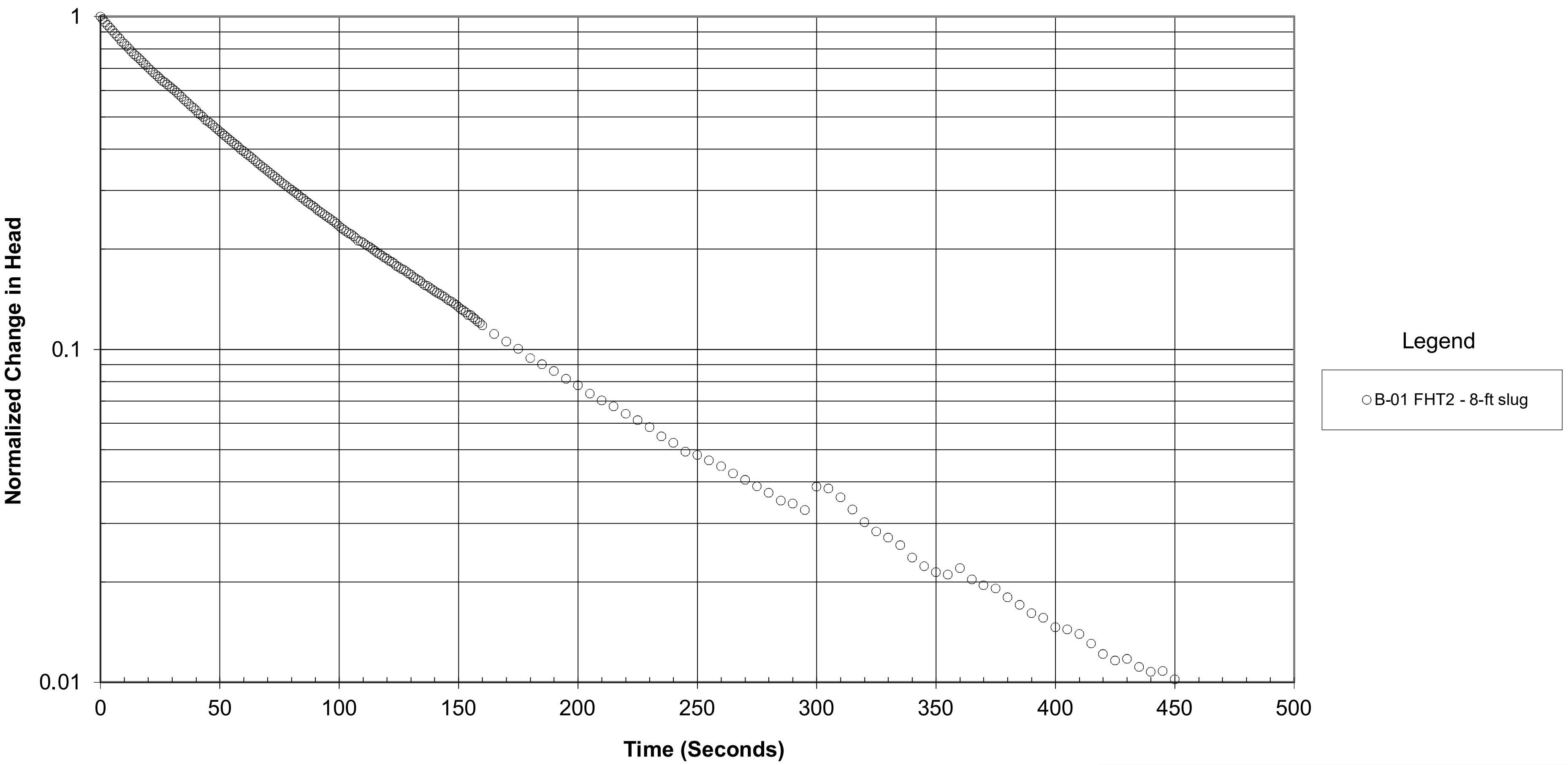
Geotechnical and Environmental Consultants

