Geotechnical Investigation and Seismic Hazard Study

Lebanon Water Treatment Plant

Lebanon, Oregon

Prepared for:

Carollo Engineers, Inc.
Portland, Oregon

April 22 2016

Professional Geotechnical Services

Foundation Engineering, Inc.

Foundation Engineering, Inc.
Professional Geotechnical Services

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720 SW Washington St., Suite 550
Portland, Oregon 97205

April 22, 2016

Lebanon Water Treatment Plant
Geotechnical Investigation and Seismic Hazard Study
Lebanon, Oregon

Project 2131070-101

Dear Mr. Gresh:

We have completed the requested geotechnical investigation for the above-referenced project. Our report includes a description of our work, a discussion of site conditions, a summary of laboratory and field testing and a discussion of key geotechnical issues pertaining to the proposed project. Recommendations for site preparation, foundation design and construction are also provided.

The project is classified as an essential facility. Therefore, we have completed a site-specific seismic hazard study for the project in accordance with the 2014 Oregon Structural Specialty Code (OSSC). The OSSC is based on the 2012 International Building Code (IBC).

It has been a pleasure assisting you with this phase of your project. Please do not hesitate to contact us if you have any questions or if you require further assistance.

Sincerely,

FOUNDATION ENGINEERING, INC.

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FOUNDATION INVESTIGATION AND SEISMIC HAZARD STUDY

LEBANON WATER TREATMENT PLANT
LEBANON, OREGON

BACKGROUND

The City of Lebanon is planning to construct a new water treatment plant (WTP), an intake structure, a pump station, and associated water and wastewater transmission lines in Lebanon, Oregon. The locations of the project elements are shown on Figure 1A (Appendix A).

Foundation Engineering previously completed preliminary geotechnical work for the project. That work included test pits and borings at the WTP site to evaluate the subsurface conditions, determine the suitability of the site, and identify geotechnical considerations associated with the proposed development of the WTP facility. The findings of the preliminary work were summarized in a Preliminary Geotechnical Data Report dated April 18, 2014.

The design of the project has progressed since the completion of the preliminary geotechnical work. The layout for the WTP has been established and the intake structure, pump station, and transmission lines have been added.

Carollo Engineers, Inc. (Carollo) is the principal designer. Foundation Engineering, Inc. was retained by Carollo as the geotechnical consultant. Our scope of work was outlined in a proposal dated August 6, 2015, and authorized by a signed agreement dated September 21, 2015.

PROJECT ELEMENTS

The following sections provide a brief description of the various project elements. The proposed layouts and alignments are shown on Figures 2A through 7A (Appendix A).

Water Treatment Plant

The new WTP will include a water storage tank, a process building, a pre-treatment facility, and settling basins. A UV reactor may be added in the future. The proposed site layout is shown on Figure 2A. The WTP project elements are described below:

Water Storage Tank

The water storage tank will be a 65-foot diameter, glass-lined, steel structure with 20-foot tall walls. The tank will be supported on a perimeter ring footing with a thin slab membrane in the interior. The tank will have a finish floor elevation (FFE) at El. 366.5. The current ground elevations within the tank footprint range from ±El. 363 to ±El. 364. Therefore, the site grade will be raised by ±2.5 to 3.5 feet in this area. The tank will have a maximum freeboard water height of 17.5 feet, corresponding to El. 384.0.

Process Building

The process building will be a single-story, steel-framed structure. The building will have shallow foundations and a slab-on-grade floor. Column loads are expected to be 50 kips or less. The FFE will be at El. 366.5. The current ground elevations within the building footprint range from ±El. 363 to ±El. 364. Therefore, the site grade will be raised by ±2.5 to 3.5 feet in this area. The process building will include below-grade components, including two containment areas extending 4 feet below the FFE and a pipe gallery extending 7 feet below the FFE.

Pre-Treatment Facility

The pre-treatment equipment will be supported on a mat slab. The FFE varies from El. 366.0 to El 366.5. The current ground within the mat footprint lies at ±El. 363. Therefore, the site grade will be raised by ±3.5 feet in this area. Pre-treatment is an additive bid. We understand the base bid will include two above-ground wet wells (glass lined tanks) supported on the pre-treatment area slab.

Future UV Reactor

UV reactors may be added in the future, if needed, to improve the water quality. Similar to the pre-treatment facility, the UV reactor equipment will be supported on a mat slab with a FFE at El. 366.5. The current ground elevations within the mat footprint range from ±El. 363 to ±El. 363.5. Therefore, the site grade will be raised by ±3 to 3.5 feet in this area.

Settling Basins

Two settling basins are planned. The basins will be ±35 feet wide by ±140 feet long at the base with 2(H):1(V) side slopes on three sides and a ramp graded at 10(H):1(V) to provide access for cleaning. Preliminary plans indicate the FFE at the base of the basins will range from El. 361.0 (at the bottom of the access ramp) to El. 360.0 (at the low point). The top of the basins will be at El. 367.3 and the adjacent ground will be at El. 366.8. A settling basin sump will be constructed upstream of the settling basins with a FFE of El. 343.5. A decant pump station will also be constructed with a FFE of El. 347.5. The decant box will have a FFE of El. 357.3.

We understand the basins will be lined with ±6 inches of fiber-reinforced concrete, underlain by ±6 inches of Base Rock. At least 6 inches of Drain Rock will underlie the Base Rock at the bottom of the basins. The current ground elevations within the basin areas range from ±El. 364 to ±El. 365. Therefore, the basin excavations will extend ±5.5 to 6.5 feet below the current ground and ±1 to 2 feet of fill will be required for site grading around the perimeter of the basins.

An underdrain system will likely be installed in the drain rock beneath the settling basins. Water from the drain system will be directed to a below-grade sump pump. Preliminary plans indicate the base of the sump pump will be at El. 352.

Storm Water Swales

Storm water swales are planned at the WTP site to detain and infiltrate surface runoff. The swales will be constructed west of the tank and settling basins and in the northeast and southeast corners of the site. The swales will have a base elevation ranging from ±El. 361 to ±El. 363. The base of the drain rock placed in the bottom of the lower (El. 361) swale may extend to El. 385.5. The current ground elevations within the swale areas range from ±El. 363 to ±El. 365. Therefore, the swale excavations will extend ±1 to 6.5 feet below the current grades.

Pavement

Paved driveways and parking lots will wrap around the new process building and provide access to the site.

Intake Structure

A new intake facility is planned north of River Drive, ±3,400 feet east of the WTP site. The intake will be constructed on the south bank of the South Santiam River. The plans indicate the intake will be supported on a concrete mat slab with a FFE at ±El. 339.3 and a base elevation of ±El. 333.0. A concrete key may be constructed at the bottom of the mat foundation to help resist lateral loads. The base elevation for the intake is ±8 feet below the adjacent river bed. Riprap will be used to backfill the structure excavation in the river channel to provide scour protection. The structure will step up the river bank and an at-grade access platform will be built adjacent to River Drive with a FFE at ±El. 366.1.

A 36-inch diameter raw water pipe will extend from the intake structure beneath River Drive to the new pump station on the south side of River Drive. Where it leaves the intake, the pipe will have a flowline elevation matching the intake FFE (El. 338.47). The pipe will dip to the south with an invert elevation of El. 337.75 at the manhole on the south side of River Drive. The paved surface of River Drive lies at ±El. 367. Therefore, the base of the pipe will be ±28.5 to 29 feet below the road surface. Trenchless construction methods are planned to install the pipe.

Raw Water Pump Station

The new raw water pump station will be located ±110 feet south of the new intake structure. Preliminary drawings indicate the pump station will have a 10-foot wide by ±70-foot long wet well with a FFE at ±El. 329.0 and a base elevation at ±El. 326.0.

A pump station building will be constructed above the wet well. The building will be a single-story, steel-framed structure. The building will have shallow foundations and a slab-on-grade floor. The FFE will be at El. 364.5. The current ground elevations within the building footprint range from ±El. 361.5 to ±El. 365. Therefore, the site grades will be raised by up to ±3 feet in this area.

The pump station will be fed by the 36-inch diameter raw water pipe from the intake structure. The raw water pipe will pass through a 60-inch diameter manhole with a base elevation of ±El. 337.0 and a 72-inch diameter manhole with a base elevation of ±El. 334.25. The pipe will connect to the wet well with an invert elevation of ±El. 335.0. A 12-inch diameter pipe will also extend from the pump station to the intake structure at a shallower depth to provide backwash for sediment control.

A paved driveway and parking lots will provide access to the site. The new driveway will follow the existing gravel-surfaced driveway along the west side of the property. The parking lot will be on the south side of the pump station.

Storm water swales are planned at the pump station site to detain and infiltrate surface runoff. The swales at the pump station site will be constructed east of the new pump station. These swales will have a base elevation of ±El. 359.0. The current ground elevations within the swale areas range from ±El. 361 to ±El. 365. Therefore, the swale excavations will extend ±2 to 6 feet below the current grades.

Raw Water Pipeline

A 24-inch diameter raw water pipeline will extend between the pump station and the WTP. The new pipe will follow River Drive to the Santiam-Albany Canal. The pipe will cross under the Canal and then again follow River Drive to the new WTP. The bottom of the pipe is expected to extend ±6.5 to 7.5 feet below the road surface on River Drive. The top of the pipe will extend ±3 feet below the base of the Canal at the undercrossing.

Open-trench construction is planned along River Drive. Trenchless construction is planned for the Canal undercrossing. A 60-inch diameter casing is anticipated at the Canal undercrossing.

Treated Water Main and Sanitary Sewer Main

A new treated water main and sanitary sewer main will extend from the WTP along River Drive and Russell Drive. The top of the water main is expected to extend ±3 to 5 feet below the road surface. The top of the sewer main is expected to extend ±9 to 11 feet below the road surface. The sewer main will be shallower (±5 to 7 feet deep) at the WTP site. Storm drainage piping will also be installed as part of the work.

Open-trench construction is planned at the WTP site and along River Drive and Russell Drive. Trenchless construction may be used for the undercrossing of the railroad tracks on Russell Drive.

LOCAL GEOLOGY

An abbreviated discussion of the local geology is provided below. Detailed discussions of the regional geology, tectonic setting, local faulting and historical seismicity are presented in the Seismic Hazard Study Report (Appendix E). References cited in this section are provided in Appendix E.

The City of Lebanon, including the project area, is located within the transition between the Southern Willamette Valley and the west foothills of the Western Cascades. The South Santiam River flows west along the north edge of the project area and drains into the Willamette River ±19 miles to the northwest.

Most of the City is underlain by Quaternary alluvial deposits consisting of silt, sand and gravel with localized deeply weathered zones (Beaulieu et al., 1974; Yeats et al., 1996; Ferns and McClaughry, 2008). Younger alluvium is found along the South Santiam River and associated tributaries (Beaulieu et al., 1974; Yeats et al., 1996; Ferns and McClaughry, 2008).

Bedrock outcropping in the northeast hills and underlying the City is composed of ±47 to 33-million year old (Eocene to Oligocene), non-marine sedimentary and volcanic deposits of the Fisher Formation (Ferns and McClaughry, 2008) and associated older, volcanic deposits of the Little Butte Volcanics (Beaulieu et al., 1974; Yeats et al., 1996; Ferns and McClaughry, 2008; McClaughry et al., 2010). Our exploratory drilling and review of local water well logs available from the Oregon Water Resources Department website indicate the area is underlain by alluvial silt, sand and gravel. Volcaniclastic sandstone underlies the alluvium at the intake and pump station sites and along a portion of the raw water line at depths of ±23 to 42 feet. The depth to bedrock at the WTP site is unknown (Ferns and McClaughry, 2008; McClaughry et al., 2010), but our exploratory drilling indicates the rock depth exceeds 61 feet.

The subsurface conditions encountered in our explorations are consistent with the mapped local geology. Details are provided in the Subsurface Conditions section of this report.

FIELD EXPLORATION

The field exploration program was completed in two phases. The preliminary (2013) investigation, included exploratory test pits and boreholes at the WTP site. The layout and location of the WTP facilities had not been established at the time of that exploration. Therefore, the explorations were laid out on a grid pattern to evaluate the overall site conditions.

Following the preliminary investigation, the layout and locations of the WTP facilities were established and the intake facility, pump station, and transmission lines were added. The second phase of the investigation included additional test pits and borings completed in 2015, to evaluate the subsurface conditions at each project element location. The explorations are described in the following sections.

Each of the borings and test pits was continuously logged in field. The final logs (Appendix B) were prepared based on a review of the field logs, the laboratory test results, and an examination of the soil and rock samples in our office. The boring and test pit locations were surveyed by Crawford, Drummond and Associates, Inc. The surveyed ground elevations are shown on the logs.

Water Treatment Plant

Test Pits

Sixteen (16) exploratory test pits (TP-1 through TP-16) were dug at the WTP site on November 1, 2013, for the preliminary investigation work. The test pits were dug using a Case CX 130B excavator provided by the City.

An additional twelve (12) test pits (TP-17 through TP-28) were dug at the site on October 1, 2015, to provide supplemental information. The test pits were dug using a Kubota KX080-3 track-mounted excavator.

Three (3) exploratory test pits (P-5 through P-7) were also completed as part of infiltration testing on December 4, 2015. The test pits were dug using a Case 580 backhoe provided by the City.

The test pit locations at the WTP site are shown on Figure 2A. The test pits extended to depths of ±4 to 13 feet and all terminated in alluvial gravel. The subsurface profiles were logged and disturbed soil samples were obtained for possible laboratory testing. Field vane measurements were made on the test pit side walls at selected depths to estimate the undrained shear strength of the fine-grained soils. The soil profiles, sampling depths and strength measurements are summarized on the test pit logs (Appendix B).

Borings

Two (2) exploratory boreholes (BH-1 and BH-2) were drilled in the northwest and southeast quadrants of the WTP site on November 6, 2013. The drilling was completed using a CME 55 track-mounted drill rig with mud-rotary drilling techniques. The borings were used to supplement the test pits.

At the completion of drilling, BH-1 and BH-2 were flushed with water to remove drilling fluid and 1-inch diameter, standpipe PVC piezometers were installed to allow monitoring of ground water levels. The standpipes extended to a depth of ±20 feet and were slotted from ±10 to 20 feet. The installations were capped with flush-mount Morris monuments set in concrete.

Two (2) additional borings (BH-3 and BH-4) were drilled at the site on September 28, 2015. The drilling was completed using a CME 75 truck-mounted drill rig with mud-rotary drilling techniques. BH-3 was drilled within the proposed water storage tank footprint and BH-4 was drilled in the proposed process building footprint. These borings were completed to evaluate the foundation conditions beneath these structures and provide deeper subsurface information for the seismic hazard study. The borehole locations are shown on Figure 2A.

The boreholes extended to depths of ±31.5 to 60.9 feet. Samples were typically obtained at 2½-foot intervals to ±5 to 25 feet and at 5-foot intervals thereafter. Disturbed samples were obtained with a split-spoon sampler. The Standard Penetration Test (SPT), which is run when the split-spoon is driven, provides an indication of the relative stiffness or density of the soils. Relatively undisturbed Shelby tube samples were obtained in BH-1 and BH-3 at selected depths. The soil profiles, sampling depths and strength measurements are summarized on the boring logs (Appendix B).

Intake Structure

We drilled one (1) exploratory borehole (BH-5) near the proposed intake on September 24, 2015. The drilling was completed using a CME 75 truck-mounted drill rig with hollow-stem auger and mud-rotary drilling and HQ wire-line coring techniques. The borehole was located ±58 feet southeast of the proposed intake structure on a relatively flat bench, north of River Drive. The location is shown on Figure 3A.

The borehole extended to a maximum depth of ±56 feet. Samples were obtained at 2½-foot intervals to ±25 feet and at 5-foot intervals thereafter. Disturbed samples were obtained with a split-spoon sampler during SPT testing. HQ-size, wire line coring was completed from ±36 to 56 feet to obtain bedrock core samples for further evaluation and possible laboratory testing. The boring log is included in Appendix B.

Falling head permeability tests were run through the hollow-stem augers at depths of ±25 and ±30 feet. The information was used to estimate the hydraulic conductivity of the alluvial gravel. The results are discussed in a subsequent section of this report.

At the completion of drilling, BH-5 was flushed with water to remove drilling fluid and a 1-inch diameter, standpipe PVC piezometer was installed to allow monitoring of ground water levels. The standpipe extended to a depth of ±30 feet, and was slotted from ±20 to 30 feet. The installation was capped with a flush-mount with Morris monument set in concrete.

Raw Water Pump Station

Test Pits

Four (4) exploratory test pits (TP-29 through TP-32) were dug at the pump station site on October 1, 2015, using a Kubota KX080-3 track-mounted excavator. The test pit locations are shown on Figure 2A.

Three (3) exploratory test pits (P-8 through P-10) were also completed as part of infiltration testing on December 4, 2015. The test pits were dug using a Case CX130 excavator provided by the City.

The test pits extended to depths of ±5.3 to 12 feet. The soil profiles, sampling depths and strength measurements are summarized on the test pit logs (Appendix B).

Boring

We drilled one (1) exploratory borehole (BH-7) at the pump station site on September 25, 2015. The boring was drilled using a CME 75 truck-mounted drill rig with mud-rotary drilling techniques. The borehole was located within the pump station footprint. The location is shown on Figure 3A.

The borehole extended to a maximum depth of ±40.3 feet. Samples were obtained at 2½-foot intervals to ±20 feet and at 5-foot intervals thereafter. Disturbed samples were obtained with a split-spoon sampler during SPT testing. The boring log is included in Appendix B.

At the completion of drilling, BH-7 was flushed with water to remove drilling fluid and a 1-inch diameter, standpipe PVC piezometer was installed to allow monitoring of ground water levels. The standpipe extended to a depth of ±25 feet, and was slotted from ±15 to 25 feet. The installation was capped with a flush-mount Morris monument set in concrete.

Raw Water Pipeline

We drilled six (6) exploratory boreholes (BH-6 and BH-8 through BH-13) along the planned raw water pipeline route on September 21, 22 and 29, 2015. The borings were drilled using a CME 75 truck-mounted drill rig with hollow-stem auger and mud-rotary drilling techniques. BH-6 was drilled at the planned River Drive undercrossing and BH-12 and BH-13 were drilled at the ends of the proposed Santiam-Albany Canal undercrossing. BH-8 through BH-11 were drilled along River Drive between the two undercrossing locations. The borehole locations are shown on Figures 3A through 5A.

The boreholes at the undercrossing locations (BH-6, BH-12, and BH-13) extended to depths of ±35.4 to 36.5 feet. Disturbed SPT samples were obtained at 2½-foot intervals to ±20 feet and at 5-foot intervals thereafter.

At the completion of drilling, BH-13 was flushed with water to remove drilling fluid and a 1-inch diameter, standpipe PVC piezometer was installed to allow monitoring of ground water levels. The standpipe extended to a depth of ±25 feet, and was slotted from ±15 to 25 feet. The installation was capped with a flush-mount Morris monument set in concrete.

The boreholes along the planned raw water line route (BH-8 through BH-11) extended to depths of ±10.9 to 11.5 feet. Disturbed SPT samples were obtained at 2½-foot intervals to the bottom of the borings. The boring logs are included in Appendix B.

Water Transmission Main and Sanitary Sewer Main

We drilled two (2) exploratory boreholes (BH-14 and BH-15) along the planned route of the treated water transmission main and sanitary sewer main. The borings were completed on September 29, 2015, using a CME 75 truck-mounted drill rig with mud-rotary drilling techniques. The borehole locations are shown on Figure 6A.

The boreholes extended to depths of ±10.9 to 11.5 feet. Disturbed SPT samples were obtained at 2½-foot intervals to the bottom of the borings. The boring logs are included in Appendix B.

SURFACE AND SUBSURFACE CONDITIONS

Brief discussions of the surface and subsurface conditions at the project sites are provided below. A detailed summary of the subsurface conditions encountered in each exploration are provided in the appended logs (Appendix B).

Water Treatment Plant

Surface Conditions

The WTP site is located on a parcel that was previously part of a mill site. The site has been regraded and is relatively flat and covered with grass. Topographic information indicates the ground surface elevations range from ±El. 363 to ±El. 365.

Subsurface Conditions

Water Storage Tank. TP-3 and BH-3 were completed within the footprint of the water storage tank. The explorations encountered ±3.5 feet of fill and topsoil consisting of medium stiff, low plasticity silt with trace to some sand and gravel and scattered organics. The fill and topsoil are underlain by alluvium including stiff, medium plasticity clayey silt to ±8 feet, followed by soft to medium stiff, low plasticity sandy silt to ±11 in BH-3 and ±12 feet in TP-3.

Very dense sandy gravel and cobbles extends below the sandy silt to ±20 feet, followed by very stiff, low plasticity silty clay to ±25 feet. Dense sandy gravel with some silt and medium dense silty sand were encountered below the silty clay from ±25 to 36.5 feet (the maximum depth of BH-3).

Process Building. TP-10, TP-11, and TP-24 through TP-28, and BH-4 were completed within the planned process building area. The subsurface conditions vary across the building footprint.

Explorations at the west end of the building (TP-10 and TP-24) encountered ±1.5 to 2 feet of fill and topsoil consisting of predominantly medium stiff to stiff, low plasticity silt with trace to some sand and gravel and scattered cobbles and organics. The fill and topsoil are underlain by alluvium, including stiff to very stiff, medium to high plasticity clayey silt to ±4 to 5 feet, followed by medium stiff sandy silt to ±6.5 to 7.5 feet. A thin lens (±6 to 12 inches thick) of medium dense silty sand was encountered below the sandy silt, followed by dense to very dense sandy gravel and cobbles at depths of ±7.5 to 8 feet. The sandy gravel extends to the bottom of the test pits (±9 feet)

Explorations in the central and eastern portions of the building (TP-11, TP-26 through TP-28, and BH-4) encountered ±1.2 to 2.5 feet of fill and topsoil consisting of predominantly medium stiff to stiff silt with some sand and gravel and scattered organics. The fill and topsoil are underlain by alluvium. In TP-26 through TP-28, the alluvium includes very stiff sandy silt to medium dense silty sand with trace gravel extending to depths of ±5 to 5.2 feet. Very dense sandy and silty gravel and cobbles was encountered beneath the topsoil and fill in TP-11 and beneath the sandy silt in TP-26 through TP-28 at depths of ±1.5 to 5.2 feet. Silty and sandy gravel stratum extends to ±18.5 feet in BH-4, followed by medium dense silty sand from ±18.5 to 24.8 feet. Very dense silty gravel with some sand extends below the silty sand to the maximum depth of BH-4 (±60.9 feet).

Pre-Treatment Facility. TP-5, TP-18, and TP-19 were completed near or within the planned pre-treatment facility area. The subsurface conditions vary slightly in this area.

Exploration at the west end of the facility (TP-18) encountered ±1 foot of topsoil and fill consisting of very stiff gravelly silt with scattered organics. The topsoil and fill are underlain by alluvium including very stiff, medium plasticity clayey silt to ±4 feet, followed by dense sandy gravel with trace silt to ±5 feet (the maximum depth of TP-18).

Exploration at the east end of the facility (TP-5 and TP-19) encountered ±1.5 to 1.7 feet of fill and topsoil consisting of medium stiff to very stiff, low plasticity silt with trace to some gravel and scattered organics. The fill and topsoil are underlain by dense gravel and cobbles with varying sand and silt content to the maximum depths of the explorations (±4.5 to 6 feet).

Future UV Reactor. TP-4, TP-17, and TP-18 were completed near or within the proposed future UV reactor area. The explorations encountered ±1 to 2 feet of fill and topsoil consisting of medium stiff to stiff, low plasticity silt to gravelly silt with scattered organics.

The topsoil and fill are underlain by alluvium including stiff to very stiff, medium plasticity clayey silt to ±4 to 6.5 feet. In TP-4 and TP-17, the clayey silt is underlain by medium stiff sandy silt and/or medium dense silty sand extending to depths of ±8 to 8.5 feet. Dense to very dense sandy gravel and cobbles with trace silt was encountered beneath the clayey silt in TP-18 at ±4 feet and beneath the silty sand in TP-14 and TP-17 at ±8 to 8.5 feet. The sandy gravel extends to the maximum depths of the explorations (±5 to 9.5 feet).

Settling Basins. TP-9, TP-15, and TP-20 through TP-23 were completed within the footprint of the settling basins. The explorations encountered ±1.5 to 3.5 feet of fill and topsoil consisting of medium stiff to very stiff, low plasticity silt with variable sand, gravel and cobble content and scattered organics. The fill and topsoil are underlain by alluvium including stiff to very stiff, medium plasticity clayey silt to ±4 to 7.5 feet.

In TP-21 through TP-23, the clayey silt is underlain by medium stiff, low plasticity sandy silt to ±7.5 to 9 feet, followed by a thin lens (i.e., ±6 to 12 inches thick) of medium dense silty sand. At TP-9 and TP-15, the clayey silt is underlain by medium dense silty sand to ±9 to 12.5 feet. In each of the test pits, the silty sand is underlain by dense to very dense sandy gravel and cobbles with trace silt to the maximum depths of the explorations (±9.5 to 13 feet).

Storm Water Swales. TP-2, TP-8, TP-14, P-6, and P-7 were completed within the vicinity of the storm water swales west of the water tank and settling basins. The explorations encountered ±1 to 1.5 feet of fill and topsoil consisting of soft to stiff, low plasticity silt with scattered organics. The fill and topsoil are underlain by alluvium including stiff, medium plasticity clayey silt to ±3 to 5.3 feet. The clayey silt is underlain by dense sandy gravel and cobbles to the maximum depths of the explorations (±4 to 7.3 feet).