FIG. C1



Notes:
1. FHT = falling head test; RHT = rising head test

Albany Riverfront Interceptor
Sewer Pump Station and Force Main
Albany, Oregon

OBSERVATION WELL B-01
FALLING HEAD SLUG TEST 2

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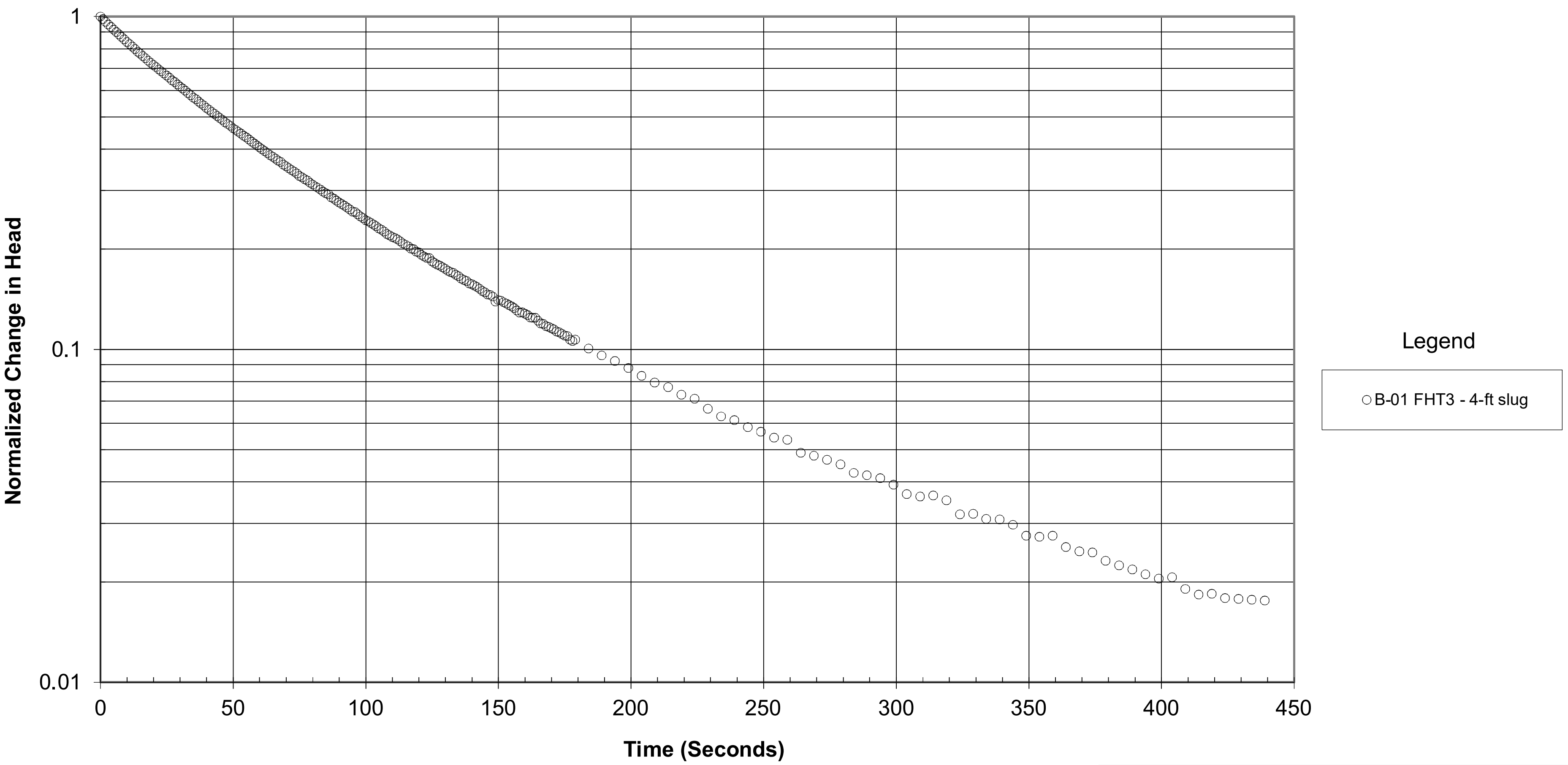
FIG. C2