TP-18 and TP-19 were dug near the proposed storm water swale in the northeast portion of the facility. The explorations encountered ±1 to 2 feet of fill and topsoil consisting of very stiff, low plasticity silt with trace to some gravel and scattered organics. The fill and topsoil are underlain by alluvium including very stiff, medium plasticity clayey silt to ±4 feet in TP-18. The clayey silt in TP-19 and topsoil in TP-18 are underlain by dense sandy gravel and cobbles to the maximum depths of the explorations (±4.5 to 5 feet).

TP-28, BH-2, and P-5 were completed in the vicinity of a potential storm water swale in the southeast portion of the facility. The explorations encountered ±1.5 to 2.3 feet of fill and topsoil consisting of stiff, low plasticity silt with variable gravel content and scattered organics. The fill and topsoil are underlain by alluvium including very stiff sandy silt and medium dense silty sand to ±5 feet in TP-28. The sandy silt/silty sand in TP-28 and topsoil in BH-2 and P-5 are underlain by dense sandy gravel and silty gravel and cobbles to the maximum depths of the explorations (±5 to 31.5 feet).

Intake Structure

Surface Conditions

The intake structure site lies on the south bank of the South Santiam River. A ±15 to 20-foot wide, relatively flat bench lies at the top of the river bank, which currently slopes down to the river at ±1.8(H):1(V). Ground elevations at the site range from ±El. 367 (along the edge of River Drive) to ±El. 341.0 (at the toe of the river bank). The site is currently covered by tall deciduous trees, ivy, and brambles.

Subsurface Conditions

Figure 7A shows an estimated surface and subsurface cross-section through the intake and raw water pump station sites. The following sections summarize the subsurface conditions.

BH-5 was drilled southeast of the intake structure. The drilling encountered fill consisting of medium dense silty gravel and cobbles to ±3 feet. The fill is underlain by alluvium including medium dense silty sand with scattered organics to ±6 feet, followed by very dense sandy gravel with trace silt to ±32 feet.

Volcaniclastic sandstone (Fisher Formation) was encountered below the sandy gravel at ±32 feet (±El. 330.2). The sandstone is highly weathered and extremely weak (R0) from ±32 to 37.5 feet. Below that depth, the sandstone becomes moderately weathered and very weak (R1). The sandstone extended to the maximum depth of BH-5 (±56 feet). Very weak (R1), moderately weathered tuffaceous claystone interbeds were noted within the sandstone at ±41.4 and 51.3 feet.

Raw Water Pump Station

Surface Conditions

The raw water pump station site is located on a parcel that was previously used to grow trees. The site is relatively flat and is covered by tree stumps, tall weeds and brambles. Tall fir trees line the perimeter of the property. Topographic information indicates the ground surface elevations range from ±El. 365 at the north end of the property to ±El. 361 at the south end.

Subsurface Conditions

The explorations at the pump station site (TP-29 through TP-32, P-8 through P-10, and BH-7) encountered ±7 to 12 inches of topsoil consisting of predominantly medium stiff silt with some sand and scattered organics.

The topsoil is underlain by alluvium comprised of predominantly loose silty sand, grading to loose sand with some silt below ±2 to 7.5 feet. The sand is underlain by dense to very dense sandy gravel at ±8 to 11 feet. The surface of the gravel stratum appears to be dipping to the south with elevations ranging from ±El. 354.8 to ±El. 351.0. Very dense sandy gravel extended to ±28 feet in BH-7.

Volcaniclastic sandstone (Fisher Formation) was encountered below the sandy gravel at ±28 feet (±El. 335.1) in BH-7. The sandstone is highly weathered and extremely weak to very weak (R0 to R1) from ±28 to 40.3 feet (the maximum depth of BH-7). Comparison of the rock depths in BH-7 and BH-5 and the absence of bedrock within the drilling depth of BH-6 suggests the surface of the bedrock underlying the pump station site dips gently to the north.

Raw Water Pipeline

Surface Conditions

The proposed pipeline route will follow River Drive from the raw water pump station to the WTP. The terrain is relatively flat along the alignment, with gentle grade changes. Available topographic information indicates the paved road surface along the alignment typically lies between ±El. 365 and ±El. 367.

Subsurface Conditions

River Drive Undercrossing. BH-6 was drilled in the center of the southbound travel line on River Drive at the planned trenchless undercrossing location. The drilling encountered a pavement section consisting of ±7 inches of asphaltic concrete (AC) over ±12 inches of dense sandy gravel. The pavement section is underlain by alluvium including loose silty sand to ±10 feet. Very dense sandy gravel with trace silt and scattered cobbles followed to ±36.5 feet (the maximum depth of the boring). Bedrock was not encountered to the bottom of BH-6 (±El. 331.2). However, based on the conditions in the borings at the adjacent intake site (BH-5) and pump station site (BH-7), we anticipate bedrock at a shallow depth below the bottom of BH-6.

Explorations Along River Drive. BH-8 through BH-11 were drilled along the raw water pipeline route between the intake structure and Santiam-Albany Canal. BH-8 and BH-9 were drilled on the east road shoulder and BH-10 and BH-11 were drilled in the center of the northbound travel lane.

Drilling at BH-8 encountered fill consisting of medium dense sandy gravel and cobbles to ±2.5 feet. The fill is underlain by loose silty sand (alluvium) to ±7.5 feet, followed by medium dense grading to very dense sandy gravel with trace silt to ±10.9 feet (the maximum depth of the boring).

Drilling at BH-9 encountered ±5 feet of fill including ±12 inches of medium dense crushed gravel over medium dense sandy gravel. The fill is underlain by loose silty sand (alluvium) to ±10 feet and medium dense silty sand to ±11.5 feet (the maximum depth of the boring).

Drilling at BH-10 encountered a pavement section consisting of ±6 inches of AC over ±24 inches of dense crushed gravel. The pavement section is underlain by alluvium including soft, low plasticity silt to ±7.5 feet, followed by very dense sandy gravel with trace silt to ±10 feet. Medium dense sand with some silt and trace gravel was encountered from ±10 to 11.5 feet (the maximum depth of the boring).

Drilling at BH-11 encountered a pavement section consisting of ±5 inches of AC over ±25 inches of dense crushed gravel. The pavement section is underlain by alluvium including medium stiff grading to soft sandy silt to ±7.5 feet, followed by medium dense sandy gravel with trace silt to ±11.5 feet (the maximum depth of the boring).

Santiam-Albany Canal Undercrossing. BH-12 was drilled on the south shoulder of River Drive at the east end of the planned Canal undercrossing. BH-13 was drilled in the grassy area north of the Cheadle Lake parking lot at the west end of the undercrossing.

Drilling at BH-12 encountered ±7 inches of AC underlain by very dense sandy gravel with trace silt (fill) to ±5 feet. The fill is underlain by alluvium including very stiff, low plasticity silt with trace gravel to ±6 feet, followed by dense to very sandy gravel with trace silt and scattered cobbles to ±20 feet. Medium dense grading to very dense silty gravel with some sand was encountered from ±20 to 36.5 feet (the maximum depth of the boring).

Drilling at BH-13 encountered alluvium including very dense silty gravel with some sand and scattered cobbles to ±6 feet and dense grading to very dense sandy gravel with trace silt to ±25 feet. Dense grading to very dense silty gravel with some sand was encountered from ±25 to 35.4 feet (the maximum depth of the boring).

Water Transmission Main and Sanitary Sewer Main

Surface Conditions

The proposed treated water main and sanitary sewer main will follow River Drive and Russell Drive. The terrain is relatively flat along the alignment, with gentle grade changes. Available topographic information indicates the paved road surface along the alignment typically lies between ±El. 359.5 and ±El. 365.

Subsurface Conditions

BH-14 and BH-15 were drilled along the planned water transmission main and sanitary main route. BH-14 was drilled on River Drive north of the WTP and BH-15 was drilled on Russell Drive near the proposed railroad undercrossing. Both borings were drilled in the center of the northbound travel lane.

Drilling at BH-14 encountered a pavement section consisting of ±6 inches of AC over ±18 inches of crushed gravel. The pavement section is underlain by alluvium including stiff, medium plasticity clayey silt to ±4 feet, followed by very dense sandy gravel with trace silt and scattered cobbles to ±10.9 feet (the maximum depth of the boring).

Drilling at BH-15 encountered a pavement section consisting of ±6 inches of AC over ±18 inches of crushed gravel. The pavement section is underlain by alluvium including stiff, low plasticity silt to ±6 feet, followed by loose silty sand to ±10 feet. Very dense sandy gravel with trace silt extends below the silty sand to ±11.5 feet (the maximum depth of the boring).

GROUND WATER

Water Treatment Plant

Iron-staining was noted in some of the test pits below the fill/topsoil layer at depths of ±1.5 to 4 feet. We anticipate the iron-staining is due to seasonal rainfall that collects within the fill and topsoil and perches on the underlying stiffer, less permeable soil.

During the preliminary investigation in November of 2013, ground water infiltration was observed in six of the test pits completed at the WTP at depths of ±7 to 8.5 feet. The seepage was typically associated with the alluvial sand or gravel strata. At other test pits, excavation was discontinued in the gravels before ground water was encountered. No ground water infiltration was observed in any of the test pits completed during the supplemental investigation in October of 2015. The supplemental work was completed following a period of extended dry weather.

Slow ground water infiltration was observed in P-5 at ±4.1 feet. This excavation was completed on December 4, 2015, following a period of wet weather. The infiltration appeared to be perched water, since the static ground water level in the adjacent piezometer (BH-2) was at ±6.3 feet at the time.

Piezometers were installed in BH-1 and BH-2 to allow monitoring of ground water levels at the WTP site. We recorded initial piezometer measurements of ±6.7 feet (±El. 358.4) in BH-1 and ±6.3 feet (±El. 356.5) in BH-2 on November 7, 2013 (the day after drilling). The initial water level readings in the piezometers were consistent with the ground water seepage observed in the adjacent test pits. These measurements and observations suggested the depth to ground water varied from ±6 to 7 feet below the ground surface at the time of the preliminary investigation.

Subsequent to the preliminary investigation, the City periodically read the piezometers to estimate the seasonal fluctuation in the ground water levels. The City also concurrently measured the water levels in Cheadle Lake (±900 feet to the south of the property) and Santiam-Albany Canal (±100 feet to the east of the property) to discern if those bodies of water influence the ground water level at the site. The initial ground water measurements and subsequent ground water and water level measurements made by the City are shown in Appendix D.

Intake Structure, Raw Water Pump Station and River Drive Undercrossing

No ground water was encountered in the TP-29 through TP-32 dug to depths up to ±10 feet at the pump station site. These test pits were dug on October 1, 2015, during a period of extended dry weather. Moderate ground water infiltration was observed in P-8 at ±10.1 feet (±El. 351.9). This excavation was completed on December 4, 2015, following a period of wet weather.

Mud-rotary drilling methods precluded the measurement of the static ground water levels in BH-6 and BH-7 at the time of drilling. In BH-5, hollow-stem auger drilling techniques were used in the upper ±30 feet, which allowed an estimate of the

ground water level. The water depth in BH-5 was measured at ±15.3 feet (±El. 346.9) during drilling on September 24, 2015.

Piezometers were installed in BH-5 and BH-7 to allow monitoring of ground water levels. The initial piezometer measurement in BH-5 indicated ground water at ±15.7 feet (±El. 346.5) on September 30, 2015. The initial piezometer measurement in BH-7 indicated ground water at ±13.4 feet (±El. 349.7) on October 1, 2015. The initial ground water measurements in BH-5 and BH-7 and subsequent measurements taken by the City are included in Appendix D.

Raw Water Pipeline and Santiam-Albany Canal Undercrossing

Mud-rotary drilling methods precluded the measurement of the static ground water levels in BH-10 through BH-13 at the time of drilling. Hollow-stem auger drilling techniques were used in BH-8 and BH-9, which allowed an estimate of the ground water levels. However, ground water was not encountered in BH-8 or BH-9, indicating water was deeper than ±11.5 feet at the time of drilling.

A piezometer was installed in BH-13 to allow monitoring of ground water levels. The initial piezometer measurement in BH-13 indicated ground water at ±9.3 feet (±El. 352.8) on October 1, 2015. The initial ground water measurement in BH-13 and subsequent measurements taken by the City are included in Appendix D.

Water Transmission Main and Sanitary Sewer Main

Mud-rotary drilling methods precluded the measurement of the static ground water levels in BH-14 and BH-15 at the time of drilling. We anticipate the ground water levels in those borings will be similar to the water levels at the WTP site.

Discussion of Ground Water Conditions

The ground water data collected to date shows low ground water levels at the WTP site of ±12.2 feet (±El. 352.9) in BH-1 recorded in November 2015, and ±9.3 feet (±El. 353.5) in BH-2 recorded in September 2015. High ground water levels at the WTP site were ±2.5 feet (±El. 362.6) in BH-1 recorded in December 2015, and ±4.3 feet (±El. 358.5) in BH-2 recorded in February 2014.

The ground water data and water level measurements show a direct correlation between the water level at Cheadle Lake and the ground water level at the WTP site. The data does not appear to show a significant correlation between the water level in the Canal and the ground water level at the WTP site. It is noted the Canal level measurement shown on the plots in Appendix D is not taken at the proposed undercrossing location. Therefore, the plot does not show a direct correlation between the water level in the Canal and BH-13.

The water level in BH-1, drilled in the western portion of the WTP site, shows greater fluctuations in water levels, with significant dips in the water levels in the dry summer and early fall months. We anticipate these lower ground water levels are the result of drawdown from wells at adjacent residences.

The ground water data collected to date at the intake and pump station sites shows low ground water levels of ±15.8 feet (±El. 346.4) in BH-5 recorded in November 2015, and ±13.4 feet (±El. 349.7) in BH-7 recorded in October 2015. High ground water levels at these sites were ±10.8 feet (±El. 351.4) in BH-5 and ±9.6 feet (±El. 353.5) in BH-7. Both high water levels were recorded in December 2015.

The ground water data collected to date at the canal crossing (BH-13) shows a low ground water level of ±9.3 feet (±El. 352.8) recorded in October 2015, and a high ground water level of ±6.1 feet (±El. 356.0) recorded in December 2015.

The preliminary piezometer measurements in BH-5 and BH-7 suggest the ground water levels at the pump station and intake sites rise with increased rainfall and rising Lake levels. The difference in the water level between BH-5 (intake) and BH-7 (pump station) suggests the water surface is dipping toward the river. The overall trend of the data from the WTP, pump station and intake sites appears to indicate the local ground water in these areas is draining to the east toward the river.

We understand in the winter, the Lake fed by rainfall, runoff and ground water. In the summer, the Lake level is artificially raised above the local ground water level by pumping in water. Therefore, the Lake likely has a greater influence on the local ground water levels in the summer. In the winter, the available ground water and rainfall data (Appendix D) suggests ground water responds relatively quickly to rainfall (rising and falling with rainfall spikes). Therefore, in the winter, rainfall and runoff likely has a greater influence on the ground water levels relative to the Lake.

As indicated by the available data, seasonal fluctuations in ground water levels should be anticipated. Contractors should anticipate the need for dewatering during trenching and deep excavations for the intake, the pump station wet well and undercrossings. Contractors should review the piezometer data prior to preparing bids and prior to construction to evaluate potential ground water impacts and plan their dewatering methods.

LABORATORY AND FIELD TESTING

Laboratory Testing

The laboratory work included natural water contents, Atterberg limits tests, and percent fines content determinations to classify the soils and estimate their engineering properties. Results of these tests are summarized in Table 1C (Appendix C). The natural water content results are also summarized on the boring logs (Appendix B).

Unconfined compression tests were completed on two bedrock samples from BH-5 to estimate the bedrock strength at the intake structure site. The test results, summarized in Table 2C (Appendix C), indicate unconfined compressive strengths ranging from ±219 to 513 psi.

A one-dimensional consolidation test was run on Shelby tube sample SH-3-2 obtained from BH-3 at a depth of ±5 to 7 feet. The test was completed to estimate the compressibility of the shallow, fine-grained soils beneath the water storage tank. Based on the test results, we calculated a recompression index ($C_{r\varepsilon}$) of 0.007 and a preconsolidation pressure (P'_p) of ±3.7 ksf. The consolidation test report is shown in Figure 1C (Appendix C).

pH and Resistivity Testing

We completed in-situ resistivity tests (ASTM G57) at the WTP site on November 1, 2013, and at the pump station site on October 1, 2015. The approximate test locations are shown on Figures 2A and 3A. The testing was completed using a Nilsson 400, 4-pin, soil resistance meter. The 4-pin resistance meter provides an estimate of the average resistivity of a soil profile extending to a depth equal to the spacing between the pins. The tests were performed with the pins spaced at ±2 to 8 feet at the WTP site and at ±6 to 10 feet at the pump station site. The recorded resistivities are summarized in Table 3C (Appendix C).

Laboratory pH tests were run on selected soil samples from the WTP site. The test results shown on Table 4C (Appendix C) indicate pH values between 6.1 and 6.4 (slightly acidic). Laboratory pH tests were not run on soils from the pump station site because the site is underlain by predominantly sandy and gravelly soils that are difficult or impractical to test. These soils should have low corrosion potential.

Infiltration Testing

WTP and Pump Station Sites

We performed infiltration tests at the WTP and pump station sites on September 29 and 30, 2015. The test locations and depths were selected in consultation with the design team. The approximate test locations at the WTP site (P-1 and P-2) are shown on Figure 2A and the approximate test locations at the pump station site (P-3 and P-4) are shown on Figure 3A.

The infiltration tests were performed using the encased falling head infiltration test procedure outlined in the City of Eugene's Stormwater Management Manual (2014). This approach is also consistent with the City of Portland's Stormwater Management Manual (2014).

At each test location, we used a CME 75 truck-mounted drill rig to drill ±8 inch diameter hole to the test depth (between ±2 and 5 feet). Six-inch diameter PVC casing was then installed in each hole and embedded ±6 inches into the soil at the bottom of the holes, where practical. Embedment at P-3 was not possible due to the presence of shallow gravel. The soil at the bottom of the test holes was saturated by filling the casing with at least 12 inches of water and maintaining the depth for at least 4 hours. The infiltration rate was measured at each location the following day, after the soil had been allowed to saturate overnight.

The infiltration rate was measured by filling the casing with water to ±12 inches above the test depth and monitoring the change in the water level for 1 hour. This step was repeated one additional time for a total of two observations per test location.

Additional infiltration testing was completed at the WTP and pump station sites on December 4, 2015. The test locations and depths were selected in consultation with the design team. The approximate test locations at the WTP site (P-5 through 7) are shown on Figure 2A and the approximate test locations at the pump station site (P-8 through P-10) are shown on Figure 3A.

The infiltration tests at P-5 through P-8 were performed using the open-pit falling head infiltration test procedure outlined in the City of Eugene's Stormwater Management Manual (2014).

At the WTP site (P-5 through P-7), ±2-foot wide test pits were dug into the alluvial gravel using a backhoe. Then a ±2 by 2-foot or ±2 by 3-foot was dug ±1 to 1.5 feet deeper into the gravel. The deeper pits were filled with ±12 inches of water and were allowed to saturate for ±10 to 20 minutes. The water in P-6 and P-7 typically drained out over a ±10 minute period. Therefore, the tests were allowed to resume without a 4-hour saturation period. The drop in water level was recorded P-6 and P-7 in 1 minute increments for a 10 minute period. Three to four trials were completed in each of these pits until the calculated infiltration rates on successive trials were within 10 percent of each other.

The infiltration in P-5 was significantly slower and perched ground water was encountered in the pit. For the infiltration test, water was added to raise the water level ±12 inches above the perched water level. The soil was allowed to saturate for ±1 hour and then the infiltration measurements were recorded for ±1 hour.

At the pump station site, a test pit (P-8) was dug to a depth of ±12 feet using an excavator. Due to caving sidewalls, the low point of the pit was ±4 feet wide and ±6 feet long. Loose silty sand and sand was encountered to ±11 feet. The upper ±4 feet of the sand was typically moist. The underlying sand was damp. Dense sandy gravel and cobbles were encountered below ±11 feet. Moderate ground water infiltration was observed at the top of the gravel stratum and the pit quickly filled with water to ±10.1 feet.

The test pit sidewalls were unstable due to the loose sand. Therefore, there was no time for a saturation period. We filled the pit with water to ±12 inches above the ground water level and recorded infiltration rates for 10 minutes before the sidewalls caved. The water apparently infiltrated into the sandy gravel and cobbles and the loose sand above the gravel stratum.

P-9 and P-10 were completed in the loose silty sand and sand at depths of ±4.8 and 6.3 feet. The tests were completed by excavating a test pit to ±6 inches above the test depth and then pushing a 6-inch diameter PVC casing into the soil, consistent with the encasing falling head procedure described above. The soil at the bottom of the

test holes was saturated by filling the casing with at least 12 inches of water and maintaining the depth for at least 1 hour. The infiltration rate was periodically measured for ±1.8 hours.

Table 1 provides a summary of the test depths, soil conditions, and infiltration rates at each test location. We were not able to calculate an infiltration rate at P-3 using the encased method because we could not embed the PVC casing into the gravel and obtain an adequate seal per the test specifications.

Table 1. Falling Head Infiltration Test Results – WTP and Pump Station

Site	Test Location	Test Depth (feet)	Soil Description at Test Depth	Infiltration Rate (inch/hour)
Pump Station	P-1	±4.9	Loose, brown silty SAND	±0.6
	P-2	±1.9	Loose, brown silty SAND	±0.8
	P-8	±10 - 11	Loose brown silty SAND and dense silty, sandy GRAVEL	±5.8
	P-9	±6.3	Loose, brown silty SAND	±0.3
	P-10	±4.8	Loose, brown silty SAND to SAND	±1.3
WTP	P-4	±1.9	M. Stiff, brown, low to medium plasticity SILT	±0.4
	P-5	±3.5 - 4.5	Dense, brown, silty sandy GRAVEL	±1.6
	P-6	±6.0 – 7.0	Dense, brown, sandy GRAVEL and COBBLES	±21.6
	P-7	±5.5 – 6.5	Dense, brown, silty sandy GRAVEL and COBBLES	±15.1

Notes:
1. Tests at P-1, P-2, P-4, P-9 and P-10 followed the Encased Falling Head Test procedure as specified in Appendix G of the City of Eugene Stormwater Management Manual (2014). The depths correspond to the top of the soil plug inside the casing.
2. Tests at P-5, P-6, P-7, and P-8 followed the Open Pit Falling Head Test procedure as specified in Appendix G of the City of Eugene Stormwater Management Manual (2014). The depths correspond to the range of soil subjected to vertical and lateral infiltration.
3. Tests at P-1 through P-4 were conducted between September 29 and 30, 2015.
4. Tests at P-5 through P-10 were conducted on December 4, 2015.
5. Infiltration rate at P-3 was not reported because an adequate seal was not obtained between the soil and the bottom of the ±6-inch diameter casing. Infiltration rate at P-3 was estimated at ±1.9 in/hr. based on correlations with the infiltration and hydraulic conductivity data.
6. See Figures 2A and 3A for test locations.
7. All test results reported to the first decimal place and are unfactored.

The infiltration test measurements were also used to calculate the hydraulic conductivity (coefficient of permeability, k) of the soil at each test location. For P-1, P-2, P-4, P-9 and P-10, the k values were calculated using an equation for a cased falling-head permeability test. For P-3, we removed the casing and calculated the k value using an equation for an uncased falling-head permeability test. The calculated k values are summarized in Table 2. The k values fall within the typical range of values for these types of soils.

Table 2. Field Hydraulic Conductivity Test Results – WTP and Pump Station

Site	Test Location	Test Depth (feet)	Soil Description at Test Depth	Average Hydraulic Conductivity, k (cm/sec)
Pump Station	P-1	±4.9	Loose, brown silty SAND	$\pm 5.3 \times 10^{-5}$
	P-2	±1.9	Loose, brown silty SAND	$\pm 7.6 \times 10^{-5}$
	P-9	±6.3	Loose, brown silty SAND	$\pm 2.0 \times 10^{-5}$
	P-10	±4.8	Loose, brown silty SAND to SAND	$\pm 1.2 \times 10^{-4}$
WTP	P-3	±5	Dense silty GRAVEL, some sand, scattered cobbles	$\pm 1.8 \times 10^{-4}$
	P-4	±1.9	M. stiff, brown, low to medium plasticity SILT	$\pm 3.4 \times 10^{-5}$

Notes:
1. Results from the Encased Falling Head Tests completed at P-1 through P-4, P-9 and P-10 were used to calculate the hydraulic conductivity at each location. The depths correspond to the top of the soil plug inside the casing.
2. Tests at P-1 through P-4 were conducted between September 29 and 30, 2015.
3. Tests at P-9 and P-10 were conducted on December 4, 2015.
4. See Figures 2A and 3A for test locations.
5. All test results reported to the first decimal place and are unfactored.

Intake Site

Falling head permeability testing was completed during drilling at BH-5. The tests were performed through the hollow-stem augers at depths of ±25 and 30 feet to estimate the hydraulic conductivity of the alluvial gravel. k values were calculated using an equation for a cased falling-head permeability test. Because the tests were run inside augers with multiple joints, it is likely there was some fluid loss through the joints. Therefore, the k values, summarized in Table 3, should be considered approximate. The estimated k values are within the typical range of values for sandy gravel with silt. We estimate the corresponding infiltration rates would be ±2.4 to 3 inches/hour.

Table 3. Field Hydraulic Conductivity Test Results – Intake Site

Test Location	Test Depth (feet)	Soil Description at Test Depth	Average Hydraulic Conductivity, k (cm/sec)
BH-5	±25	Sandy GRAVEL, trace silt, scattered cobbles	$\pm 2.3 \times 10^{-4}$
	±30	Sandy GRAVEL, trace silt, scattered cobbles	$\pm 3.0 \times 10^{-4}$

Notes: 1. Tests were conducted on September 24, 2015.
2. See Figure 3A for test location.
3. All test results reported to the first decimal place and are unfactored.

Infiltration Testing Summary and Conclusions

The test results at the pump station site (P-1, P-2, and P-9) indicate infiltration rates ranging from ±0.3 to 0.8 inch/hour within the silty sand in the upper ±5 to 6 feet of the site. These rates are slightly higher than what is considered a low infiltration rate (i.e., less than 0.5 inches/hour). P-10 indicated an infiltration rate of ±1.3 inches/hour in soil that contained lenses of silty sand and sand. We believe the relatively low rates in the sandy soils at the pump station site are due to the high silt content (±33% to 34%) and the relatively fine grain size of the sand.

Our observations indicate there is not a well-defined transition between the silty sand and cleaner sand, rather these soils become intermixed below ±4 feet. Therefore, the infiltration rates in the sandy soils at the pump station site are likely to range from ±0.3 to 1.3 inches/hour with the slowest infiltration rates in the upper ±4 feet of the profile. As previously noted, the test pit for P-8 showed moist sand to ±4 feet, followed by damp sand to ±11 feet (the top of the gravel stratum). The absence of moisture below ±4 feet in the sand, following a significant period of rainfall provides additional evidence the sandy soils have a relatively slow infiltration rate.

Testing in the silt (fill/topsoil) at the WTP site (P-4) indicated relatively low permeability with an infiltration rate of ±0.4 inch/hour. Testing in the shallow silty gravel at P-5 indicated in infiltration rate of ±1.6 inches/hour. We were unable to measure the infiltration rate in the silty gravel in P-4 directly in accordance with the encased falling head test, but we estimated an infiltration rate of ±1.9 inches/hour for that soil by correlating the hydraulic conductivity and infiltration rate data from the WTP and pump station sites. The test data showed a relatively tight linear correlation between hydraulic conductivity and infiltration rate. Testing in the deeper sandy gravel and cobbles at the WTP site indicated infiltration rates of ±15.1 to 21.6 inches/hour. This soil is relatively free draining.

In general, the test results indicate the upper silt and silty sand at the WTP and pump station sites have relatively poor drainage characteristics and the underlying sandy gravel and cobbles have good drainage properties. Based on the test results, it is anticipated it will be necessary to trench down to the gravel stratum to reach soil with adequate infiltration rates. The effectiveness at draining surface water into the gravel will depend in part on local ground water levels.

The infiltration test results provided in this report are unfactored. Designers using this information should provide an appropriate amount of conservatism in their design.

SEISMIC DESIGN

A detailed Site Specific Seismic Hazard Study was completed for the project. The report presented in Appendix E indicates no major seismic hazards that would preclude the proposed development. However, a limited liquefaction hazard may exist in some areas depending on ground water conditions. The following sections provide a discussion of the liquefaction hazard and provide acceleration site response spectra for the design of the structures in accordance with OSSC 2014. The evaluation of other geologic hazards is presented in Appendix E.

Liquefaction and Lateral Spread

Water Treatment Plant

Liquefiable soils typically consist of loose, fine-grained sand and non-plastic or low plasticity silt (i.e., silt with a plasticity index (PI) less than 8) below the ground water table. The WTP site is typically capped with medium stiff to stiff silt and clayey silt, which are not considered liquefiable due to their stiffness and plasticity. Additionally, the site is underlain by predominantly medium dense to very dense sandy gravel and cobbles that are not considered liquefiable due to their density and coarse gradation.

Relatively thin (±1 to 4.5-foot thick) lenses of medium stiff sandy silt and loose to medium dense silty sand were observed between the upper fine-grained soil and the gravel stratum in test pits at scattered locations across the WTP site, including the footprints of the water storage tank, the settling basins, future UV reactors and the west side of the process building depths. Medium dense silty sand lenses were also noted from ±32 to 36 feet in BH-3 and from ±18.5 to 24.8 feet in BH-4.

The loose to medium dense silty sand is potentially susceptible to liquefaction, provided it is saturated at the time of a large earthquake. However, based on the limited thickness of the potentially liquefiable soil, we believe the surface settlement, if any, associated with liquefaction would be modest (±1 inch or less). The site is relatively flat and there is no indication of a continuous liquefiable sand layer. Therefore, the risk of lateral spread is considered very low.

Intake Structure

The intake site is capped with medium dense silty sand underlain by very dense sandy gravel and bedrock. The foundation excavation for the structure will remove the silty sand and the structure will bear on very dense sandy gravel. Therefore, the risk of liquefaction impacting the intake structure is negligible. The structure is located on a slope. However, based on the absence of liquefiable soils, the risk of lateral spread is also considered negligible.

Raw Water Pump Station

The pump station site is capped by ±8 to 11 feet of loose silty sand and sand underlain by very dense sandy gravel and bedrock. Construction of the wet well and associated underground piping will require a large excavation extending below the sand layer beneath the center of the pump station building area. To provide uniform support for the building's shallow foundations, we recommend excavating any loose sand that extends below the building footprint and placing it back in compacted lifts or replacing it with imported granular fill. The trenches for the raw water transmission lines entering and exiting the site will also bypass the loose sand. Therefore, the liquefaction risk for the pump station building and associated piping is negligible.

Loose sand will remain beneath the pavements and drainage swales outside the building area. Based on the available ground water data, we anticipate the loose sand stratum is above the ground water level in the summer. However, comparing the available ground water data at the pump station site to the seasonal data from the WTP site and the estimated 100-year flood level on the river, we anticipate ground water could rise as high as ±El. 361 at the pump station site during the 100-year flood. This ground water elevation would extend ±6 to 8 feet above the bottom of the loose sand stratum at the site. Therefore, it is possible liquefaction could occur in the lower portion of the loose sand stratum, if a large earthquake struck in the winter or spring during a period of extended heavy rainfall and/or flooding. Based on the thickness of the saturated soil, the associated liquefaction-induced settlement could be ±1 to 3 inches, depending on the water ground level and level of ground shaking. The site is relatively flat. Therefore, the risk of lateral spread is considered negligible.

Because the liquefaction risk is limited to the pavements and drainage swales, we assume mitigation will not be necessary and these areas will be repaired, if needed, following an earthquake.

Water and Sewer Transmission Lines

Loose sand was also encountered in the borings along the alignments for the raw water transmission line and the finished water and sewer mains. The sand is likely above the ground water level through much of the year. Therefore, we anticipate an earthquake would have to occur coincident with a period of extended heavy rainfall and/or flooding, when the ground water level is elevated, to develop saturated conditions in the potentially liquefiable sandy soil. Even then, it is likely saturated soil conditions will be limited to the lower portion of the sand stratum.

The trenching for the sewer main is expected to extend through the loose sand stratum. The liquefaction risk for this component is considered negligible. Depending on location, the trenching for the shallower finished water main and raw water transmission line is expected to remove most or all of the loose sand. Where loose sand remains below the bottom of the trench, the compaction of the pipe bedding material and trench backfill is likely to densify a portion of the sand, leaving

a relatively thin layer of loose sand. Therefore, we anticipate the liquefaction risk is relatively low and the liquefaction-induced settlement, if any, is expected to be less than ±1 inch. The transmission line alignments are relatively flat. Therefore, the risk of lateral spread is considered negligible.

Bedrock Acceleration and Site Response

Site response spectra for the project were prepared based on the general procedure in OSSC 2014. The design maximum considered earthquake ground motion maps provided in the 2014 OSSC are modified 2008 USGS maps with a 1% probability of exceedance in 50 years (i.e., a ±4,975-year return period). The modifications include factors to adjust the spectral accelerations to account for directivity and risk.

An OSSC 2014 Site Class D (stiff soil) is recommended for the WTP and pump station sites, based on the current conditions and proposed site grading. We anticipate the intake structure will be buried in very dense sandy gravel and will bear on a relatively thin layer of very dense sandy gravel underlain by sandstone. Therefore, an OSSC 2014 Site Class C (very dense soil and soft rock) is appropriate for the design of the intake structure.

The OSSC 2014 site response spectra and recommended seismic design parameters for the WTP, pump station and intake sites are summarized on Figures 8A and 9A (Appendix A).

DISCUSSION OF GEOTECHNCIAL CONSIDERATIONS

Construction Timing

The surficial soils at the WTP, intake and pump station site are predominantly fine-grained. These soils are moisture-sensitive and will soften considerably when wet and disturbed. Therefore, we recommend completing the earthwork in the dry summer months (typically late June through the end of September or early October). Construction during this period should minimize dewatering efforts and allow reuse of suitable native fill materials. Native materials would not be suitable for reuse as fill during wet weather.

Water Treatment Plant

A primary geotechnical consideration for the development of the WTP site is the mitigation of existing site fill and the placement of new fill for site grading. Once the fill is mitigated as described herein, the site will be suitable for supporting the proposed structures on shallow foundations.

Fill and topsoil were encountered at the ground surface in nearly all of the explorations at the WTP site, with the exception of TP-1. The fill consists of predominantly silt with variable amounts of sand and gravel. Scattered organics including roots and wood debris are also present in the fill.

The fill appears to have been spread over the site and placed without formal compactive effort. We anticipate this material will be compressible. Therefore, to reduce the risk of settlement, we recommend moisture-conditioning and compacting the fill and topsoil in the planned development areas prior to placing fill for site grading.

The thickness of the fill and topsoil varies with location. The fill and topsoil are thinnest (typically ±2 feet thick or less) in the eastern one-third of the property (within the planned processing building, pre-treatment and UV reactor areas) and in the western one-third of the property (on the west edge of the current planned development and within the undeveloped area to the west). The deepest fill and topsoil (±3 to 4 feet thick) are located in the middle one-third of the property (in the vicinity of the proposed water storage tank, and settling basins). Table 1B (Appendix B) summarizes the fill and topsoil depths at the WTP site.

Where the fill and topsoil is thin (±1.5 feet or less), processing will consist of stripping the ground surface ±4 inches to remove sod and roots, and then ripping, moisture-conditioning and compacting the underlying ±1 foot of soil. Where the fill is thicker, we recommend removing the upper ±1 to 2 feet, then ripping, moisture-conditioning and compacting the underlying ±1 foot of soil prior to placing additional fill. Suitable soils from the excavations may be stockpiled and reused on site (during dry weather).

The current grades on the WTP site range from ±El. 363 at the north to ±El. 365 at the south. The planned FFE for the structures is El. 366.5. Therefore, the site grades will be raised by ±1.5 to 3.5 feet. The site grading may be completed using a variety of materials. Materials from on-site excavations and from the excavation at the intake and pump station sites may be reused as fill during dry weather, provided they are free of organics, high plasticity clay or other deleterious materials. Imported fill will also be required, including granular fill for pavements and building pads.

Ground water levels will also be a key issue for below-grade structures, including the settling basins and the associated sump pit. Piezometer measurements to date indicate high ground water levels ranging from ±2.5 feet (±El. 362.6) in the western portion of the site (BH-1) to ±4.3 feet (±El. 358.5) in the eastern portion of the site (BH-2). We believe El. 363 is a reasonable average high ground water level for design.

Intake Structure

The primary geotechnical consideration for the construction of the intake structure will be the need for temporary shoring and dewatering for the deep foundation excavation. For the design of the permanent structure, the considerations will include resisting lateral earth pressure and hydrostatic uplift pressures.

The current ground surface in the proposed intake footprint ranges from ±El. 341 to ±El. 362 (Figure 7A). Preliminary plans indicate the base of the lower intake structure

will be supported by a ±5-foot thick concrete slab with a plan area of ±37.5 by 38.5 feet. The base slab will include a ±3.5-foot thick, unreinforced slab, poured on the subgrade to help seal the excavation and a 1.5-foot thick, reinforced structural slab poured on top of the unreinforced slab.

The base of the unreinforced slab will be at ±El. 333.5. Based on the available subsurface data, we anticipate the slab will bear on a relatively thin stratum of dense to very dense sandy gravel and cobbles underlain by sandstone. Therefore, bearing capacity and settlement are not expected to be design considerations.

The excavation for the intake will be in predominantly dense sandy gravel and cobbles. The excavation for the lower intake will extend ±8 to 9 feet below the river bed and up to ±30 feet below the bench on the south side of the structure. The plans indicate the river level is anticipated at ±El. 343 in the summer. We anticipate the ground water level in the excavation will be similar, and possibly up to 1 foot higher due to ground water draining from the south.

An open excavation will not be practical due to the elevated ground water level and the need to extend into the river. Therefore, it should be assumed full-depth shoring and dewatering will be required. We anticipate the most practical shoring option will be an internally-braced sheet pile cofferdam. For that option, we recommend the pile tips be driven to, and seated into, the bedrock (if practical) to help slow ground water infiltration. Based on the hardness of the sandstone, we anticipate penetration of the sheet pile tips into bedrock will be limited to ±6 inches or less.

Difficult driving should be anticipated and the sheet pile penetration may be variable. Some of the sheet piles may encounter refusal before the tips reach bedrock due to the density of the gravel and the presence of cobbles. We recommend reinforcing the pile tips to help facilitate penetration and reduce the risk of tip damage. It may be necessary to use an excavator to loosen the gravelly soil to help facilitate pile driving.

In the event it is impractical to drive the sheet piles to bedrock, a secondary method of ground water control may be required. Where sheet piles meet refusal above the bedrock surface, one option may be to grout the gravels outside the sheets. Another option may be to provide a seal of CLSM or concrete inside the cofferdam.

Bedrock is expected to be relatively shallow below the base of the lower intake excavation. Therefore, it should be assumed the sheet pile tips will have little or no fixity and full-depth internal bracing will be required.

The 100-year flood elevation is ±El. 360.5. Ground water behind the intake structure walls is likely to be at or ±1 to 2 feet above this level during this event due to local ground water draining toward the river. Therefore, the design of the permanent structure will need to resist a relatively large hydrostatic uplift pressure.

Raw Water Pump Station

The primary geotechnical consideration for the construction of the pump station will be the need for temporary shoring and dewatering for the excavations for the wet well, and associated pipes and manholes. For the design of the permanent wet well structure, the considerations will include resisting lateral earth pressure and hydrostatic uplift pressures. For the design of the shallow foundations supporting the pump station building, mitigation of loose sand will be a key issue.

The current ground surface at the proposed pipe and pump station building areas ranges from ±El. 361 to ±El. 366 (Figure 3A and Figure 7A). Based on our borings and test pits, we anticipate the subsurface profile at the pump station site will include ±8 to 11 feet of loose silty sand and sand underlain by dense to very dense sandy gravel below (±El. 352 to ±El. 357). Extremely weak to very weak (R0 to R1) volcaniclastic sandstone is anticipated below ±El. 331 to ±El. 336.5.

Preliminary plans indicate the excavations for the manholes will extend to ±El. 329.3 to ±El 332 (±33 to 36 feet below current grades). We understand the base of the wet well excavation will extend to ±El. 324 (±38 to 41 feet below current grades). Similar to the intake structure, we assume the wet well will be constructed with a relatively thick, unreinforced slab poured on the subgrade to help seal the excavation and a thinner, reinforced structural slab poured on top of the unreinforced slab.

Based on the available information, we anticipate the base of the wet well and manholes will likely be supported on bedrock or a thin stratum of very dense sandy gravel and cobbles over bedrock. Therefore, bearing capacity and settlement are not expected to be design considerations.

We anticipate ground water between ±El. 349 and ±El. 351 in the summer. We anticipate ground water may be up to ±El. 361 during the 100-year flood, due to the elevated river level and ground water draining toward the river.

The excavations for the wet well and associated manholes and pipes will extend ±17 to 27 feet below the anticipated summer ground water level. Therefore, an open excavation will not be practical and it should be assumed full-depth shoring and dewatering will be required.

We anticipate the most practical shoring option will be an internally-braced sheet pile cofferdam. For that option, we recommend the pile tips be driven to, and seated into, the bedrock as far as practical to help seal off the ground water infiltration. Based on the hardness of the sandstone, we anticipate penetration of the sheet pile tips into bedrock may be limited to ±6 inches or less.

Difficult driving should be anticipated and the sheet pile penetration may be variable. Some of the sheet piles may encounter refusal before reaching bedrock due to the density of the gravel and the presence of cobbles. Cobbles up to ±9 inches in diameter were observed in our test pits at the site.

We recommend reinforcing the pile tips to help facilitate penetration and reduce the risk of tip damage. It may be necessary to use an excavator to loosen the gravelly soil to help facilitate pile driving. In the event it is impractical to drive the sheet piles to bedrock, a secondary method of ground water control may be required. Where sheet piles meet refusal above the bedrock surface, one option may be to grout the gravels outside the sheets. Another option may be to provide a seal of CLSM or concrete inside the cofferdam.

The excavation into bedrock will extend below the bottom of the sheets. Therefore, it should be assumed the sheet pile tips will have no fixity and full-depth internal bracing will be required.

The footprint of the pump station building will extend beyond the limits of the wet well excavation. The building will have a slab-on-grade floor and will be supported on a continuous perimeter footing. This area is currently underlain by ±8 to 10 feet of loose silty sand and sand, followed by very dense sandy gravel and cobbles. The loose sand will be compressible relative to the sandstone that supports the wet well. To provide uniform support for the building's shallow foundations, we recommend excavating any loose sand that extends below the building footprint and replacing it with imported granular fill. In consultation with the design team, it was concluded Stabilization Rock (i.e., 2½-inch minus crushed rock) will be used for the majority of the backfill. The larger Stabilization Rock will be capped with Base Rock within ±3 feet of the finish grade. The excavation and backfilling should extend at least ±5 feet outside the building footprint.

The excavated soils may be reused for site grading outside the building area. The sandy soils will be moisture-sensitive. Therefore, it should be assumed moisture-condition and careful moisture control will be required to facilitate compaction and reduce the risk of pumping.

Ground water levels will also be a key issue for below-grade structures, including the settling basins and the associated sump pit. Piezometer measurements to date indicate a high ground water level of ±9.6 feet (±El. 353.5) in BH-7. The corresponding high ground water level in BH-5 to the north was ±10.8 feet (±El. 351.4). Based on the available data, we believe El. 361.0 is a reasonable high ground water level for pump station design. This elevation is the estimated pump station site ground water level corresponding to the 100-year flood level (±El. 360.5) at the adjacent river.

Raw Water Pipeline

The primary geotechnical consideration for the construction of the raw water pipeline will be the need for temporary shoring and dewatering for the deeper excavations, particularly at the Canal undercrossing.

Trenching for the proposed pipeline is expected to extend ±7 to 8 feet below the road surface. We anticipate the trenching will encounter granular fill followed by soft to medium stiff silt and sandy silt or loose silty sand. Caving of trench sidewalls should be expected throughout the alignment, particularly where ground water or perched water is encountered. Therefore, the contractor should plan to provide shoring and dewatering as needed to protect workers from sloughing or caving sidewalls along the entire alignment. The silty and sandy soils and underlying gravel correspond to an OR OSHA Soil Type C. OR OSHA recommends a maximum allowable cut slope of 1½(H):1(V) for open excavations in this material, assuming no ground water is present.

We anticipate more extensive shoring and dewatering will be required for the trenchless Canal undercrossing. The excavations for the jacking and receiving pits are expected to encounter predominantly dense to very dense silty and sandy gravel. Ground water levels are likely to be similar to the Canal level.

We anticipate sheet piles will be used for shoring the jacking and receiving pits. We recommend the pile tips be driven as deep into the gravel as practical to help slow ground water infiltration. Difficult driving should be anticipated and the sheet pile penetration may be variable due to the density of the gravel and the presence of scattered cobbles. We recommend reinforcing the pile tips to help facilitate penetration and reduce the risk of tip damage.

Water Transmission Main and Sanitary Sewer Main

Similar to the raw water transmission line, the primary geotechnical consideration for the construction of the treated water main and sewer main will be the need for temporary shoring and dewatering for the deeper excavations.

Trenching for the water main is expected to extend to depths of ±5 to 7 feet and the trenching for the sewer main is expected to extend to depths of ±12 to 13 feet. We anticipate the trenching will encounter granular fill followed by medium stiff to stiff silt and clayey silt and/or loose silty sand. Dense to very dense sandy gravel and cobbles are anticipated at or near the bottom of the trenches.

Caving of trench sidewalls should be expected throughout the alignment, particularly where ground water or perched water is encountered. Therefore, the contractor should plan to provide shoring and dewatering as needed to protect workers from sloughing or caving sidewalls along the entire alignment. The silt, sandy soils and gravel correspond to an OR OSHA Soil Type C. The stiff clayey silt corresponds to an OR OSHA Soil Type B. OR OSHA recommends maximum allowable cut slopes in open excavations of 1½(H):1(V) for Type C soils and 1(H):1(V) for Type B soils, assuming no ground water is present.

Based on the available information, we anticipate ground water below ±El. 356 in the dry summer months. Therefore, ground water should be anticipated in the lower portion of the trenches.

ENGINEERING ANALYSIS

Water Treatment Plant

Water Storage Tank

Bearing Capacity and Settlement. The tank will have a FFE of ±El. 366.5 and a maximum water height of ±17.5 feet. The existing ground surface lies at ±El. 363.5. Therefore, the floor slab of the tank will extend ±3 feet above the current grade. The tank site is underlain by ±3.5 feet of existing fill and topsoil. We assume this material will be moisture-conditioned and recompacted prior to raising the site grade or this material will be excavated and replaced with compacted imported fill. We assume the fill used to raise the site will be stiff or dense.

We have assumed the water storage tank will be supported on a slab and a perimeter ring footing. We recommend providing a minimum of 12 inches of compacted Base Rock beneath the slab and footings.

Based on the assumed foundation conditions, we recommend designing the perimeter footing using an allowable bearing pressure of 2,500 psf. This value includes a typical factor of safety of 3. The allowable bearing pressure may be increased by one-third for analysis using temporary live (wind and earthquake) loads. The footing should be sized to include the structure loads and the weight of the water above the footing.

Settlement analysis was completed using Settle3D software in combination with the field and laboratory data. Footing dimensions and structure loads were not available at the time this report was completed. For the analysis, we calculated a contact pressure of 1,500 psf at the current ground surface (El. 363.5) assuming the maximum water height (17.5 feet), a 6-inch thick floor slab, and ±2 feet of dense fill. A 12-inch wide perimeter ring footing was assumed with a bearing pressure of 2,500 psf. The subsurface soils were assigned compressibility properties based on SPT results and on our experience with similar soils. The calculations indicate a maximum settlement of ±1.5 inches at the center of the tank and ±¾ inch at the perimeter. We anticipate ±¼ inch of the settlement will occur during construction as the site is graded and the tank is constructed. The remaining settlement will be due to the weight of the water and is expected to occur after the initial filling of the tank. Some long-term, post-construction settlement (i.e., creep) should also be expected.

Sliding Coefficient and Passive Resistance. We have assumed the storage tank will be constructed on a pad of compacted Base Rock. For sliding analysis, we recommend using a coefficient of sliding friction of 0.5 between the base of the concrete footings and Base Rock.

Passive resistance of the soil in front of the footings was calculated as an equivalent fluid density equal to $\gamma * k_p$, where γ is the unit weight of the soil and k_p is the passive earth pressure coefficient. We assumed an angle of internal friction (ø) of 30 degrees and a moist unit weight of 115 pcf for design, assuming the footings

would be backfilled with fill, possibly including fine-grained soil. The calculations indicate an equivalent fluid density of ±345 pcf as an ultimate passive resistance, assuming drained conditions.

The base sliding resistance will develop with very small translational movement. However, development of the full passive resistance in front of the buried footings would likely require excessive translation of the footings. Assuming small movements (on the order of a fraction of an inch), we recommend using one-third of the ultimate passive resistance in combination with the base friction. Therefore, a coefficient of friction of 0.5 and a nominal passive resistance of ±115 pcf are recommended for design.

Modulus of Subgrade Reaction. We recommend using a modulus of subgrade reaction (k_s) of 200 kcf for slab design. This value assumes the slab will be built on a minimum of 12 inches of compacted Base Rock underlain by compacted, stiff or dense fill.

Process Building

Bearing Capacity and Settlement. The process building will have a FFE of ±El. 366.5. The existing ground elevations in the building range from ±El. 363 to ±El. 364. Therefore, the floor slab will extend ±2.5 to 3.5 feet above the current grades. The building site is underlain by ±1 to 2 feet of existing fill and topsoil. We assume this material will be moisture-conditioned and recompacted prior to raising the site grade or this material will be excavated and replaced with compacted imported fill. We assume the fill used to raise the site will be stiff or dense.

The building will have a slab-on-grade floor and will be supported on a continuous perimeter footing and isolated column footings. We understand the maximum column load will be 50 kips or less. We recommend providing a minimum of 12 inches of compacted Base Rock beneath the slab and 6 inches of compacted Base Rock beneath the footings.

Based on the assumed foundation conditions, we recommend designing the footings using an allowable bearing pressure of 2,500 psf. This value includes a factor of safety of 3. The allowable bearing pressure may be increased by one-third for analysis using temporary live (wind and earthquake) loads.

A formal settlement analysis was not performed. However, based on the assumed soil conditions and loads, we anticipate total and differential settlements less than ± ¾ inch and ± ½ inch, respectively.

The bearing pressure and settlement estimates provided above are also appropriate for the design of the below grade components (i.e., the containment areas and pipe gallery).

Sliding Coefficient and Passive Resistance. We have assumed the new footings will be constructed on compacted Base Rock. We assume the footings will be backfilled with fill, possibly including fine-grained soil. We recommend evaluating the sliding resistance of the footings using a coefficient of friction of 0.5 and a nominal passive resistance calculated as an equivalent fluid density of ±115 pcf.

The sliding coefficient and passive resistances provided above are also appropriate for foundation design for the below-grade components (i.e., the containment areas and pipe gallery). This assumes the foundations are constructed on compacted Base Rock.