Notes:
1. FHT = falling head test; RHT = rising head test

Albany Riverfront Interceptor
Sewer Pump Station and Force Main
Albany, Oregon

OBSERVATION WELL B-01
FALLING HEAD SLUG TEST 3

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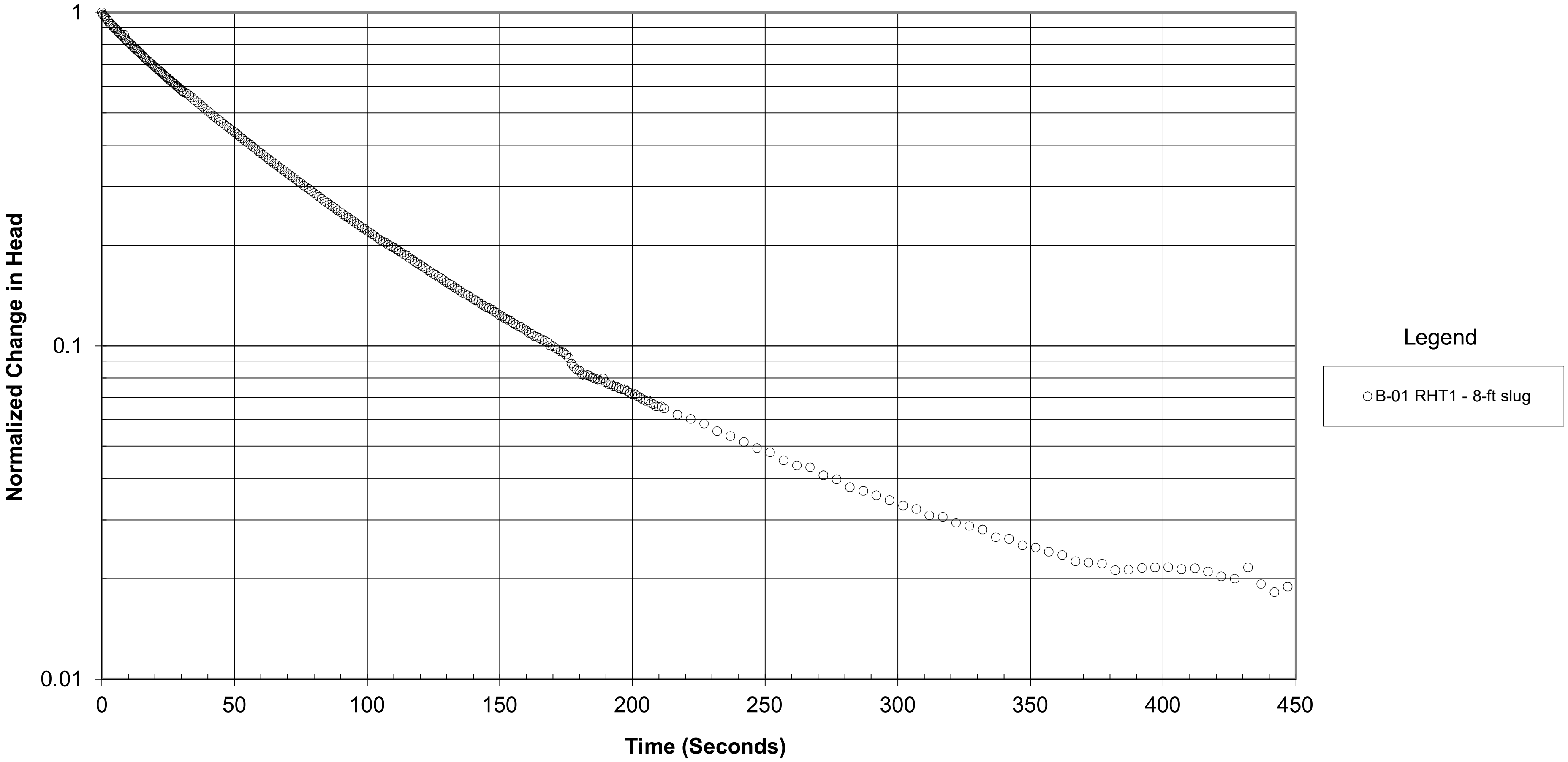
FIG. C3