Lateral Earth Pressure for Below-Grade Walls. We recommend backfilling the below-grade structures using compacted Base Rock. Lateral earth pressures on these walls should be designed using for at-rest pressures calculated using an at-rest earth pressure coefficient (k_o) of 0.44 and a moist unit weight of 125 pcf, above the design ground water level (El. 363.0). This corresponds to an equivalent fluid density of 55 pcf. Below the design ground water level, the at-rest pressures should be designed using a buoyant equivalent fluid density of 27.5 pcf (to represent the effective soil pressure) plus a hydrostatic pressure of 62.4 pcf.

Modulus of Subgrade Reaction. We recommend using a modulus of subgrade reaction (k_s) of 200 kcf for slab design. This value assumes the slab will be built on a minimum of 12 inches of compacted Base Rock underlain by compacted, stiff or dense soil.

Pre-Treatment Facility and Future UV Reactor

Bearing Capacity and Settlement. The pre-treatment and UV reactor equipment will be supported on concrete mat foundations with a FFE of ±El. 366.5. The existing ground surface in these area lies at ±El. 363 to ±El. 363.5. Therefore, the mat slabs will extend ±3 to 3.5 feet above the current grades. This area is underlain by ±1 to 2 feet of existing fill and topsoil. We assume this material will be moisture-conditioned and recompacted prior to raising the site grade or this material will be excavated and replaced with compacted imported fill. We assume the fill used to raise the site will be stiff or dense.

We recommend providing a minimum of 12 inches of compacted Base Rock beneath the mat foundations. Based on the assumed foundation conditions, we recommend designing the mat foundations using an allowable bearing pressure of 2,500 psf. This value includes a factor of safety of 3. The allowable bearing pressure may be increased by one-third for analysis using temporary live (wind and earthquake) loads.

A formal settlement analysis was not performed. However, based on the soil conditions and assumed loading, we anticipate total and differential settlements less than ± ¾ inch and ± ½ inch, respectively.

Sliding Coefficient and Passive Resistance. We have assumed the mat foundations will be constructed on compacted Base Rock. We assume the footings will be backfilled with fill, possibly including fine-grained soil. We recommend evaluating

the sliding resistance of the footings using a coefficient of friction of 0.5 and a nominal passive resistance calculated as an equivalent fluid density of ±115 pcf.

Modulus of Subgrade Reaction. We recommend using a modulus of subgrade reaction (k_s) of 200 kcf for slab design. This value assumes the mat foundations will be built on a minimum of 12 inches of compacted Base Rock underlain compacted, stiff or dense fill.

Settling Basins

Basin Construction. Two settling basins are planned. The basins will be ±35 feet wide by ±140 feet long at the base with 2(H):1(V) side slopes on three sides and a ramp graded at 10(H):1(V) to provide access for cleaning. Preliminary plans indicate the FFE at the base of the basins will range from El. 358.5 (at the bottom of the access ramp) to El. 358.0 (at the low point), but the actual base elevations may be adjusted based on ground water considerations. The top of the basins will be at El. 366.0 and the adjacent ground will be at El. 366.7.

We understand the basins will be lined with ±6 inches of fiber-reinforced concrete, underlain by ±6 inches of asphalt-treated base course (ATB). At least 6 inches of drain rock will underlie the ATB at the bottom of the basins. The current ground elevations within the basin areas range from ±El. 364 to ±El. 365. Therefore, the basin excavations will extend ±7.5 to 8.5 feet below the current ground and ±1 to 2 feet of fill will be required for site grading around the perimeter of the basins.

In this portion of the site, the alluvial gravel is relatively deep, ranging from ±8.5 to 12.5 feet (±El. 351.4 to ±El. 355.7). Therefore the excavations will likely terminate in stiff clayey silt, medium stiff sandy silt or medium dense silty sand. Our explorations to the west (TP-8 and TP-14) encountered alluvial gravel at depths of ±3 to 4.4 feet (±El. 360.4 to ±El. 361.3), therefore, it is possible gravel will be encountered in the western portion of the basin excavations.

These soils will likely be moisture-sensitive and susceptible to pumping, if wet of optimum. Therefore, it will be necessary to complete this work in the dry summer months. Even during the summer, the soils at the bottom of the basin excavations may remain wet of optimum. Therefore, overexcavation and replacement with angular quarry rock may be required for subgrade stabilization.

Underdrain. An underdrain system will be installed in the drain rock beneath the settling basins. We completed preliminary calculations to estimate pumping rates for the drainage system. For the analysis, the two basins were modeled as an equivalent circular well. Ground water was assumed at El. 363.0.

Our explorations in the vicinity of the settling basins indicate fine-grained soils and silty sand typically extend to ±El. 351.4 to ±El. 355.7 in the central and northern portion of the settling basins, and are underlain by sandy gravel. The gravel stratum is shallower (±El. 360.4 to ±El. 361.3) in our explorations to the west of the settling basins. Therefore, the foundation soils beneath the settling basins are likely to include a layer of silt or silty sand over sandy gravel. Sandy gravel may be encountered at the foundation grade in the southern portion of the basin excavations.

In the analysis, the soil is modeled with a single coefficient of permeability (k) and a drawdown radius has to be assumed. To account for the range of soil conditions, analyses were completed with k values of $3x10^{-5}$ cm/s (silt), $3x10^{-4}$ cm/s (silty sand), and $2x10^{-3}$ cm/s (sandy gravel). Drawdown radii of 125 and 150 feet were assumed based on the site conditions and basin geometry.

Figure 10A (Appendix A) shows the estimated pumping rates plotted as a function of k value, drawdown radii, and basin finish floor elevation. Finish floor elevations ranging from El. 358 to El. 363 were assumed. We assumed the ground water would be drawn down to the flowline of the underdrain, located ±2 feet below the finish floor elevation. The plots indicate a wide range of pumping rates depending on the assumed conditions with the greatest rates assuming the basins our founded on sandy gravel and the slowest rates assuming the basins are founded on silt. We anticipate the actual pumping rates will be between the values estimated for silty sand and sandy gravel. The sizing of the underdrain should be conservative, if it is designed for the flow rates estimated for the sandy gravel.

Sump Pit and Decant Box. Water from the drain system will be directed to a below grade sump pump. Preliminary plans indicate the finish floor of the sump pit will be at El. 344 and the floor will be at least 2 feet thick, extending the base to ±El. 342. The sump pit elevation may be adjusted, as the design progresses.

A settling basin decant box will also be constructed. The preliminary plans indicate the finish floor of the box will be at El. 355.2 and the floor will be at least 1 foot thick, extending the base to ±El. 354.2.

We recommend backfilling these below-grade structures using compacted Base Rock. Lateral earth pressures on the walls should be designed for at-rest pressures using an equivalent fluid density of 55 pcf above the design ground water level (El. 363.0). Below the ground water level, the at-rest pressures should be designed using a buoyant equivalent fluid density of 27.5 pcf (to represent the effective soil pressure) plus a hydrostatic pressure of 62.4 pcf.

Uplift pressures should be calculated based on the depth of the foundation below the design high ground water level (El. 363.0). Uplift pressure may be resisted using one of the following options:

- Option 1 - Oversize the base and use the moist and buoyant weights of the backfill above the oversized base. Assuming compacted Base Rock backfill is used, we recommend using a moist unit weight (γ_m) of 125 pcf above the design high ground water level (El. 363.0) and a buoyant unit weight (γ_b) of 63 pcf below the design high ground water level. Due to arching, the soil can be assumed to form an inverted pyramid wedge above the foundation base with a slope of 20 degrees measured from a vertical line extending up from the edges of the base.

- Option 2 – Do not oversize the base and use side friction between the concrete and backfill on the sides of structures. For round structures, the total side friction can be calculated as $0.5*k_o*\tan\delta*\gamma*\pi*D*H^2$, where k_o is the at-rest

earth pressure coefficient (0.44), tan δ is a sliding coefficient (0.40), D is the diameter, H is the total embedment depth, and γ is the unit weight of the backfill. A moist unit weight (γ_m) of 125 pcf should be used above the design high ground water level (El. 363.0) and a buoyant unit weight (γ_b) of 63 pcf should be used below. For square or rectangular structures, π*D should be replaced with the perimeter length of the structure.

Stormwater Swales

The infiltration testing indicates the silt and silty gravel that cap the WTP site have relatively low permeabilities. Therefore, to provide adequate drainage, we anticipate it will be necessary to trench beneath the base of the stormwater swales and replace the fine-grained soils with relatively free-draining granular material extending to the alluvial gravel.

Intake Structure

Bearing Capacity and Settlement

It is currently planned to drive sheet piles around the perimeter of the intake structure. The footprint of the intake will then be excavated to El. 333.5 and a ±3.5-foot thick unreinforced concrete slab will be poured over the subgrade to help seal the excavation and provide a working pad. The excavation will be dewatered and a ±1.5-foot thick reinforced concrete slab will be formed and poured.

Based on the available subsurface data, we anticipate the slab will bear on a relatively thin stratum of dense to very dense sandy gravel and cobbles underlain by sandstone. The alluvial gravel and sandstone have relatively high strength, therefore, bearing capacity should not be a design consideration for the lower intake. The foundation materials have low compressibility, therefore, settlement is expected to be minor (< ½ inch). The settlement, if any, is expected to occur during construction, as the structure is built.

The valve room and access platform for the intake structure will extend above the lower intake. We assume these portions of the structure will bear on at least 12 inches of compacted crushed Base Rock underlain by medium dense to dense silty sand or sandy gravel. An allowable bearing pressure of 2,000 psf. This value includes a factor of safety of 3 should be used to design the foundations for these portions of the structure. Settlement of these foundations is expected to be minor (< ½ inch) and should occur during construction.

Sliding Resistance

We recommend using a coefficient of sliding friction of 0.5 for evaluating the sliding resistance between the bottom of the unreinforced slab and the sandy gravel subgrade. This value assumes the subgrade surface will be relatively clean and undisturbed or recompacted. A coefficient of sliding friction of 0.7 may be used to evaluate the sliding resistance between the reinforced and unreinforced concrete slabs.

We recommend using a coefficient of sliding friction of 0.5 for evaluating the sliding resistance between the bottom of the valve room and access platform footings and slabs and the compacted Base Rock.

Lateral Earth Pressure and Hydrostatic Pressures for Temporary Shoring

Lateral pressures for the design of the temporary shoring were established based on the soil profile at BH-5. The soil conditions consist of medium dense silty gravel fill over dense to very dense sandy gravel and cobbles.

We anticipate the sheet piles will be relatively flexible and will likely deflect sufficiently to mobilize active earth pressures as the excavation progresses downward. Therefore, active earth pressure conditions were assumed. Strength parameters were assigned assuming the walls will support dense alluvial sandy gravel and cobbles. We assumed an internal friction angle (ϕ) of 36 degrees and a moist unit weight of 130 pcf for this material.

Figure 11A (Appendix A) provides a lateral pressure diagram for the design of the temporary shoring, based on active conditions. Ground water was assumed outside the cofferdam below $\pm$El. 343.4. This elevation is the Design Water Level shown on the plans.

Lateral earth pressures on the cofferdam walls due to surface surcharge pressures should be included in the shoring design. We recommend calculating the surface surcharge lateral earth pressures using a pressure distribution taking into account the footprint area and magnitude of the surface surcharge pressure and its proximity to the structure per AASHTO (2012) Section 3.11.6.2. A schematic of the pressure distribution is shown on Figure 11A. The shape of the curve and magnitude of the pressure will depend on the loading conditions and will need to be determined by the shoring designer based on the contractor's proposed construction approach and equipment.

The shoring system should be designed by a qualified engineer licensed in Oregon. We should review the shoring system design, if the actual sheet pile penetration, excavation depth, or ground water levels encountered during construction differ significantly from the assumed values.

Lateral Earth Pressure and Hydrostatic Pressure for the Permanent Structure

We recommend backfilling the intake structure using Base Rock or Stabilization Rock placed in lifts and compacted using vibratory equipment. Figures 12A through 14A (Appendix A) provide lateral earth pressure diagrams for the design of the permanent intake structure. Hydrostatic uplift pressures are also shown.

For static analysis, we considered two scenarios:

1) Earth pressures associated with the Design Water Level (±El. 343.4), and

2) Earth pressures associated with the 100-year flood elevation (±El. 360.5).

We assume the intake structure walls will be relatively rigid. Therefore, at-rest (k_o) earth pressures were used for static design. Strength parameters were assigned assuming the walls will support dense alluvial sandy gravel or dense, compacted Base Rock backfill. We assumed an internal friction angle (ϕ) of 36 degrees and a moist unit weight of 130 pcf for the backfill.

For seismic analysis, ground water was assumed at the Design Water Level (±El. 343.4). The static earth pressure component was calculated using active (k_a) earth pressures. The seismic earth pressure component (Δk_{ae}) was calculated using the Mononobe Okabe method. A horizontal acceleration coefficient (k_h) of 0.23g was used, calculated as the short period design spectral acceleration (S_{DS}) divided by 2.5.

We recommend evaluating the passive resistance on the north (river) side of the intake structure using an equivalent fluid density of ±237 pcf. This value assumes this portion of the intake structure footing will be backfilled with riprap as shown on the drawings. We understand a concrete key may be constructed at the bottom of the mat foundation to help resist lateral loads. Passive resistance against this key trench may also be calculated using an equivalent fluid density of ±237 pcf.

Lateral earth pressures on the buried walls of the intake structure due to surface surcharge pressures should also be included in the wall design. In consultation with the structural designer the surcharge pressure for the permanent structure was assumed to be a uniform lateral earth pressure calculated using the at-rest earth pressure coefficient (k_o) multiplied by an assumed surface surcharge pressure of 260 psf. Alternatively, the surcharge could be calculated using a surcharge pressure distribution similar to that recommended for the temporary shoring.

We anticipate the critical vertical loading condition will be uplift on the base of the intake due to hydrostatic pressure from ground water. A hydrostatic uplift pressure of ±618 psf was calculated for the design water level (El. 343.4) and a hydrostatic uplift pressure of ±1,685 psf was calculated for the 100-year flood.

Foundation Anchors

Tie-back foundation anchors will be used to help resist lateral loads on the intake structure. Six anchors are planned. The anchors will be drilled-and-grouted through the alluvial gravel and into the underlying bedrock. Solid bar or strand tie-back anchors may be used. Anchors can be constructed using a variety of materials and drilling methods, which can affect their performance. Therefore, we recommend the anchor diameters and bond lengths be selected by a design-build drilling contractor based on the required axial resistances provided by the structural

designer. We have limited our work to selecting the required anchor unbonded lengths, minimum bond lengths, corrosion protection requirements, and providing preliminary estimates of the nominal axial resistances. We have also estimated the total anchor length for preliminary bidding purposes.

Anchor Inclination. We recommend using an anchor installation between 20 and 30 degrees. This angle will assist in grouting the anchors by gravity methods. It will also shorten the distance between the wall and bedrock and may reduce the risk of conflicts between the drilling equipment and shoring. We understand an anchor inclination of 25 degrees is currently planned.

Unbonded Length. We recommend providing minimum unbonded length of 20 feet for either solid bar or strand anchors to extend the bond zone beyond the limits of the backfill zone and soil potentially disturbed by the foundation excavation work. Typical minimum unbonded lengths are 10 feet for solid bar anchors and 15 feet for strand anchors (FHWA, 1999).

Anchor Resistance, Minimum Bond Length and Estimated Total Length. The structural designer indicates the maximum working anchor load will be 100 kips. We recommend designing the anchors for an ultimate capacity of 200 kips assuming a typical factor of safety of 2.

The site is underlain by very dense, sandy gravel and cobbles over extremely weak to very weak (R0 to R1) volcaniclastic sandstone. Based on the available subsurface information, we estimate a distance of ±27 feet between the wall and the bedrock contact, assuming a 25 degree anchor inclination. The actual distance may vary.

We anticipate the anchors will be sleeved through the gravel stratum to mitigate caving issues, limit the grout take, and reduce the risk of grout seeping into the river. Therefore, we have assumed the anchors will develop their resistance solely in the bedrock.

Grout-to-rock bond resistance in the bedrock was estimated from available literature (FHWA, 1999). The literature indicates an average nominal grout-to-rock bond resistance in the range of ±29 to 116 psi in soft shale, 100 to 116 psi in weathered sandstone, and 116 to 247 psi in sandstone. These bond strengths assume gravity-grouted anchors. Moderately to highly weathered, R0 to R1 sandstone is anticipated. We believe 50 psi is a reasonable (possibly conservative) ultimate grout-to-rock bond strength for this material.

A minimum bond length of 21 feet was calculated based on the required ultimate capacity of 200 kips and assuming a nominal drilled anchor diameter of 5 inches and a grout-to-rock bond strength of 50 psi. Assuming a distance of ±27 feet between the wall and rock contact, we estimate a minimum anchor length of ±48 feet (excluding the tail on the tendon).

Table 1 summarizes the recommended unbonded length, minimum bond length, and estimated minimum total length for the anchors based on our preliminary calculations.

These lengths are intended for estimating purposes. The actual bond lengths and total lengths will need to be determined by the contractor responsible for designing the tie-back anchors. It may be prudent to assume longer anchors for bidding to account for potential variability in the depth to rock. The required tie-back anchor axial resistances will be confirmed with proof tests and performance tests.

Table 1. Estimated Tie-Back Anchor Lengths

Required Ultimate Resistance (kips)	Unbonded Length (ft)	Minimum Bond Length (ft)	Minimum Total Length (ft)
200	20	21	48

Note: Minimum total length assumes a minimum bond length of 21 feet and a distance of 27 feet between the wall and bedrock contact. The actual bond length to be determined by the contractor designing the tie-back anchors.

Corrosion Protection. The granular backfill fill and alluvial gravel is expected to have low corrosion potential. However, considering the importance of the structure and its anticipated life-span, PTI Class 1 corrosion protection is recommended for the anchors.

Raw Water Pump Station and Manholes

Bearing Capacity and Settlement

Similar to the intake structure, we anticipate sheet piles will be driven around the perimeter of the wet well and manholes. The footprint of the wet well and manholes will then be excavated to the planned grades. For the wet well, we assume an unreinforced concrete slab will be poured over the subgrade to help seal the excavation and provide a working pad. The excavation will then be dewatered and a reinforced concrete slab will be formed and poured for the base of the wet well. For the manholes, we anticipate the bases will be set on either an unreinforced concrete slab or a leveling course of compacted Base Rock.

Based on the available subsurface data, we anticipate the wet well and manholes will be supported on volcaniclastic sandstone or a thin mantle of dense sandy gravel and cobbles underlain by sandstone. The gravel and sandstone have relatively high strength and very low compressibility, therefore, bearing capacity and settlement will not be a design consideration. The settlement, if any, is expected to occur during construction, as the structures are built.

The footprint of the pump station building will extend beyond the limits of the wet well excavation. The building will have a slab-on-grade floor and will be supported on a continuous perimeter footing. This area is currently underlain by ±8 to 11 feet of loose sand and silty sand followed by very dense sandy gravel and cobbles. We recommend excavating the loose sand within the building area and replacing it with imported granular fill. In consultation with the design team, it was concluded Stabilization Rock will be used for the majority of the backfill. The larger Stabilization Rock will be capped with Base Rock within ±3 feet of the finish grade.

The excavation and backfilling should extend at least ±5 feet outside the building footprint.

Assuming the site is prepared as recommended herein, we recommend designing the shallow building foundations using an allowable bearing pressure of 2,500 psf. This value includes a factor of safety of 3. Foundation settlement is expected to be minor (< ½ inch) and should occur during construction.

Sliding and Passive Resistance

We have assumed the new footings will be constructed on compacted Base Rock. We assume the footings will be backfilled with fill, possibly including fine-grained soil. We recommend evaluating the sliding resistance of the footings using a coefficient of friction of 0.5 and a nominal passive resistance calculated as an equivalent fluid density of ±115 pcf.

Lateral Earth Pressure and Hydrostatic Pressures for Temporary Shoring

Lateral pressures for the design of the temporary shoring for the wet well and manholes were established based on the soil profile at BH-7. For shoring, we assumed the ground will be at the average existing ground surface (±El. 364.0).

We anticipate the sheet piles will be relatively flexible and will likely deflect sufficiently to mobilize active earth pressures as the excavation progresses downward. Therefore, active earth pressure conditions were assumed. Strength parameters were assigned assuming the walls will support ±11 feet of loose sand over dense to very dense sandy gravel and cobbles. We assumed an internal friction angle (ϕ) of 30 degrees and a moist unit weight of 115 pcf for the loose sand. We assumed an internal friction angle (ϕ) of 36 degrees and a moist unit weight of 130 pcf for the underlying dense to very dense sandy gravel and cobbles.
Figure 15A (Appendix A) provides a lateral pressure diagram for the design of the temporary shoring, based on active conditions. Ground water was assumed outside the sheet piles below ±El. 351.0. This elevation is the estimated high summer water level, based on the available ground water data.

Lateral earth pressures on the cofferdam walls due to surface surcharge pressures should be included in the shoring design. We recommend calculating the surface surcharge lateral earth pressures using a pressure distribution taking into account the footprint area and magnitude of the surface surcharge pressure and its proximity to the structure per AASHTO (2012) Section 3.11.6.2. A schematic of the pressure distribution is shown on Figure 15A. The shape of the curve and magnitude of the pressure will depend on the loading conditions and will need to be determined by the shoring designer based on the contractor's proposed construction approach and equipment.

The shoring system should be designed by a qualified engineer licensed in Oregon. We should review the shoring system design, if the actual sheet pile penetration,

excavation depth, or ground water levels encountered during construction differ significantly from the assumed values.

Lateral Earth Pressure and Hydrostatic Pressure for the Permanent Structure

We recommend backfilling the deeper excavations (below the alluvial gravel stratum) using Stabilization Rock or Base Rock placed in lifts and compacted using vibratory equipment. We assume crushed rock or recompacted sand or gravel will be used to backfill the structures above the alluvial gravel stratum.

Figures 16A through 18A (Appendix A) provide lateral earth pressure diagrams for the design of the permanent wet well. These earth pressure diagrams can also be used for the manholes, with adjustments made for excavation depths. For the permanent structures, we assumed the ground will be raise to El. 366.5.

For static analysis, we considered two scenarios:

1) Earth pressures associated with ground water at ±El. 351 (i.e., the estimated high summer water level, based on the available ground water data), and

2) Earth pressures associated with ground water at ±El. 361.0 (the estimated pump station ground water level corresponding to the 100-year flood.

We assume the walls will be relatively rigid. Therefore, at-rest (k_o) earth pressures were used for static design. Strength parameters were assigned assuming the walls will support dense alluvial sandy gravel and cobbles and/or dense, compacted granular backfill. We assumed an internal friction angle (ϕ) of 34 degrees and a moist unit weight of 120 pcf for the upper ±11 feet of backfill. We assumed an internal friction angle (ϕ) of 36 degrees and a moist unit weight of 130 pcf for the lower backfill.

For seismic analysis, ground water was assumed ground water at ±El. 351.0. The static earth pressure component was calculated using active (k_a) earth pressures. The seismic earth pressure component (Δk_{ae}) was calculated using the Mononobe Okabe method. A horizontal acceleration coefficient (k_h) of 0.25g was used, calculated as the short period design spectral acceleration (S_{DS}) divided by 2.5.

Lateral earth pressures due to surface surcharge pressures should also be included in the wall design. Consistent with the design of the intake structure, the surcharge pressure for the permanent wet well was assumed to be a uniform lateral earth pressure calculated using the at-rest earth pressure coefficient (k_o) multiplied by an assumed surface surcharge pressure of 260 psf. Alternatively, the surcharge could be calculated using a surcharge pressure distribution similar to that recommended for the temporary shoring.

We anticipate the critical vertical loading condition will be uplift on the base of the wet well due to hydrostatic pressure from ground water. A hydrostatic uplift pressure of ±1,810 psf was calculated for ground water at ±El. 351 (i.e., the anticipated high summer ground water level) and a hydrostatic uplift pressure of ±2,309 psf was calculated for ground water at ±El. 361.0 (i.e., the anticipated

ground water level associated with the 100-year flood). It should be noted that the uplift resistances on Figures 16A through 18A were calculated assuming water pressure from the water surface to the base of the excavation (below the bedrock surface). The lateral hydrostatic pressure was calculated down to the bedrock surface. The hydrostatic pressure diagrams may need to be adjusted based on the planned construction approach.

Pavement Design

Traffic at both sites is expected to include a combination of light passenger vehicles, mid-sized delivery trucks, garbage trucks and occasional heavy (80,000 lb/GVW) trucks. We completed pavement analysis using a computer program based on the AASHTO 1993 design method.

A subgrade resilient modulus (M_r) value of 4,500 psi was assumed for the WTP site assuming the pavement subgrade will be predominantly fine-grained soil. An M_r value of 5,000 psi was assumed for the pump station site assuming the pavement subgrade will be silty sand.

Our analysis indicates a minimum flexible pavement section of 4 inches of AC over 12 inches of crushed Base Rock for the WTP site and a minimum flexible pavement section of 3 inches of AC over 12 inches of crushed Base Rock for the pump station site. These minimum pavement sections assume the pavements will be built on compacted subgrade prepared as recommended herein.

A 20-year design life was assumed for the analysis. However, the pavement should be inspected by an experienced engineer every 2 to 3 years to determine its condition and need for rehabilitation.

For a rigid pavement provide a minimum of 7 inches of PCC over 6 inches of Base Rock. Specify a minimum compressive strength of 4000 psi and a minimum flexural strength of 600 psi for the PCC.

RECOMMENDATIONS

General Earthwork and Material Recommendations

1. Base Rock as defined in this report should consist of durable, clean well-graded, crushed gravel or rock meeting the requirements of ¾″ – 0 or 1″ – 0 Granular Structure Backfill in Section 00510.13 of ODOT (2015). We should be provided grain-size analysis for the intended fill for approval, prior to delivery to the site.

2. Stabilization Rock should consist of durable, clean, well-graded, angular quarry rock meeting the requirements of 2½″ – 0 Granular Structure Backfill in Section 00510.13 of ODOT (2015). Uncrushed (bar-run) gravel is not acceptable as Stabilization Rock. We should be provided

grain-size analysis for the intended fill for approval, prior to delivery to the site.

3. Granular Site Fill should consist of sand, gravel or rock, or mixtures of the above that are free of plastic clay, organic matter or construction debris.

4. Fine-Grained Site Fill should consist of approved soil that is free of organics, construction debris or expansive clay. Unless approved by us, silts or clays should not be placed under foundation areas or under settlement-sensitive structures.

5. Drain Rock used beneath the settling basins should satisfy the requirements of the settling basin designer. We recommend the Drain Rock consist of angular quarry rock to enhance constructability.

6. Controlled Low Strength Material (CLSM) should consist of highly flowable lean concrete with a 28-day unconfined compressive strength in the range of 100 to 200 psi.

7. Separation Geotextile as defined in this report should consist of a woven geotextile with Mean Average Roll Value (MARV) strength properties meeting the requirements in AASHTO M 288-06 Table 1 for a Class 2 geotextile (AASHTO, 2006).

   The geotextile should have MARV hydraulic properties meeting the requirements of AASHTO M 288-06 Table 3 with a permittivity greater than 0.10 $sec.^{-1}$ and an AOS less than 0.6 mm. We should be provided a specification sheet for the intended geotextile for approval prior to delivery to the site.

8. Efficient compaction of fine-grained soils will typically require the use of a padfoot or kneading roller to achieve the required compaction. Granular fill (sand, rock or gravel) will compact more efficiently with a smooth drum, vibratory roller.

   Adjust the moisture of the fill to within 2% of the optimum value prior to compaction. Compact all fill to a minimum of 95% relative compaction. The maximum dry density of ASTM D 698 should be used as the standard for estimating relative compaction. Compact all fill in loose lifts not exceeding 12 inches. Thinner lifts (8 inches or less) will be required where light or hand-operated equipment is used.

   Field density tests should be run frequently to confirm adequate compaction. Adequate compaction of materials that are too coarse or variable for density testing will need to be verified by observation of the compaction method. Any geotechnical engineering judgment in the field should be provided by a Foundation Engineering representative.

9. Overexcavate any test pits that extend beneath settlement-sensitive structures. Backfill the test pits with compacted Base Rock.

10. Contractors should anticipate ground water infiltration in the trenches, and pump station and intake structure excavations. Excavations should be pumped dry (if practical) prior to placing the backfill. It may be necessary to place Stabilization Rock, CLSM or concrete in the bottom of excavations, where it is not possible to completely remove the ground water.

11. Shoring will be needed in all trenches to protect workers from sloughing or caving soils. The silty soils typically correspond to an OR-OSHA Type B soil. The alluvial sands and gravels correspond to OR-OSHA Type C soil.

12. Deep excavations for the intake structure, wet wells and River Drive and Canal undercrossings will require shoring and dewatering.

WTP Facility Site Preparation

We recommend completing site grading for the WTP site during dry weather as follows:

13. Strip the ground ±4 inches or as required to remove roots and sod. Dispose of the strippings outside fill areas.

14. In the pavement areas rip, moisture-condition, and compact the subgrade to a depth of ±12 inches (as recommended in Item 8) prior to placing fill to raise the site. The pavement areas may be raised using a variety of materials including suitable materials from on-site excavations or excavations for the intake structure, pump station and utility trenches. A Foundation Engineering representative should confirm the subgrade conditions and suitability of proposed fill materials.

15. Beneath settlement-sensitive structures, excavate as required to leave no more than ±1.5 feet of existing fill/topsoil in place. Table 1B summarizes the fill/topsoil thicknesses in our explorations. This information can be used for estimating excavation depths. The excavations should extend ±5 feet beyond the edges of the structures.

    Rip, moisture-condition, and compact the subgrade in the excavations (as recommended in Item 8) prior to placing fill to raise the site. The performance of the structures will depend in part on the quality of the fill. Therefore, if practical, we recommend using Granular Site Fill or Base Rock to raise the grades under the structures. A Foundation Engineering representative should confirm the subgrade conditions and suitability of proposed fill materials.

16. Construct minimum 12-inch thick Base Rock building pads beneath floor slabs and mat foundations. Provide a minimum of 6 inches of Base Rock beneath the footings. Compact the Base Rock as recommended in Item 8.

17. Grade the finished ground surface surrounding the buildings to promote runoff away from the foundations.

WTP Facility Foundation Design

18. Design the WTP structures using the seismic parameters shown on Figure 8A (Appendix A).

19. Design all new footings using an allowable bearing pressure of 2,500 psf. A one-third increase in these allowable bearing pressures may be assumed for transient (i.e., wind and seismic) loading. These bearing pressures assume the foundations will be underlain by at least 6 inches of compacted Base Rock over stiff or dense fill.

20. Assume maximum total and differential settlements of ± ¾ inch and ± ½ inch, respectively, if the building footings are designed and built as recommended herein. For the water storage tank, assume a maximum settlement of ± 1.5 inches at the center and ± ¾ inch at the perimeter.

21. Use a modulus of subgrade reaction, k_s, of 200 pci for floor slab and mat foundation design. This value assumes the slabs will be supported on a minimum of 12 inches of compacted Base Rock over stiff or dense fill. Reinforce all new floor slabs to reduce the risk of cracking and warping.

22. Provide a suitable vapor barrier under building slabs (where needed) that is compatible with the proposed floor covering and the method of slab curing. Foundation drains will not be required provided the site is raised as planned and the ground surface is graded to promote runoff away from the foundations.

Settling Basins

23. Construct the settling basins according to the civil designer's recommendations. The foundation excavations should be completed using an excavator equipped with a smooth-edged bucket to limit subgrade disturbance. Compact the subgrade as shown on the plans. Stabilization Rock or open-graded angular drain rock may be required to stabilize the subgrade if the soils are soft or wet of optimum.

24. Soils taken from the settling basin excavations may be used to construct the berms. The berms should be placed and compacted as recommended in Item 8.

Stormwater Swales

25. Excavate the stormwater swales to the planned grades according to the civil designer's recommendations. Do not compact the subgrade.

26. Trench the bottom of the swales as recommended by the civil designer to drain surface water to the alluvial gravel stratum. The excavations should extend at least 2 feet below the top of the gravel stratum to remove the siltier gravel. Backfill the trenches with free-draining granular fill as recommended by the civil designer.

Intake Structure Site Preparation

We recommend completing site grading for the intake site during dry weather as follows:

27. Strip the ground ±4 inches or as required to remove roots and sod. Deeper grubbing depths will be required where trees are present. Dispose of all strippings outside of construction areas, but not on slopes.

28. Design the temporary shoring for the intake structure using the lateral loads shown on Figure 11A (Appendix A). The shoring system should be designed by an experienced engineer licensed in the State of Oregon. A nominal water table elevation of ±El. 343.4 was assumed for Figure 11A. Prior to construction, the contractor should measure the water level in the BH-5 piezometer to confirm the ground water elevation.

29. Provide a cofferdam for the intake construction. The cofferdam should be able to prevent sustained ground water flow through the side walls and any loss of ground due to internal piping. A slide rail system is not recommended due to the presence of relatively shallow ground water and sandy soils.

30. We anticipate an internally-braced sheet pile cofferdam will be required. Hard driving should be anticipated in the dense sandy gravel and cobbles. Sheet piles with a web thickness of ±½ inch and tip reinforcement are recommended to help the sheets penetrate the sandy gravel and cobbles and reduce the risk of tip damage. If practical, the sheet pile tips should be seated into the bedrock to help seal the bottom of the cofferdam and reduce ground water infiltration.

    Difficult driving should be anticipated and the sheet pile penetration may be variable. Some of the sheet piles may encounter refusal before reaching bedrock due to the density of the gravel and the presence of large cobbles. It may be necessary to use an excavator to loosen the gravelly soil to help facilitate pile driving.

In the event it is impractical to drive the sheet piles to bedrock, a secondary method of ground water control may be required. Where sheet piles meet refusal above the bedrock surface, one option may be to grout the gravels outside the sheets. Another option may be to provide a seal of CLSM or concrete inside the cofferdam.

31. Excavate to the planned the foundation grade. Remove any loose debris from the bottom of the excavation. A Foundation Engineering representative should confirm the suitability of the subgrade prior to backfilling.

32. Install dewatering wells, well points, deep sumps or a combination thereof, as required, to provide continuous pumping of ground water until the entire structure is backfilled and capable of withstanding the hydrostatic pressures. The dewatering should maintain the ground water level at or below the bottom of the foundation.

33. Pour unreinforced concrete or CLSM over the finished subgrade to help seal the base of the excavation and provide a leveling pad for foundation construction.

34. Once the intake structure is completed, backfill the sides of the lower intake structure with Base Rock or CSLM. Base Rock, if used as backfill, should be placed in 12-inch thick maximum lifts and compacted with 1 pass of a jumping-jack type compactor. A hoe-pac style compactor should only be used with extreme caution to avoid overstressing the structure. Use riprap to backfill the toe of the structure at the river bank.

    If the sheet piles are removed, they should be extracted in a manner that will not lift or loosen the backfill. Where the sheet piles are incorporated into the active dewatering method or used as a form for the foundation, the sheet piles may need to be cut off and portions left in place.

35. Bench and compact the subgrade beneath the valve room and access platform. Use Granular Site Fill or Base Rock to grade the terrain beneath these structures. Compact the fill as recommended in Item 8. Provide a minimum of 12 inches of compacted Base Rock beneath the valve room and access platform foundations.

Intake Structure Foundation Design

36. Design the WTP structures using the seismic parameters shown on Figure 9A.

37. Design the intake structure for the lateral and hydrostatic uplift pressures shown in Figures 12A through 14A (Appendix A). If an oversized base is constructed, wall backfill placed above the base may

be used to help resist hydrostatic uplift pressures. If Base Rock is used as backfill, assume the buoyant unit weight of ±63 pcf for the backfill mass. If CLSM is used as backfill, assume the buoyant unit weight of ±78 pcf for the backfill mass

38. Bearing capacity will not be a design issue for the lower intake structure. Design the footings for the valve room and access platform using an allowable bearing pressure of 2,000 psf. A one-third increase in these allowable bearing pressures may be assumed for transient (i.e., wind and seismic) loading. These bearing pressures assume the foundations will be underlain by at least 12 inches of compacted Base Rock over dense fill.

39. Assume a total foundation settlement up to ± ½ inch, if the footings are designed and built as recommended herein.

40. Provide a suitable vapor barrier under the valve room and access platform slabs (if needed) that is compatible with the proposed floor covering, if any, and the method of slab curing. Foundation drains will not be required provided the site is raised as planned and the ground surface is graded to promote runoff away from the foundations.

Intake Structure Foundation Anchor Construction and Testing

Construction recommendations contained herein refer to the ODOT Geotechnical Design Manual (ODOT, 2011) and the FHWA Ground Anchors and Anchor Systems Design Manual (FHWA, 1999)

41. The foundation anchors will be drilled through very dense, sandy gravel and cobbles (alluvium) over extremely weak to very weak (R0 to R1) volcaniclastic sandstone. We anticipate temporary casing or sleeves will be required in the alluvial gravel to mitigate caving issues, limit the grout take, and reduce the risk of grout seeping into the river. The drilling subcontractor should be prepared to drill into rock with an unconfined compressive strength up to 1,000 psi.

42. Based on the available subsurface information, we estimate a distance of ±27 feet between the wall and the bedrock contact, assuming a 25 degree anchor inclination. The actual distance may vary. A Foundation Engineering representative should be present during drilling to confirm the subsurface conditions and contact with the bedrock.

43. We recommend providing an unbonded length of 20 feet and a minimum bond length of 21 feet into bedrock. Based on the estimated distance between the wall and bedrock, we anticipate a minimum anchor length of 48 feet will be required (excluding the tail on the tendon). The actual bond length and total length will be selected by the contractor.

44. It will be extremely difficult or impractical to replace an anchor following construction of the intake structure. Therefore, the contractor should consider this risk when evaluating anchor lengths. We recommend the anchors include a secondary pressure grouting system to allow for regrouting any anchors that do not develop the required capacity during testing.

45. It will be important to protect the anchors from damage during construction. We recommend the contractor prepare a construction plan detailing how the work is to be completed. The construction plan should be submitted for review and approval prior to construction.

46. Each anchor should be tested following the procedures in FHWA (1999) and ODOT (2011) to confirm the required axial resistance are achieved. We anticipate the testing will need to be completed after the intake wall is constructed to provide a reaction force. The intake wall should be backfilled at least 1 foot above the anchor elevation prior to testing to provide adequate passive resistance and reduce the risk of wall damage.

47. We recommend Performance Testing at least two anchors. Performance Testing involves incrementally loading and unloading the tie-back anchor in cycles up to 100% of the factored design load. All other tie-back anchors should be Proof Tested. Proof Testing involves incrementally loading the tie-back anchor up to 100% of the factored design load in a single cycle. Creep testing should be part of the test procedures. Load testing increments and testing requirements are provided in the project specifications.

48. Following testing, tension and lock-off the tie-back anchors at ±40 percent of the design load (i.e., ±40 kips) to allow for some wall deflection and reduce the risk of overstressing the anchors during final backfilling.

Raw Water Pump Station Facility Site Preparation

We recommend the site grading for the pump station site be completed during dry weather as follows:

49. Strip the ground ±4 inches or as required to remove roots and sod. Deeper grubbing depths will be required where tree stumps are present. Dispose of all strippings outside of construction areas.

50. In the pavement areas rip, moisture-condition and compact the subgrade as recommended in Item 8 prior to placing fill to raise the site. The pavement areas may be raised using a variety of materials including suitable materials from on-site excavations or excavations for the intake structure and utility trenches. A Foundation Engineering representative should confirm the subgrade conditions and suitability of proposed fill materials.

51. Design the temporary shoring for the wet well, manhole and River Drive undercrossing jacking and receiving pits using the lateral loads shown on Figure 15A (Appendix A). Adjust the diagrams as required based on the planned grades. The shoring system should be designed by an experienced engineer licensed in the State of Oregon. A nominal water table elevation of ±El. 351.0 was assumed for design, based on the anticipated high summer water level. Prior to construction, the contractor should measure the water level in the BH-5 and BH-7 piezometers to confirm the ground water elevation.

52. Provide cofferdams for the excavations. The cofferdams should be able to prevent sustained ground water flow through the side walls and any loss of ground due to internal piping. A slide rail system is not recommended due to the presence of relatively shallow ground water and sandy soils.

53. We anticipate an internally-braced sheet pile cofferdam will be required. Hard driving should be anticipated in the dense sandy gravel and cobbles. Sheet piles with a web thickness of ±½ inch and tip reinforcement are recommended to help the sheets penetrate the sandy gravel and cobbles and reduce the risk of tip damage. If practical, the sheet pile tips should be seated into the bedrock to help seal the bottom of the cofferdam and reduce ground water infiltration.

    Difficult driving should be anticipated and the sheet pile penetration may be variable. Some of the sheet piles may encounter refusal before reaching bedrock due to the density of the gravel and the presence of large cobbles. It may be necessary to use an excavator to loosen the gravelly soil to help facilitate pile driving.

    In the event it is impractical to drive the sheet piles to bedrock, a secondary method of ground water control may be required. Where sheet piles meet refusal above the bedrock surface, one option may be to grout the gravels outside the sheets. Another option may be to provide a seal of CLSM or concrete inside the cofferdam.

54. Install dewatering wells, well points, deep sumps or a combination thereof, as required, to provide continuous pumping of ground water until the structures are backfilled and capable of withstanding the hydrostatic pressures. The dewatering should maintain the ground water level at or below the bottom of the foundations until the structure is completed and backfilled sufficiently to resist buoyancy forces.

55. Excavate to the planned the foundation grades. Remove any loose debris from the bottom of the excavations. A Foundation Engineering representative should confirm the suitability of the subgrade prior to backfilling.

56. Pour unreinforced concrete or CLSM over the finished subgrade to help seal the base of the excavations and provide a leveling pad for foundation construction.

57. Once the structures are completed, backfill the sides of the structures with Base Rock or CSLM to the top of the alluvial gravel stratum (±El. 353). Base Rock, if used as backfill, should be placed in 12-inch thick maximum lifts and compacted with 1 pass of a jumping-jack type compactor. A hoe-pac style compactor should only be used with extreme caution to avoid overstressing the structure.

    If the sheet piles are removed, they should be extracted in a manner that will not lift or loosen the backfill. Where the sheet piles are incorporated into the active dewatering method or used as a form for the foundation, the sheet piles may need to be cut off and portions left in place.

58. Within the footprint of the pump station building and other settlement-sensitive structures (outside the wet well excavation), excavate all existing loose alluvial sand and silty sand above the gravel stratum (i.e., above ±El. 353). The excavation should extend at least 10 feet beyond the edges of the pump station building.

59. Use Stabilization Rock to backfill the building excavation to ±3 feet below the finish grade. Use Base Rock to backfill the upper portion of the excavation. Compact the backfill as recommended in Item 8.

60. Grade the finished ground surface surrounding the pump station building to promote runoff away from the foundations.

Raw Water Pump Station Foundation Design

Foundation construction should be in general accordance with the following recommendations.

61. Design the pump station structures using the seismic parameters shown on Figure 8A (Appendix A).

62. Design the wet well and manholes for the lateral and hydrostatic uplift pressures shown in Figures 16A through 18A (Appendix A). If an oversized base is constructed, wall backfill placed above the bases may be used to help resist hydrostatic uplift pressures. If Base Rock is used as backfill, assume the buoyant unit weight of ±63 pcf for the backfill mass. If CLSM is used as backfill, assume the buoyant unit weight of ±78 pcf for the backfill mass.

63. Bearing capacity will not be an issue for the wet well or manholes. Design the shallow footings for the pump station building using an allowable bearing pressure of 2,500 psf. A one-third increase in these allowable bearing pressures may be assumed for transient (i.e., wind

and seismic) loading. These bearing pressures assume the foundations will be underlain by at least 6 inches of compacted Base Rock over stiff or dense fill.

64. Assume maximum total and differential settlements of ± ¾ inch and ± ½ inch, respectively, if the building footings are designed and built as recommended herein.

65. Use a modulus of subgrade reaction, k_s, of 200 pci for floor slab design. This value assumes the slab will be supported on a minimum of 12 inches of compacted Base Rock over stiff or dense fill. Reinforce all new floor slabs to reduce the risk of cracking and warping.

66. Provide a suitable vapor barrier under building slab (where needed) that is compatible with the proposed floor covering, if any, and the method of slab curing. Foundation drains will not be required provided the site is raised as planned and the ground surface is graded to promote runoff away from the foundations.

Water and Sewer Transmission Line Construction

67. Excavate the trenches to the required grade for the transmission lines. Provide shoring and dewatering for the trenches, as required. The loose sandy soils will tend to flow in the presence of ground water.

68. Trenches should be pumped dry prior to placing the backfill. Use Stabilization Rock in the base of the trenches where complete dewatering is not possible.

69. Use Base Rock as backfill in the pipe zone. Base Rock should also be used for the balance of the backfill for trenches that extend beneath the structures, pavements and hardscapes. Compact the backfill in lifts as recommended in Item 8.

70. Granular Site Fill may be used to backfill above the pipe zone in undeveloped areas. It should be assumed it may be difficult to adequately compact this material. Therefore, settlement of the trenches and future grading of the ground surface should be anticipated where this material is used.

Pavement Construction

71. Construction the pavements on subgrade prepared as recommended in Item 8. Proof-roll the subgrade prior to placing Base Rock to identify any soft or pumping areas. Moisture-condition and recompact soft subgrade or overexcavate and replace the soft soil with Base Rock or Stabilization Rock. Maintain the moisture in the subgrade to prevent excessive drying and cracking.

72. Prior to placing Base Rock, cover the prepared subgrade with a Separation Geotextile. The Separation Geotextile should be laid smooth, without wrinkles or folds, in the direction of construction traffic. Overlap adjacent rolls a minimum of 2 feet. Pin fabric overlaps or place the granular fill in a manner that will not separate the overlap during construction. Seams that have separated will require removal of the granular fill to establish the required overlap.

73. Place Base Rock over the Separation Geotextile to construct the base course. Compact the Base Rock as recommended in Item 8.

74. Provide a minimum flexible pavement section of 4 inches of AC over 12 inches of Base Rock for the WTP site. Provide a minimum flexible pavement section of 3 inches of AC over 12 inches of Base Rock for the pump station site.

75. Compact the AC to a minimum of 91% relative compaction according to the theoretical maximum density calculated from the Rice specific gravity.

76. For rigid pavement, provide a minimum of 7 inches of PCC over 6 inches of Base Rock. Specify a minimum compressive strength of 4000 psi and a minimum flexural strength of 600 psi for the PCC.

DESIGN REVIEW/CONSTRUCTION OBSERVATION/TESTING

Foundation Engineering should be provided the opportunity to review all drawings and specifications that pertain to site preparation, foundation construction, foundation anchor construction and site grading. Site preparation will require field confirmation of the foundation conditions in accordance with recommendations provided herein. Mitigation of any subgrade pumping will also require engineering review and judgment. Frequent field density tests should be run on all engineered fill. Compaction of fill that is too coarse or variable for density testing should be evaluated by observation of the compaction method and proof-rolling with a loaded dump truck or other approved vehicle. We recommend we be retained to provide the necessary geotechnical construction observation and consultation.

We assume the sheet pile cofferdam and dewatering design will be provided by others. We are available to provide geotechnical input and consultation, as appropriate, to those designers.

VARIATION OF SUBSURFACE CONDITIONS, USE OF THIS REPORT AND WARRANTY

The analyses, conclusions and recommendations contained herein assume the soil conditions and ground water levels encountered in the explorations are representative of the site conditions. The above recommendations assume we will have the opportunity to review final drawings and be present during construction to confirm

the assumed foundation conditions. No changes in the enclosed recommendations should be made without our approval. We will assume no responsibility or liability for any engineering judgment, inspection or testing performed by others.

This report was prepared for the exclusive use of Carollo Engineers, Inc. and their design consultants for the Lebanon Water Treatment Plant project in Lebanon, Oregon. Information contained herein should not be used for other sites or for unanticipated construction without our written consent. This report is intended for planning and design purposes. Contractors using this information to estimate construction quantities or costs do so at their own risk. Our services do not include any survey or assessment of potential surface contamination or contamination of the soil or ground water by hazardous or toxic materials. We assume those services, if needed, have been completed by others.

Climate conditions in western Oregon typically consist of wet weather for almost half of the year (typically between mid-October and late May). It is assumed adequate drainage will be provided for construction. The recommendations for site preparation and foundation drainage are not intended to represent any warranty (expressed or implied) against the growth of mold, mildew or other organisms that grow in a humid or moist environment.

Our work was done in accordance with generally accepted soil and foundation engineering practices. No other warranty, expressed or implied, is made.

REFERENCES

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Eugene, 2014, *Eugene Stormwater Management Manual*, City of Eugene, Oregon, 2014.

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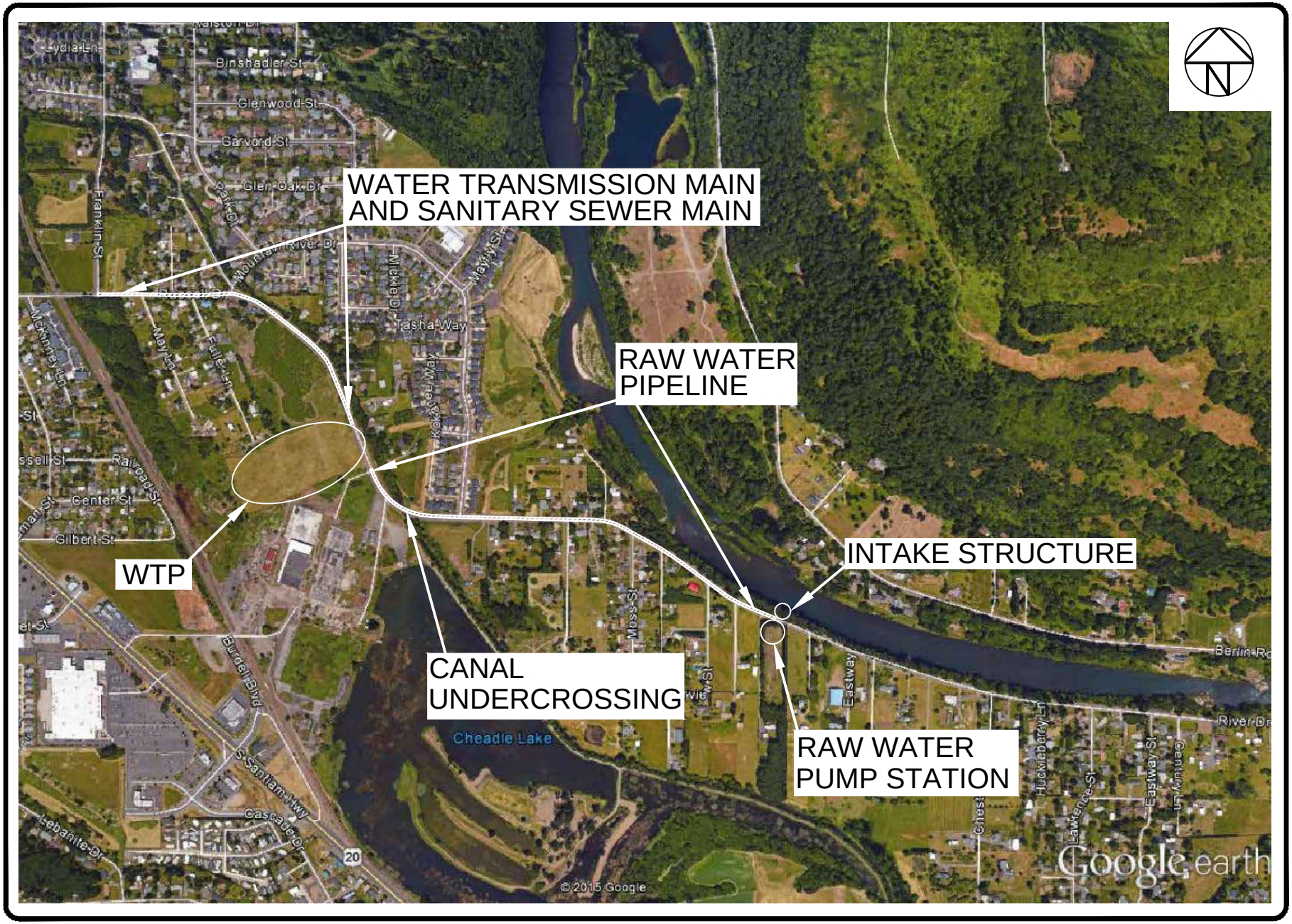
Appendix A

Figures

Professional
Geotechnical
Services

Foundation Engineering, Inc.