FIG. C3



Notes:
1. FHT = falling head test; RHT = rising head test

Albany Riverfront Interceptor
Sewer Pump Station and Force Main
Albany, Oregon

OBSERVATION WELL B-01
RISING HEAD SLUG TEST 1

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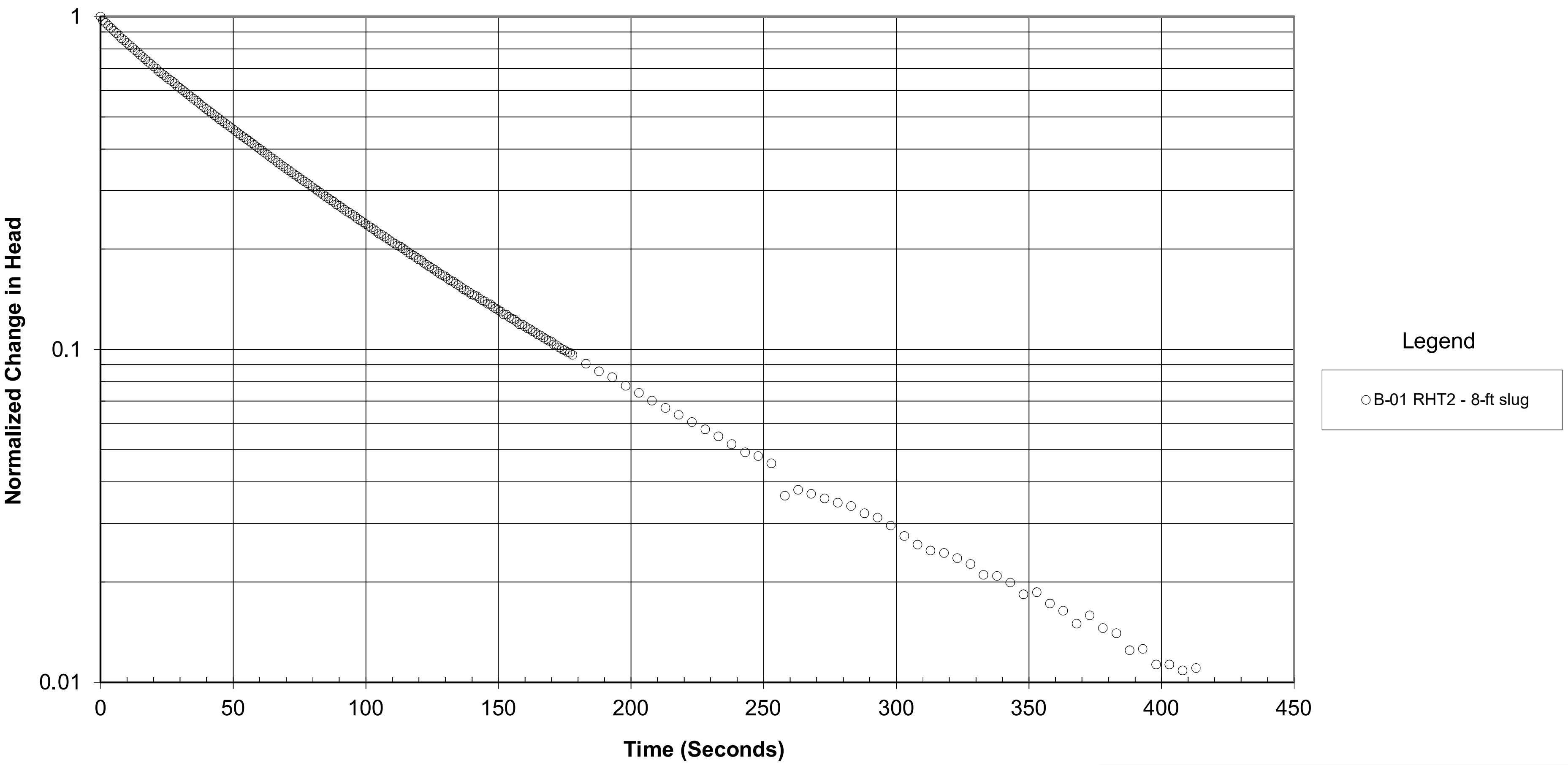
FIG. C4