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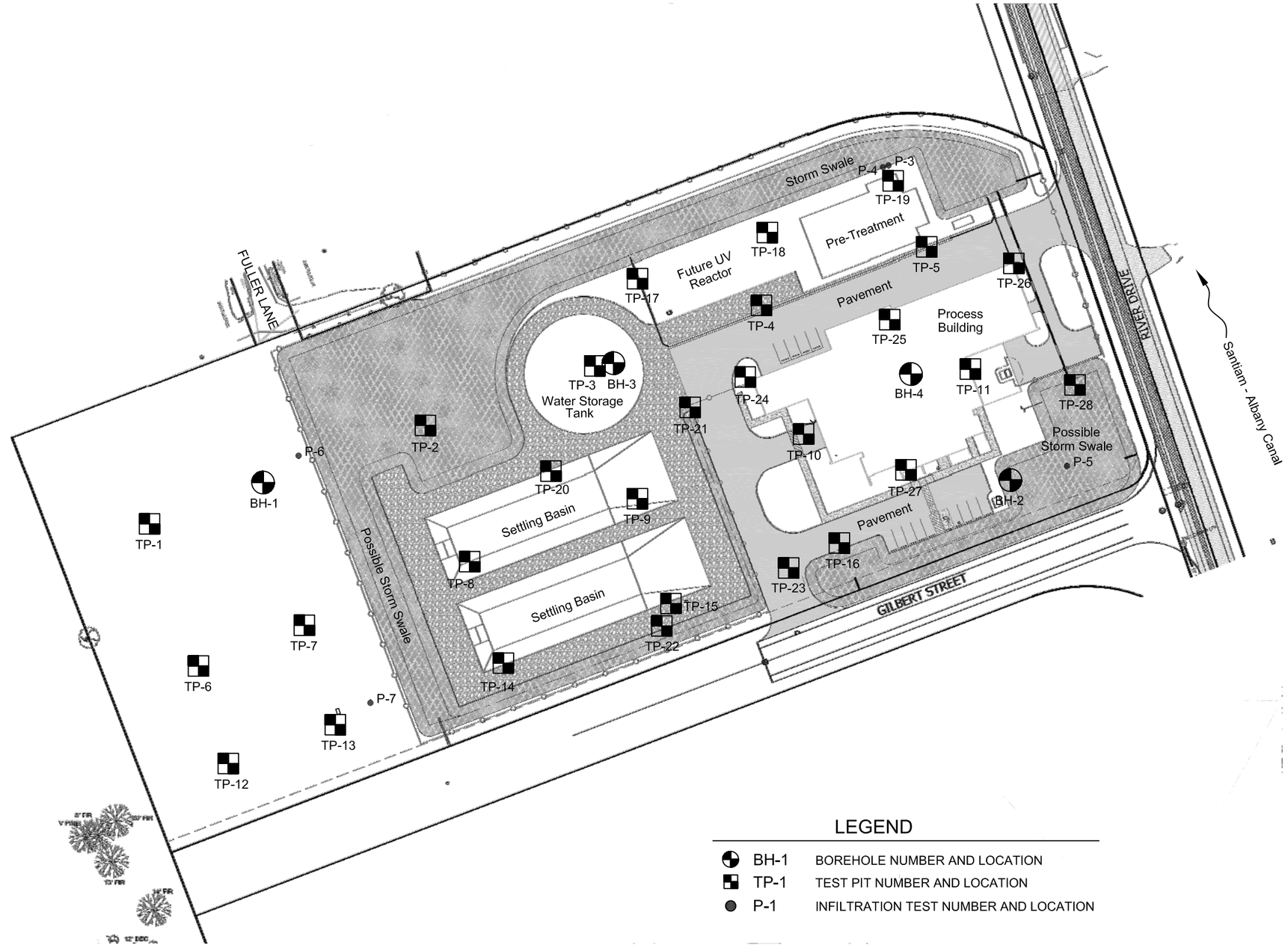
VICINITY MAP

LEBANON WATER TREATMENT PLANT
LEBANON, OREGON

FIGURE NO.
1A

FILE NAME:FIG 1A



NOTES:
1. TEST PIT AND BORING LOCATIONS WERE SURVEYED BY CRAWFORD, DRUMMOND AND ASSOCIATES, INC.
2. SEE REPORT FOR A DISCUSSION OF SUBSURFACE CONDITIONS.
3. BASE MAP WAS PROVIDED BY UDELL ENGINEERING.

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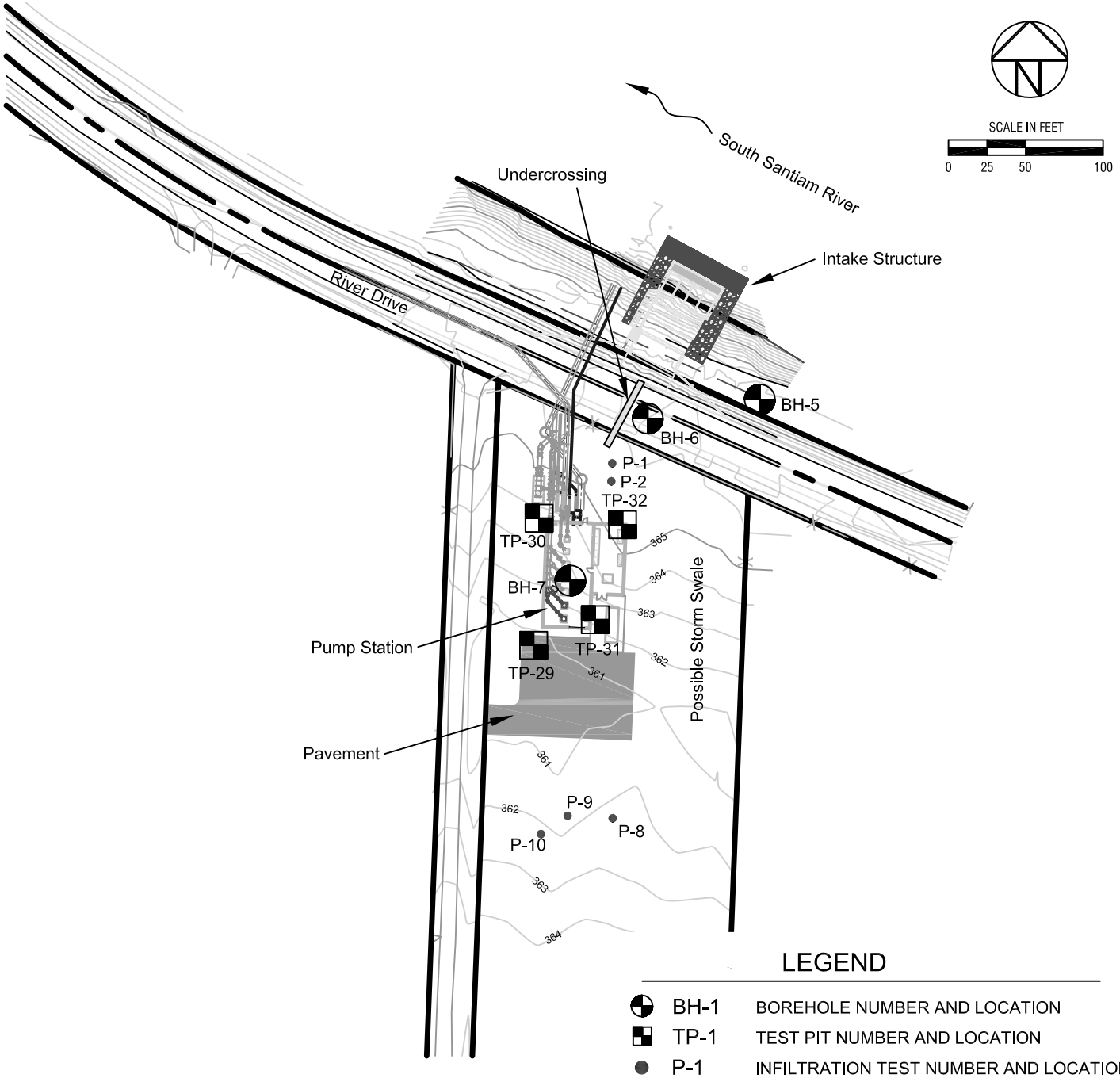
SITE LAYOUT AND EXPLORATION LOCATIONS
WATER TREATMENT PLANT
LEBANON WATER TREATMENT PLANT
LEBANON, OREGON

FIGURE NO.
2A

FILE NAME:FIG 2A



NOTES:
1. TEST PIT AND BORING LOCATIONS WERE SURVEYED BY CRAWFORD, DRUMMOND AND ASSOCIATES, INC.
2. SEE REPORT FOR A DISCUSSION OF SUBSURFACE CONDITIONS.
3. BASE MAP WAS PROVIDED BY UDELL ENGINEERING.

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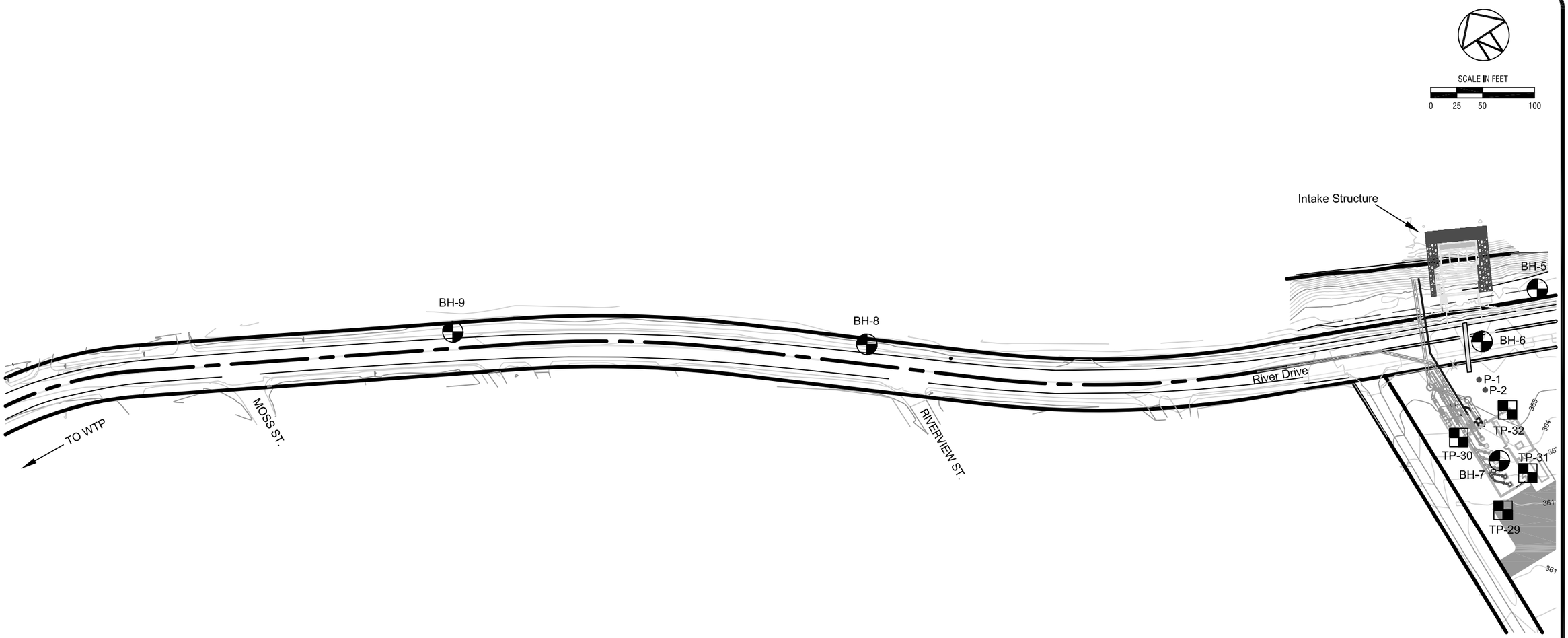
SITE LAYOUT AND EXPLORATION LOCATIONS
INTAKE STRUCTURE & RAW WATER PUMP STATION
LEBANON WATER TREATMENT PLANT
LEBANON, OREGON

FIGURE NO.
3A

FILE NAME: FIG 3A



LEGEND

BH-1	BOREHOLE NUMBER AND LOCATION
TP-1	TEST PIT NUMBER AND LOCATION
P-1	INFILTRATION TEST NUMBER AND LOCATION

NOTES:
1. TEST PIT AND BORING LOCATIONS WERE SURVEYED BY CRAWFORD, DRUMMOND AND ASSOCIATES, INC.
2. SEE REPORT FOR A DISCUSSION OF SUBSURFACE CONDITIONS.
3. BASE MAP WAS PROVIDED BY UDELL ENGINEERING.

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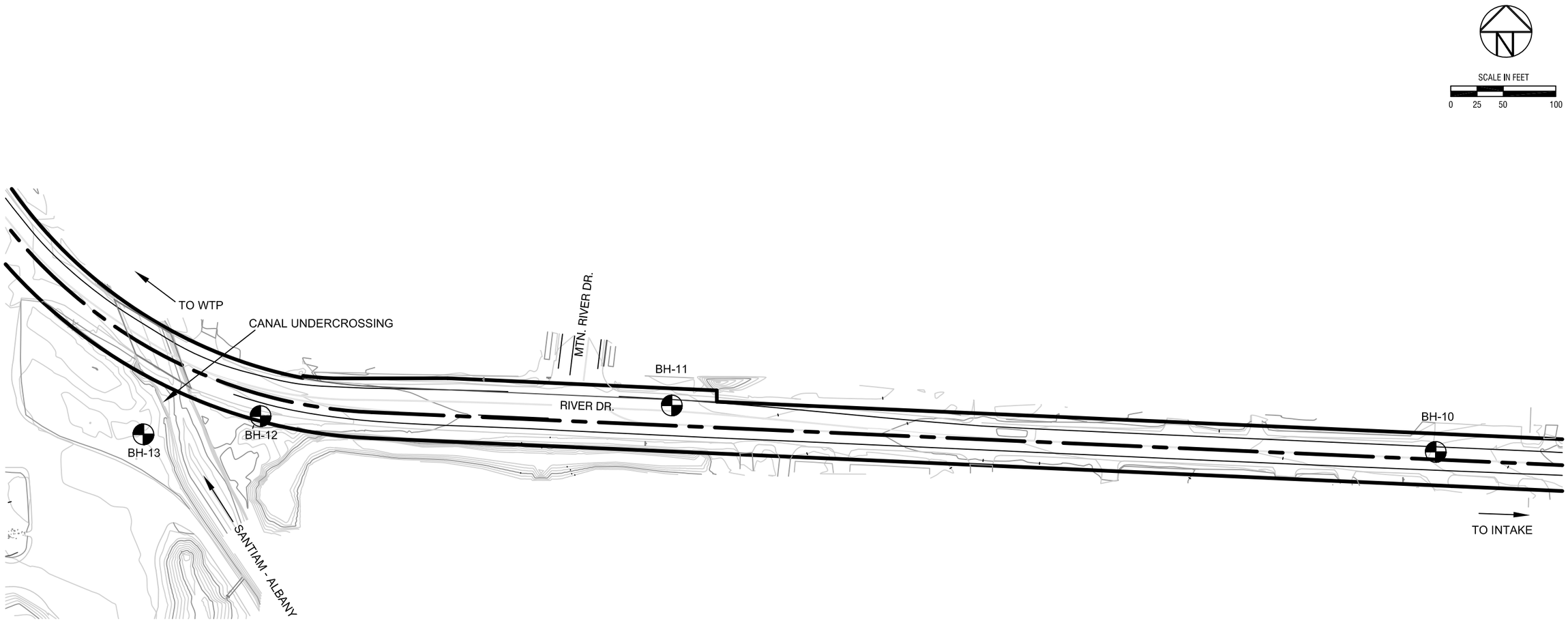
SITE LAYOUT AND EXPLORATION LOCATIONS
RAW WATER PIPELINE - EAST
LEBANON WATER TREATMENT PLANT
LEBANON, OREGON

FIGURE NO.
4A

FILE NAME:FIG 4A



LEGEND

BH-1 BOREHOLE NUMBER AND LOCATION

NOTES:
1. TEST PIT AND BORING LOCATIONS WERE SURVEYED BY CRAWFORD, DRUMMOND AND ASSOCIATES, INC.
2. SEE REPORT FOR A DISCUSSION OF SUBSURFACE CONDITIONS.
3. BASE MAP WAS PROVIDED BY UDELL ENGINEERING.

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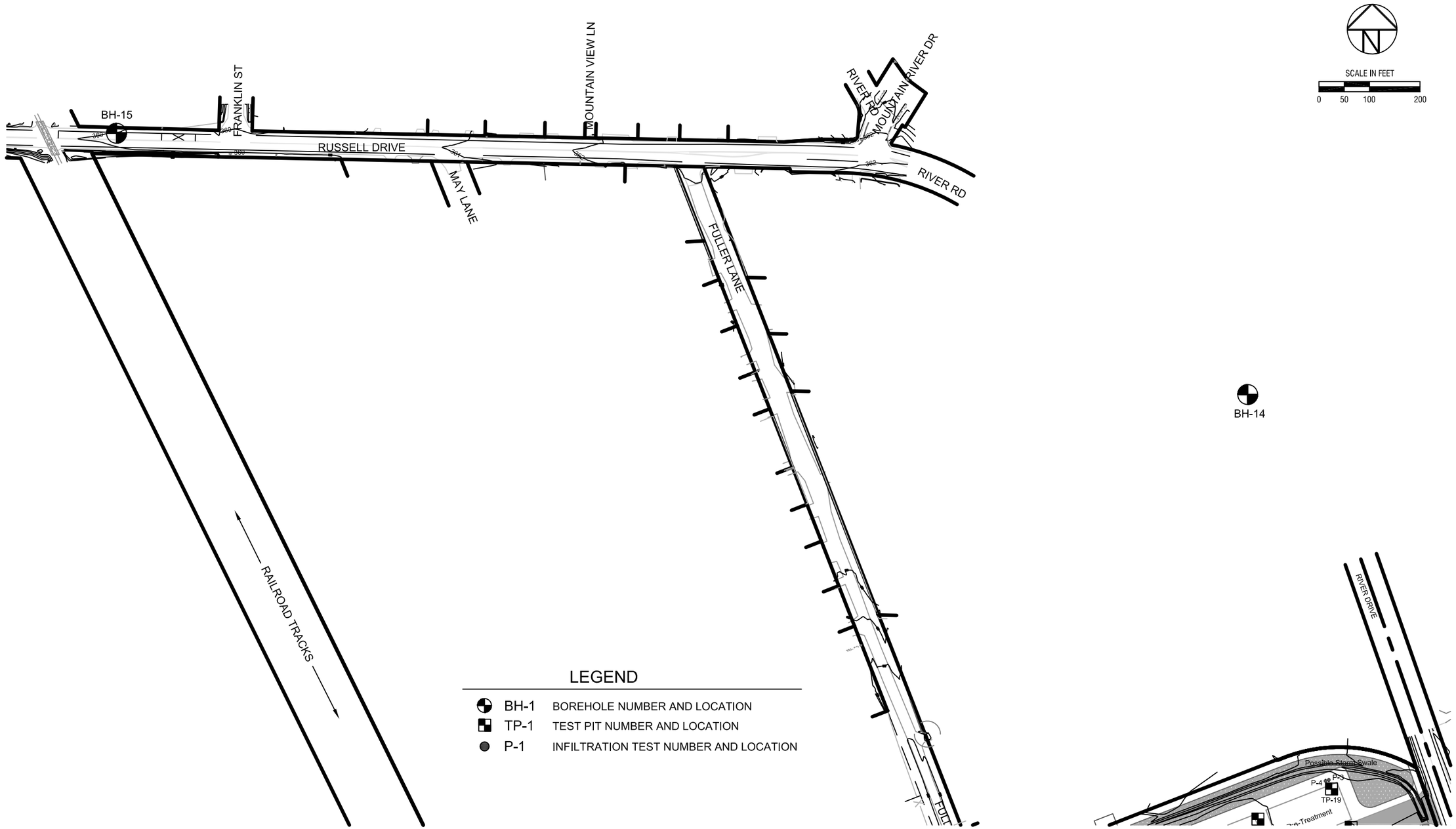
SITE LAYOUT AND EXPLORATION LOCATIONS
RAW WATER PIPELINE - WEST
LEBANON WATER TREATMENT PLANT
LEBANON, OREGON

FIGURE NO.
5A

FILE NAME:FIG 5A



NOTES:
1. TEST PIT AND BORING LOCATIONS WERE SURVEYED BY CRAWFORD, DRUMMOND AND ASSOCIATES, INC.
2. SEE REPORT FOR A DISCUSSION OF SUBSURFACE CONDITIONS.
3. BASE MAP WAS PROVIDED BY UDELL ENGINEERING.

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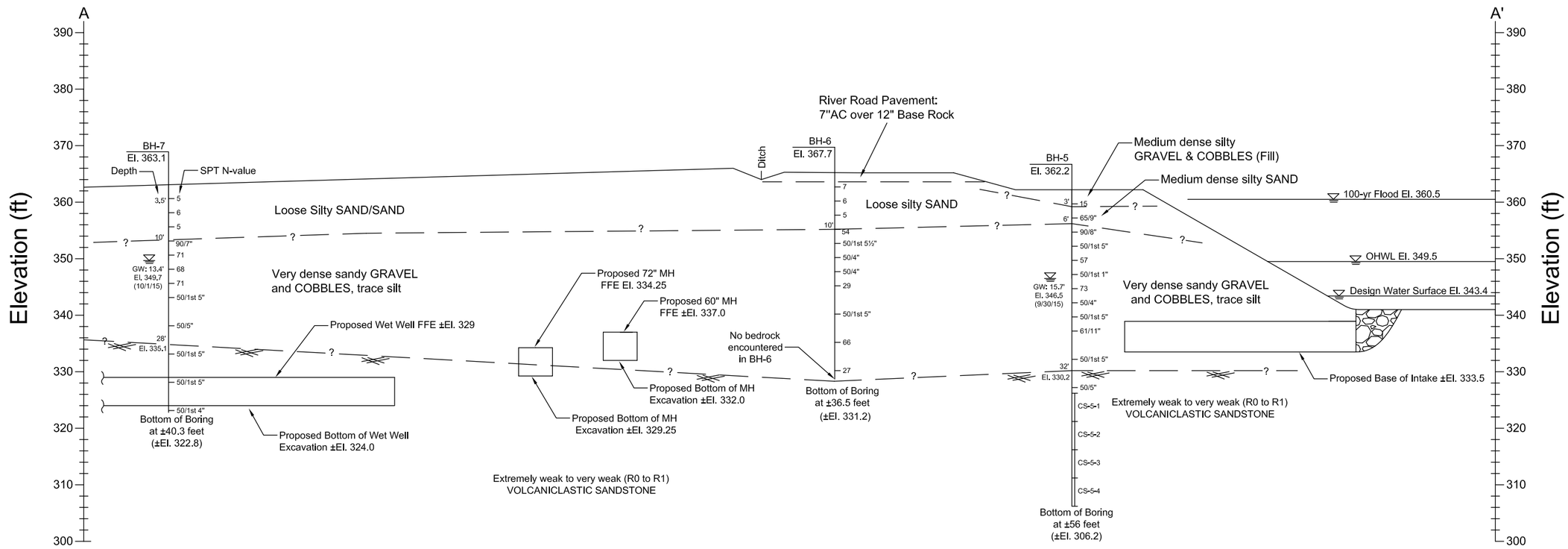
SITE LAYOUT AND EXPLORATION LOCATIONS
WATER TRANSMISSION MAIN AND SANITARY SEWER MAIN
LEBANON WATER TREATMENT PLANT
LEBANON, OREGON

FIGURE NO.
6A

FILE NAME:FIG 6A



SCALE IN FEET

0 5 10 20

NOTES:
1. GROUND SURFACE IS BASED ON TOPOGRAPHIC MAP PREPARED BY UDELL ENGINEERING.
2. BEDROCK SURFACE AND SOIL LAYERS INTERPOLATED FROM ADJACENT BORINGS.
3. BASE OF INTAKE, DESIGN WATER LEVEL AND OHWL AT THE RIVER ARE BASED ON PERMIT DRAWINGS PREPARED BY RESOURCE CONSULTANTS, INC.

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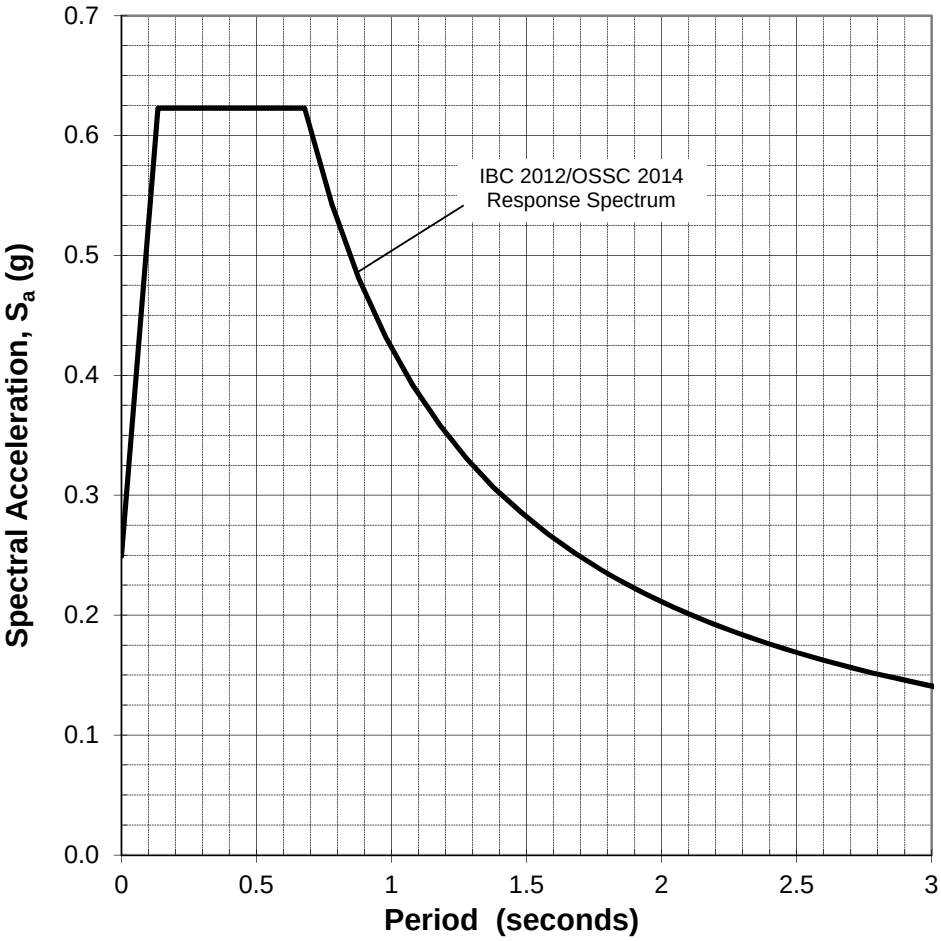
CROSS SECTION A-A'
RAW WATER PUMP STATION AND INTAKE SITES
LEBANON WATER TREATMENT PLANT
LEBANON, OREGON

FIGURE NO.
7A

FILE NAME:FIG 7A



Notes:

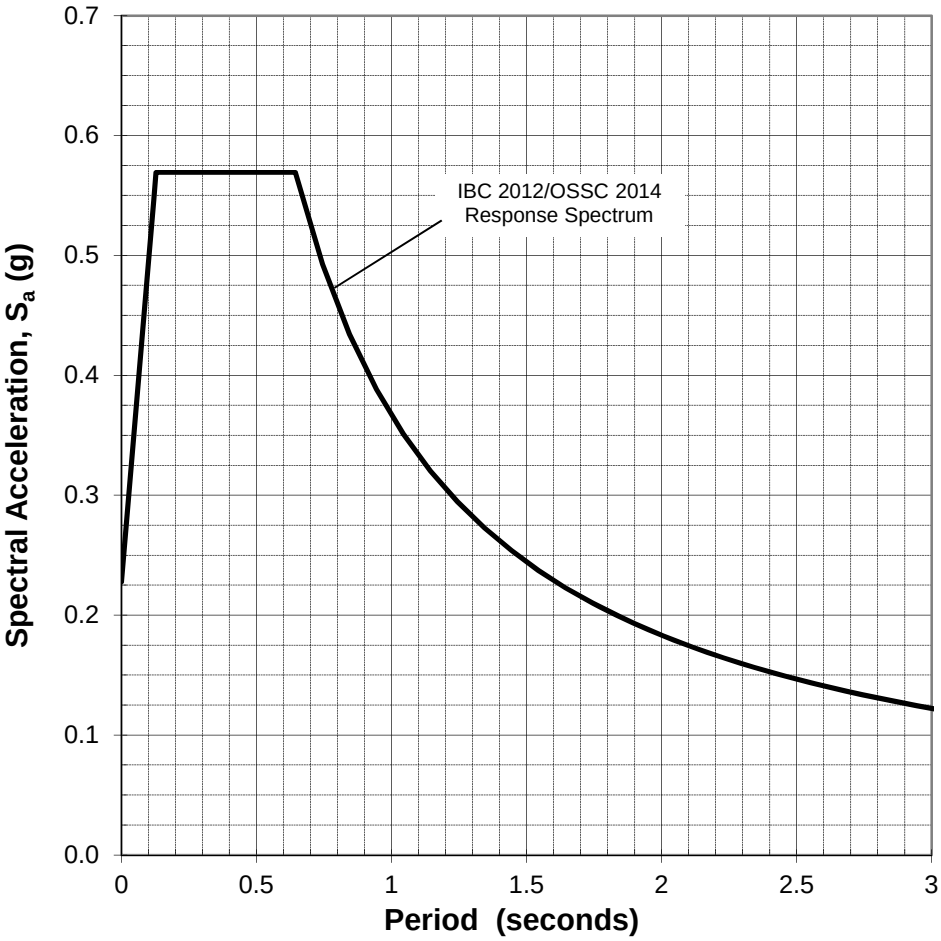
1. The Design Response Spectrum is based on IBC 2012 Section 1613.
2. The following parameters are based on the modified USGS 2008 maps provided in IBC 2012/OSSC 2014:

   Site Class= D Damping = 5%

S_S = 0.79	F_a = 1.18	S_{MS} = 0.93	S_{DS} = 0.62
S_1 = 0.39	F_v = 1.62	S_{M1} = 0.63	S_{D1} = 0.42

3. S_S and S_1 values indicated in Note 2 are the mapped, risk-targeted maximum considered earthquake spectral acclerations for 1% probability of exceedence in 50 years.
4. F_a and F_v were established based on IBC 2012, Tables 1613.3.3(1) and 1613.3.3(2) using the selected S_S and S_1 values. S_{DS} and S_{D1} values include a 2/3 reduction on S_{MS} and S_{M1} as discussed in IBC 2012 Section 1613.3.4.
5. Site location is: Latitude 44.5200, Longitude -122.8876.

FIGURE 8A
IBC 2012/OSSC 2014 SITE RESPONSE SPECTRUM
WTP STRUCTURES AND PUMP STATION
Lebanon Water Treatment Plant
Lebanon, Oregon
Project 2131070-101



Notes:

1. The Design Response Spectrum is based on IBC 2012 Section 1613.
2. The following parameters are based on the modified USGS 2008 maps provided in IBC 2012/OSSC 2014:

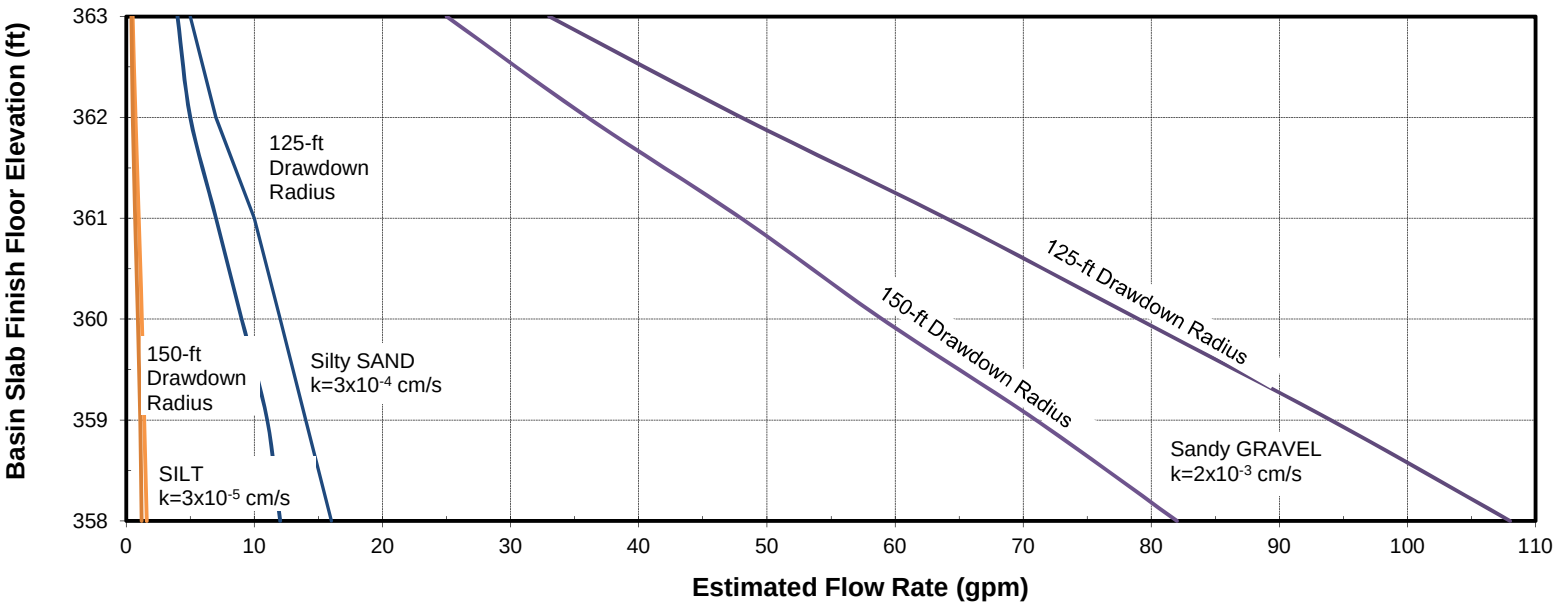
   Site Class= C Damping = 5%

S_S =	0.79	F_a =	1.09	S_{MS} =	0.85	S_{DS} =	0.57
S_1 =	0.39	F_v =	1.41	S_{M1} =	0.55	S_{D1} =	0.37

3. S_S and S_1 values indicated in Note 2 are the mapped, risk-targeted maximum considered earthquake spectral acclerations for 1% probability of exceedence in 50 years.
4. F_a and F_v were established based on IBC 2012, Tables 1613.3.3(1) and 1613.3.3(2) using the selected S_S and S_1 values. S_{DS} and S_{D1} values include a 2/3 reduction on S_{MS} and S_{M1} as discussed in IBC 2012 Section 1613.3.4.
5. Site location is: Latitude 44.5184, Longitude -122.8814.

FIGURE 9A
IBC 2012/OSSC 2014 SITE RESPONSE SPECTRUM
INTAKE STRUCTURE
Lebanon Water Treatment Plant (Intake Structure)
Lebanon, Oregon
Project 2131070-101

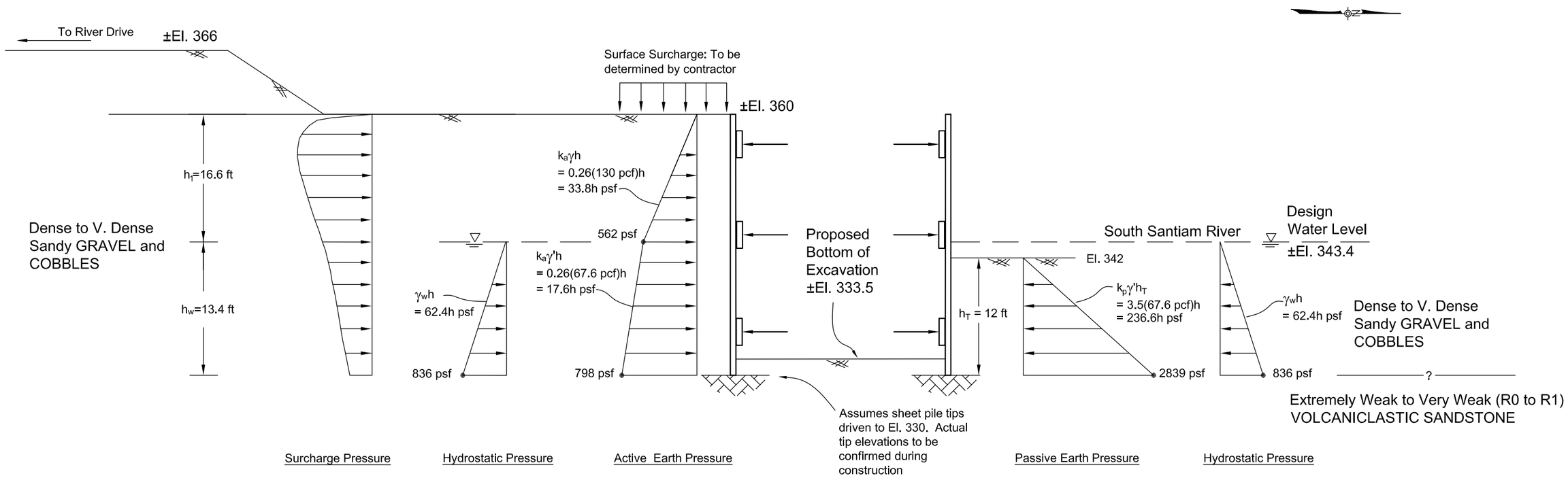
Foundation Engineering, Inc.
Lebanon Water Treatment Plant
Project 2131070-101



Notes:

1. Assumes high ground water at El. 363.0.
2. Assumes flowline of drainage system is 2 feet below the basin slab finish floor.
3. Basins (both) were modeled as a circular well with an equivalent radius of 71.6 feet.
4. Drawdown radii were assumed to range from 125 feet to 150 feet.

Figure 10A. Variation of Estimated Pumping Rates with Settling Basin Finish Floor Elevation and Soil Permeability



NOTES:

1. The earth pressure diagrams shown are based on the subsurface conditions in BH–5 and the assumed shoring layout. The diagrams should be adjusted, as needed, based on the actual shoring configuration.
2. Active earth pressure assumes the sheet piles will retain gravel alluvium and will be sufficiently flexible to mobilize active conditions. Passive pressure assumes sheet piles will be driven into gravel alluvium.
3. Surcharge pressure is based on AASHTO LRFD Bridge Design Specifications (2012) Section 3.11.3.2, p. 3–124. The shape of the surcharge pressure curve will depend on the area, location, and magnitude of the surface surcharge (to be determined by the contractor).
4. Hydrostatic pressure is based on the Design Water Level shown on the plans.
5. All units in feet, pounds, and pounds per square foot (psf).
6. See report for additional comments regarding this figure.

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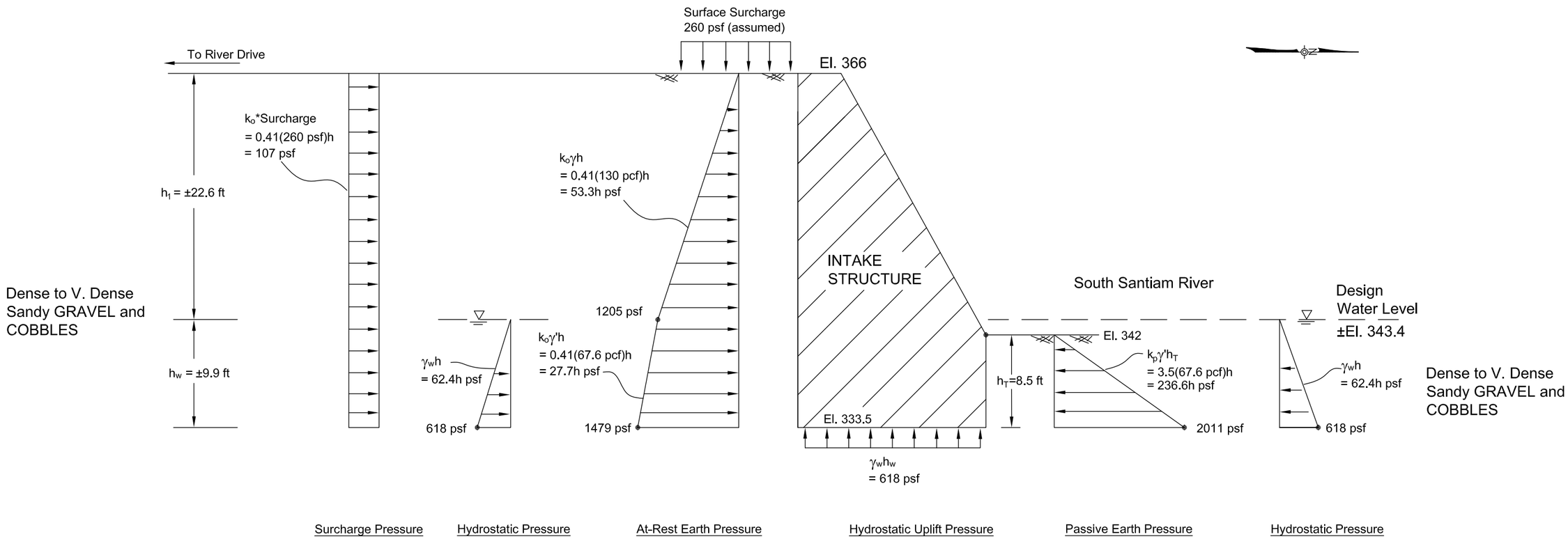
LATERAL EARTH PRESSURE DIAGRAM
INTAKE STRUCTURE - SHORING
LEBANON WATER TREATMENT PLANT
LEBANON, OREGON

FIGURE NO.
11A

FILE NAME: FIG 11A



NOTES:

1. The earth pressure diagrams shown are based on the subsurface conditions in BH–5 and preliminary plans. The diagrams should be adjusted, as needed, based on the final structure configuration.
2. At–rest earth pressure assumes the walls will be backfilled with compacted base rock. Passive earth pressure assumes intake slab will be backfilled with riprap.
3. Hydrostatic pressure is based on the Design Water Level shown on the plans.
4. All units in feet, pounds, and pounds per square foot (psf).
5. See report for additional comments regarding this figure.

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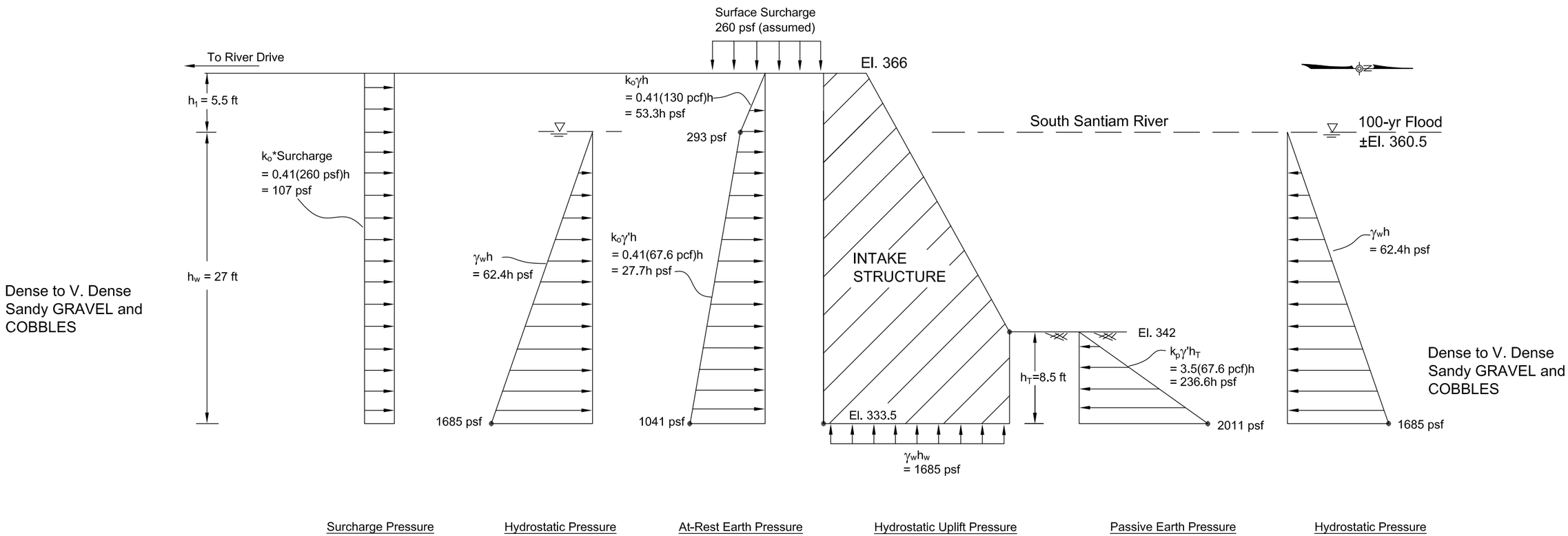
PROJECT NO.
2131070–101

LATERAL EARTH PRESSURE DIAGRAM
PERMANENT INTAKE STRUCTURE - STATIC DESIGN
LEBANON WATER TREATMENT PLANT
LEBANON, OREGON

FIGURE NO.
12A

FILE NAME: FIG 12A



NOTES:

1. The earth pressure diagrams shown are based on the subsurface conditions in BH–5 and preliminary plans. The dimensions should be adjusted, as needed, based on the final structure configuration.
2. At–rest earth pressure assumes the walls will be backfilled with compacted base rock. Passive earth pressure assumes intake slab will be backfilled with riprap.
3. Hydrostatic pressure is based on the 100–year Flood water level shown on the plans.
4. All units in feet, pounds, and pounds per square foot (psf).
5. See report for additional comments regarding this figure.

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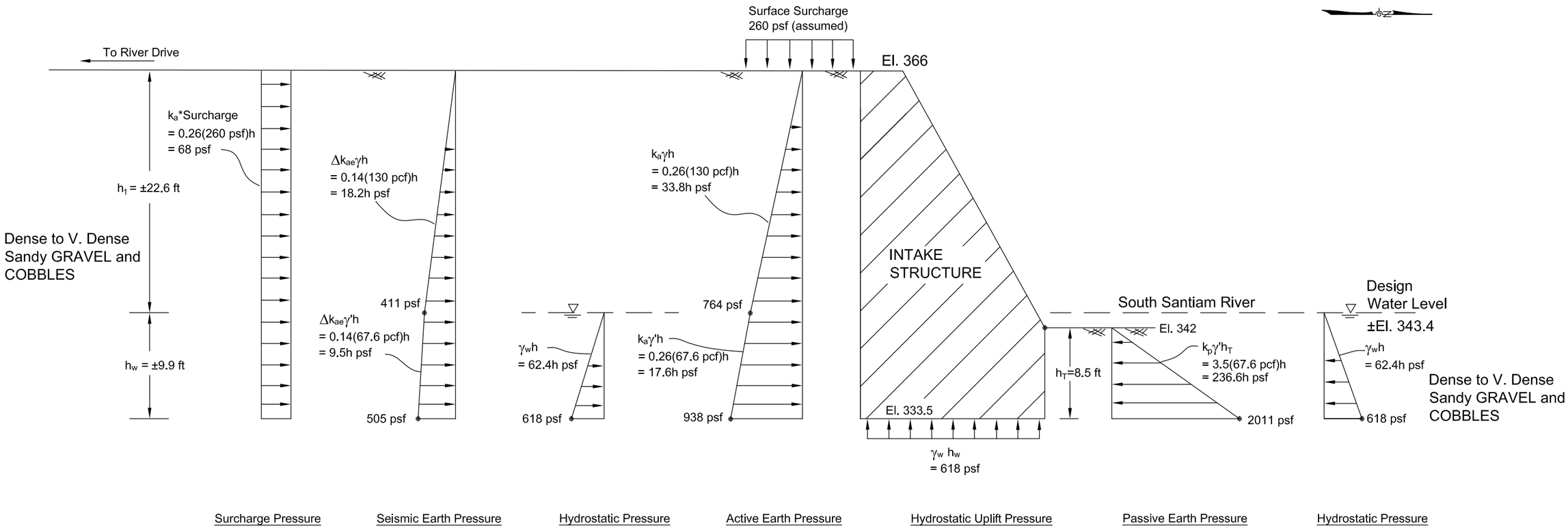
PROJECT NO.
2131070–101

LATERAL EARTH PRESSURE DIAGRAM
PERMANENT INTAKE STRUCTURE - 100-YR FLOOD
LEBANON WATER TREATMENT PLANT
LEBANON, OREGON

FIGURE NO.
13A

FILE NAME: FIG 13A



NOTES:

1. The earth pressure diagrams shown are based on the subsurface conditions in BH–5 and preliminary plans. The diagrams should be adjusted, as needed, based on the final structure configuration.
2. Active earth and seismic earth pressures assumes the walls will be backfilled with compacted base rock. Passive earth pressure assumes intake slab will be backfilled with riprap.
3. Hydrostatic pressure is based on the Design Water Level shown on the plans.
4. All units in feet, pounds, and pounds per square foot (psf).
5. See report for additional comments regarding this figure.