FIG. C4



Notes:
1. FHT = falling head test; RHT = rising head test

Albany Riverfront Interceptor
Sewer Pump Station and Force Main
Albany, Oregon

OBSERVATION WELL B-01
RISING HEAD SLUG TEST 2

December 2018

100623

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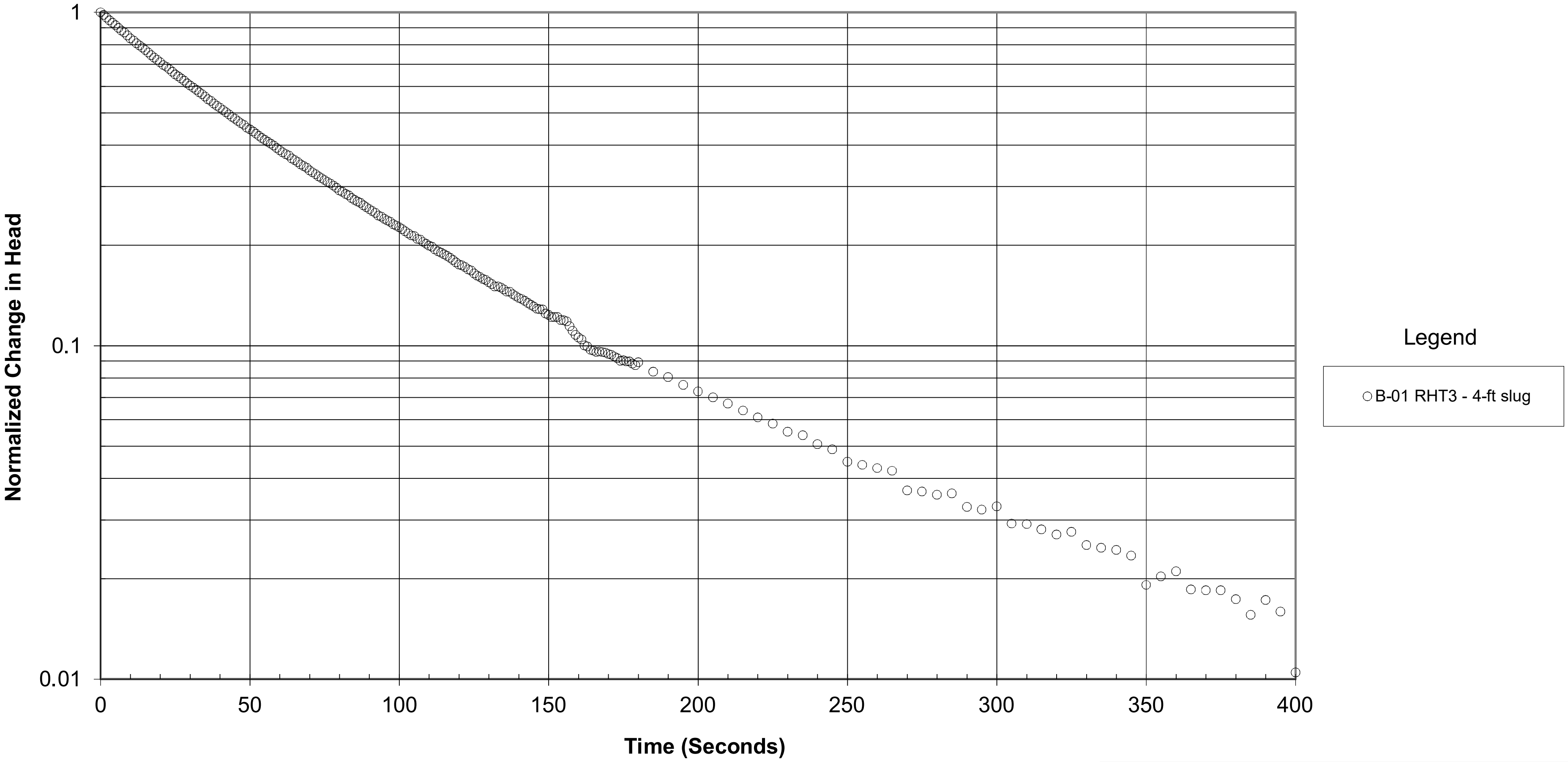
FIG. C5

FIG. C5

FIG. C6



Notes:
1. FHT = falling head test; RHT = rising head test

Albany Riverfront Interceptor
Sewer Pump Station and Force Main
Albany, Oregon

OBSERVATION WELL B-01
RISING HEAD SLUG TEST 3

December 2018 100623

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FIG. C6

IMPORTANT INFORMATION

Important Information

About Your Geotechnical/Environmental Report

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

Attachment to and part of Report: 100623
Date: April 2019
To: West Yost Associates
Mr. Matt Hewitt, Associate Engineer

IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL/ENVIRONMENTAL REPORT

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 which were considered in the development of the report have changed.

SUBSURFACE CONDITIONS CAN CHANGE.

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

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

MOST RECOMMENDATIONS ARE PROFESSIONAL JUDGMENTS.

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

A REPORT'S CONCLUSIONS ARE PRELIMINARY.

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

THE CONSULTANT'S REPORT IS SUBJECT TO MISINTERPRETATION.

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

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

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

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

READ RESPONSIBILITY CLAUSES CLOSELY.

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

The preceding paragraphs are based on information provided by the
ASFE/Association of Engineering Firms Practicing in the Geosciences, Silver Spring, Maryland