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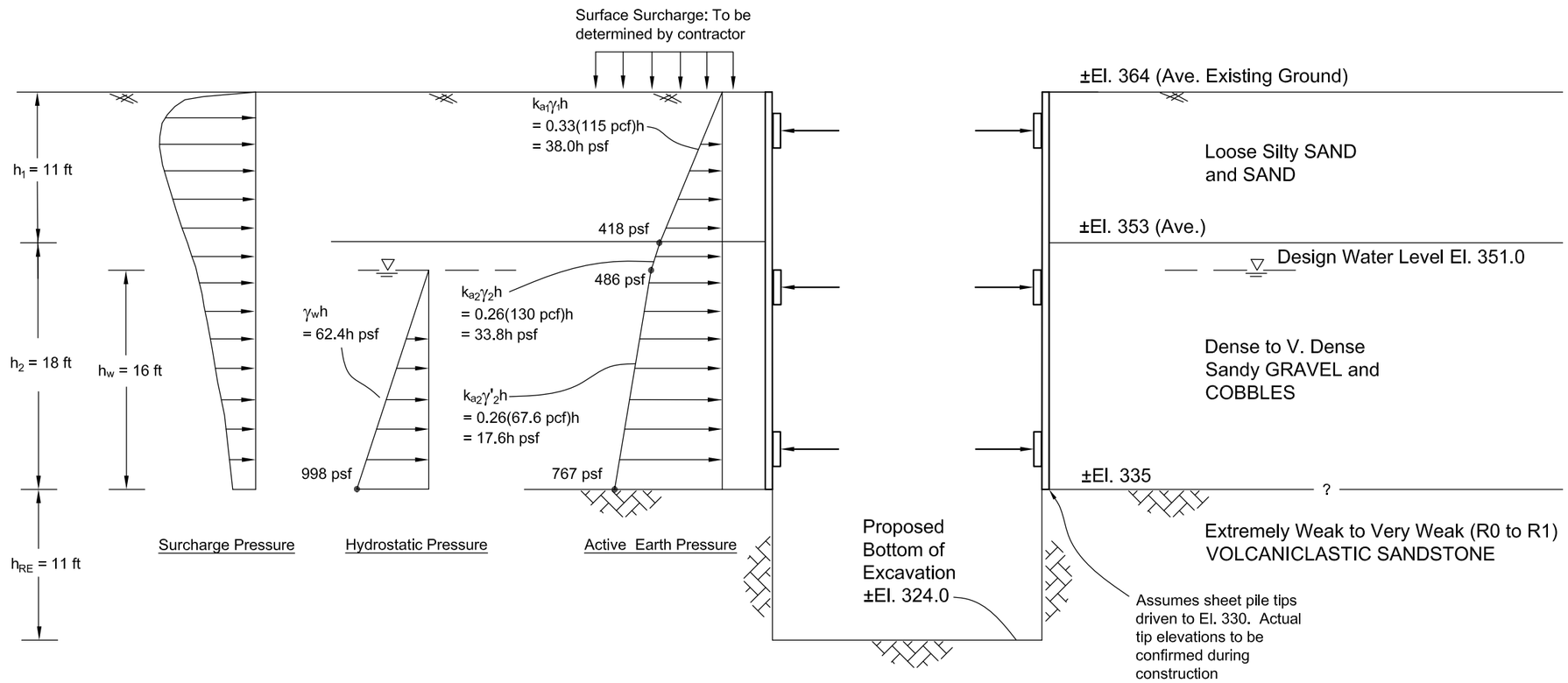
LATERAL EARTH PRESSURE DIAGRAM
PERMANENT INTAKE STRUCTURE - SEISMIC DESIGN
LEBANON WATER TREATMENT PLANT
LEBANON, OREGON

FIGURE NO.
14A

FILE NAME: FIG 14A



NOTES:

1. The earth pressure diagrams shown are based on the average subsurface conditions in BH–7 and TP–29 through TP–32 and the assumed shoring layout. The diagrams should be adjusted, as needed, based on the actual shoring configuration.
2. Active earth pressure assumes the sheet piles will retain sand and gravel alluvium and will be sufficiently flexible to mobilize active conditions.
3. Surcharge pressure is based on AASHTO LRFD Bridge Design Specifications (2012) Section 3.11.3.2, p. 3–124. The shape of the surcharge pressure curve will depend on the area, location, and magnitude of the surface surcharge (to be determined by the contractor).
4. Hydrostatic pressure is based on the anticipated high summer ground water level.
5. All units in feet, pounds, and pounds per square foot (psf).
6. See report for additional comments regarding this figure.

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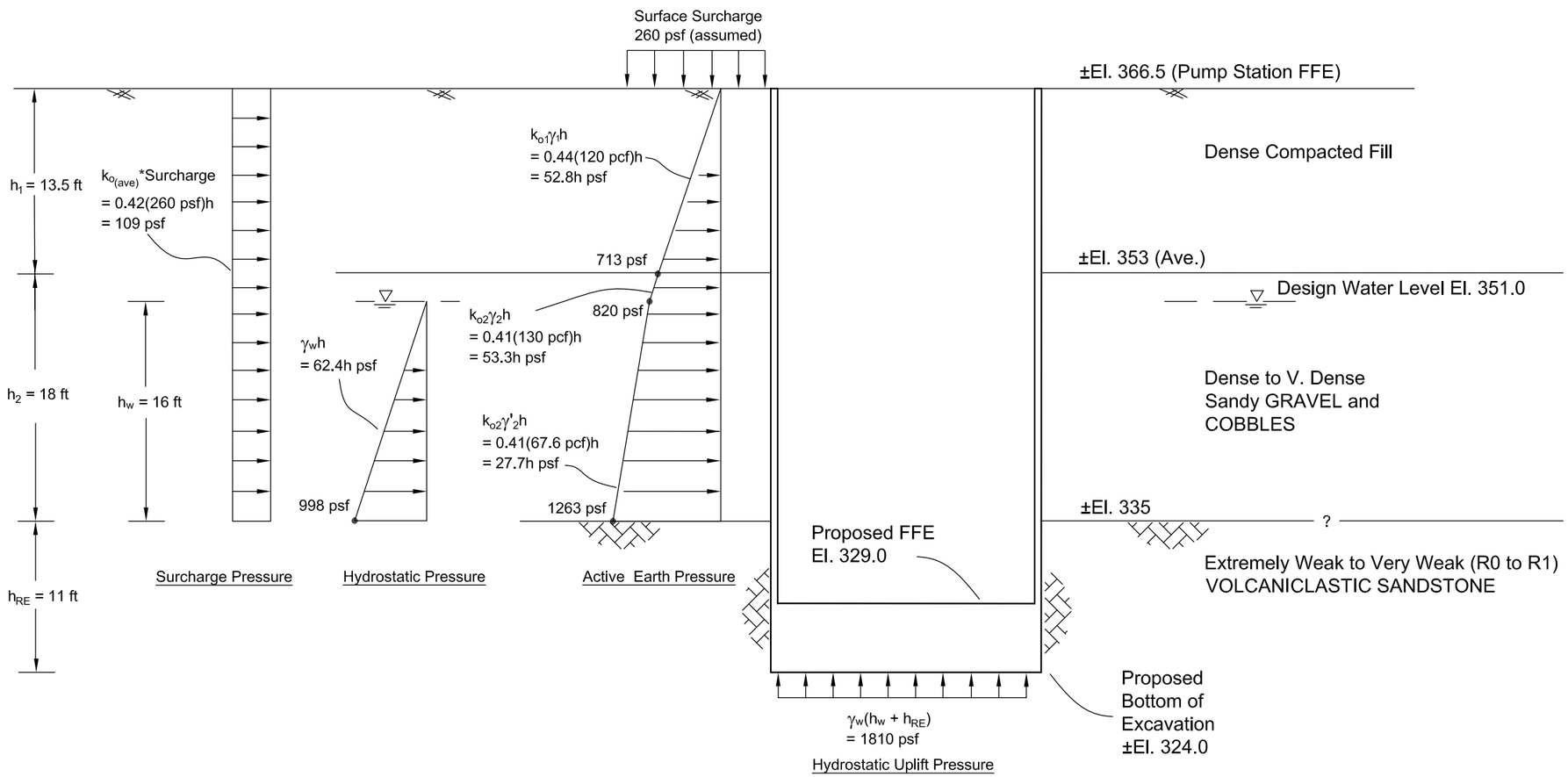
LATERAL EARTH PRESSURE DIAGRAM
WET WELL - SHORING
LEBANON WATER TREATMENT PLANT
LEBANON, OREGON

FIGURE NO. 15A

FILE NAME: FIG 15A



NOTES:

1. The earth pressure diagrams shown are based on the average subsurface conditions in BH–7 and TP–29 through TP–32 and preliminary plans. The diagrams should be adjusted, as needed, based on the final structure configuration.
2. At–rest earth pressure assumes the walls will be backfilled with compacted granular fill.
3. Hydrostatic pressure is based on the anticipated high summer ground water level.
4. All units in feet, pounds, and pounds per square foot (psf).
5. See report for additional comments regarding this figure.

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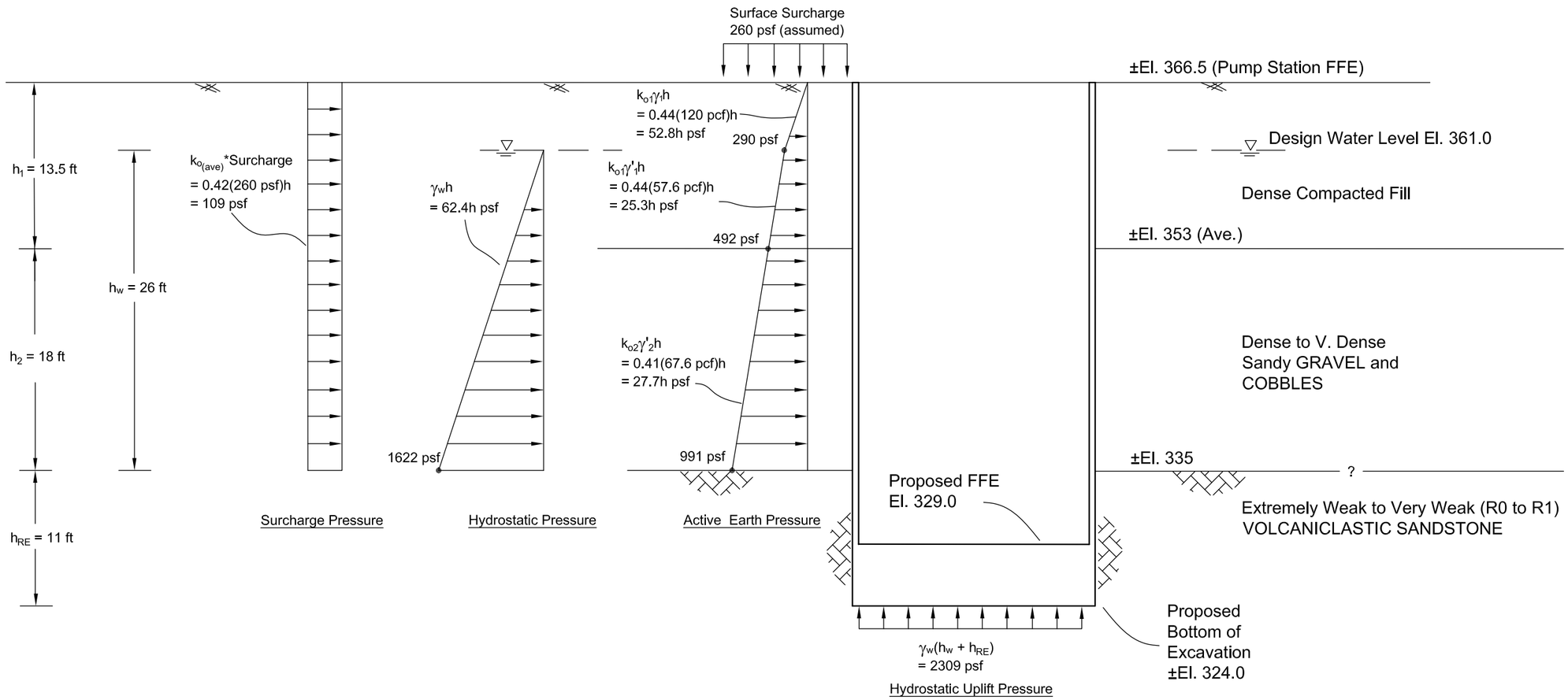
LATERAL EARTH PRESSURE DIAGRAM
PERMANENT WET WELL - STATIC DESIGN
LEBANON WATER TREATMENT PLANT
LEBANON, OREGON

FIGURE NO.
16A

FILE NAME: FIG 16A



NOTES:

1. The earth pressure diagrams shown are based on the average subsurface conditions in BH–7 and TP–29 through TP–32 and preliminary plans. The dimensions should be adjusted, as needed, based on the final structure configuration.
2. At–rest earth pressure assumes the walls will be backfilled with compacted granular fill.
3. Hydrostatic pressure is the anticipated ground water level corresponding to the based on the 100–year Flood water level shown on the plans.
4. All units in feet, pounds, and pounds per square foot (psf).
5. See report for additional comments regarding this figure.

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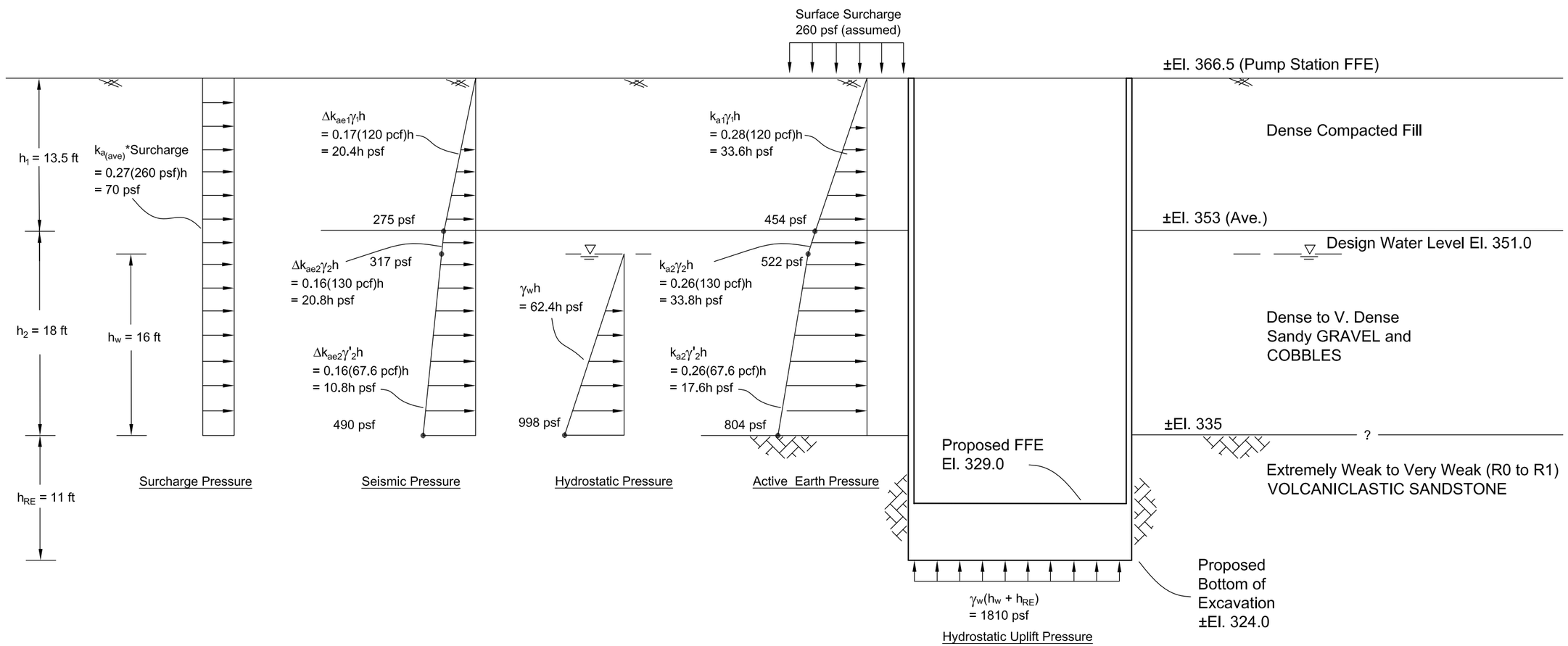
LATERAL EARTH PRESSURE DIAGRAM
PERMANENT WET WELL - 100-YR FLOOD
LEBANON WATER TREATMENT PLANT
LEBANON, OREGON

FIGURE NO.
17A

FILE NAME: FIG 17A



NOTES:

1. The earth pressure diagrams shown are based on the average subsurface conditions in BH–7 and TP–29 through TP–32 and preliminary plans. The diagrams should be adjusted, as needed, based on the final structure configuration.
2. Active earth and seismic earth pressures assumes the walls will be backfilled with compacted granular fill.
3. Hydrostatic pressure is based on the anticipated high summer ground water level.
4. All units in feet, pounds, and pounds per square foot (psf).
5. See report for additional comments regarding this figure.

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LATERAL EARTH PRESSURE DIAGRAM
PERMANENT WET WELL - SEISMIC DESIGN
LEBANON WATER TREATMENT PLANT
LEBANON, OREGON

FIGURE NO.
18A

FILE NAME: FIG 18A

Appendix B

Boring and Test Pit Logs

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DISTINCTION BETWEEN FIELD LOGS AND FINAL LOGS

A field log is prepared for each boring or test pit by our field representative. The log contains information concerning sampling depths and the presence of various materials such as gravel, cobbles, and fill, and observations of ground water. It also contains our interpretation of the soil conditions between samples. The final logs presented in this report represent our interpretation of the contents of the field logs and the results of the laboratory examinations and tests. Our recommendations are based on the contents of the final logs and the information contained therein and not on the field logs.

VARIATION IN SOILS BETWEEN TEST PITS AND BORINGS

The final log and related information depict subsurface conditions only at the specific location and on the date indicated. Those using the information contained herein should be aware that soil conditions at other locations or on other dates may differ. Actual foundation or subgrade conditions should be confirmed by us during construction.

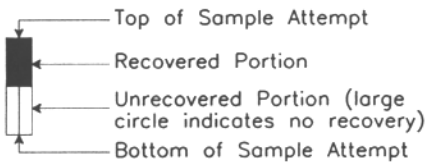
TRANSITION BETWEEN SOIL OR ROCK TYPES

The lines designating the interface between soil, fill or rock on the final logs and on subsurface profiles presented in the report are determined by interpolation and are therefore approximate. The transition between the materials may be abrupt or gradual. Only at boring or test pit locations should profiles be considered as reasonably accurate and then only to the degree implied by the notes thereon.

SAMPLE OR TEST SYMBOLS

SH-3-4

- Sample Number
- Boring or Test Pit Number
- Sample Type



S – Grab Samples
SS – Standard Penetration Test Sample (split-spoon)
SH – Thin-walled Shelby Tube Sample
C – Core Sample
CS – Continuous Sample

▲ Standard Penetration Test Resistance equals the number of blows a 140 lb. weight falling 30 in. is required to drive a standard split-spoon sampler 1 ft. Practical refusal is equal to 50 or more blows per 6 in. of sampler penetration.

● Water Content (%).

UNIFIED SOIL CLASSIFICATION SYMBOLS

G – Gravel	W – Well Graded
S – Sand	P – Poorly Graded
M – Silt	L – Low Plasticity
C – Clay	H – High Plasticity
Pt – Peat	O – Organic

FIELD SHEAR STRENGTH TEST

Shear strength measurements on test pit side walls, blocks of soil or Shelby tube samples are typically made with Torvane or pocket penetrometer devices.

TYPICAL SOIL/ROCK SYMBOLS

Sand	Silt
Clay	Gravel
Basalt	Siltstone

WATER TABLE

▼ Water Table Location

(1/31/00) Date of Measurement

Piezometer Tip Location (if used)

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SYMBOL KEY
BORING AND TEST PIT LOGS

Explanation of Common Terms Used in Rock Descriptions

Field Identification		UCS (psi)	UCS (MPa)	Strength (Hardness)
Indented by thumbnail.	R0	< 100	0.25–1.0	Extremely Weak (Extremely Soft)
Crumbles under firm blows with geological hammer, can be peeled by a pocket knife.	R1	100–1000	1.0–5.0	Very Weak (Very Soft)
Can be peeled by a pocket knife with difficulty, shallow indentations made by firm blow with geological hammer.	R2	1000–4000	5.0–25	Weak (Soft)
Cannot be scraped or peeled with a pocket knife, specimen can be fractured with a single blow of geological hammer.	R3	4000–8000	25–50	Medium Strong (Medium Hard)
Specimen requires more than one blow of geological hammer to fracture it.	R4	8000–16000	50–100	Strong (Hard)
Specimen requires many blows of geological hammer to fracture it.	R5	16000–36000	100–250	Very Strong (Very Hard)
Specimen can only be chipped with geological hammer.	R6	> 36000	> 250	Extremely Strong (Extremely Hard)

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

Spacing (meters)	Spacing (feet)	Spacing Term	Bedding/Foliation
< 0.06	< 2 in.	Very Close	Very Thin
0.06 – 0.30	2 in. – 1 ft.	Close	Thin
0.30 – 0.90	1 ft. – 3 ft.	Moderately Close	Medium
0.90 – 3.0	3 ft. – 10 ft.	Wide	Thick
> 3.0	> 10 ft.	Very Wide	Very Thick (Massive)

Vesicle Term	Volume
Some	3 – 20%
Highly	20 – 50%
Scoria	> 50%

Stratification Term	Description
Lamination	<0.39 in. thick beds
Fissile	Preferred break along laminations
Parting	Preferred break direction
Foliation	Metamorphic layering of minerals

RQD %	Designation	RQD %	Designation
0 – 25	Very Poor	75 – 90	Good
25 – 50	Poor	90 – 100	Excellent
50 – 75	Fair		

Rock Quality Designation (RQD) is the percent of a core run with intact lengths greater than 4.0 in. excluding breaks caused by drilling.

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COMMON TERMS
ROCK DESCRIPTIONS

Explanation of Common Terms Used in Soil Descriptions

Field Identification	Cohesive Soils			Granular Soils	
	SPT	S_u* (tsf)	Term	SPT	Term
Easily penetrated several inches by fist.	0 – 1	< 0.125	Very Soft	0 – 4	Very Loose
Easily penetrated several inches by thumb.	2 – 4	0.125–0.25	Soft	5 – 10	Loose
Can be penetrated several inches by thumb with moderate effort.	5 – 8	0.25 – 0.50	Medium Stiff (Firm)	11 – 30	Medium Dense
Readily indented by thumb but penetrated only with great effort.	9 – 15	0.50 – 1.0	Stiff	31 – 50	Dense
Readily indented by thumbnail.	16 – 30	1.0 – 2.0	Very Stiff	> 50	Very Dense
Indented with difficulty by thumbnail.	31 – 60	> 2.0	Hard		

* Undrained shear strength

Term	Soil Moisture Field Description
Dry	Absence of moisture. Dusty. Dry to the touch.
Damp	Soil has moisture. Cohesive soils are below plastic limit and usually moldable.
Moist	Grains appear darkened, but no visible water. Silt/clay will clump. Sand will bulk. Soils are often at or near plastic limit.
Wet	Visible water on larger grain surfaces. Sand and cohesionless silt exhibit dilatancy. Cohesive silt/clay can be readily remolded. Soil leaves wetness on the hand when squeezed. "Wet" indicates that the soil is wetter than the optimum moisture content and above the plastic limit.

Term	PI	Plasticity Field Test
Nonplastic	0 – 3	Cannot be rolled into a thread.
Low Plasticity	3 – 15	Can be rolled into a thread with some difficulty.
Medium Plasticity	15 – 30	Easily rolled into thread.
High Plasticity	> 30	Easily rolled and rerolled into thread.

Term	Soil Structure Criteria
Stratified	Alternating layers at least 1 inch thick – describe variation.
Laminated	Alternating layers at less than 1 inch thick – describe variation.
Fissured	Contains shears and partings along planes of weakness.
Slickensides	Partings appear glossy or striated.
Blocky	Breaks into lumps – crumbly.
Lensed	Contains pockets of different soils – describe variation.

Term	Soil Cementation Criteria
Weak	Breaks under light finger pressure.
Moderate	Breaks under hard finger pressure.
Strong	Will not break with finger pressure.

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COMMON TERMS
SOIL DESCRIPTIONS

Table 1B. Fill/Topsoil Thickness and Depth to Alluvial Gravel Stratum (Water Treatment Plant Site)

Location	Ground Elevation (feet)	Fill/Topsoil Thickness (feet)	Bottom of Topsoil El. (feet)	Depth to Gravel (feet)	Top of Gravel El. (feet)
TP-1	363.8	1.2	362.6	10	353.8
TP-2	364.6	1.4	363.2	4.4	360.2
TP-3	363.5	3.7	359.8	12	351.5
TP-4	362.9	1	361.9	7.5	355.4
TP-5	361.9	1.6	360.3	1.6	360.3
TP-6	364.4	1.2	363.2	2.2	362.2
TP-7	364.7	2.3	362.4	4.8	359.9
TP-8	364.8	0.7	364.1	4.4	360.4
TP-9	363.9	3.4	360.5	12.5	351.4
TP-10	363.1	1.5	361.6	7.5	355.6
TP-11	362.4	1.5	360.9	1.5	360.9
TP-12	364.1	0.7	363.4	6.5	357.6
TP-13	364.4	1.1	363.3	4.2	360.2
TP-14	364.3	1	363.3	3	361.3
TP-15	363.4	3.5	359.9	9	354.4
TP-16	362.9	1.2	361.7	9	353.9
TP-17	363.5	2	361.5	8.5	355
TP-18	363	1	362	4	359
TP-19	362.5	1.8	360.7	1.7	360.8
TP-20	364.2	2.2	362	8.5	355.7
TP-21	363.6	2.8	360.8	9.5	354.1
TP-22	364.4	3.1	361.3	9.5	354.9
TP-23	363.4	1.5	361.9	8.5	354.9
TP-24	362.8	2	360.8	8	354.8
TP-25	362.7	2	360.7	2	360.7
TP-26	363.8	1.2	362.6	5.2	358.6
TP-27	363.1	1.2	361.9	5	358.1
TP-28	364.3	1.5	362.8	5	359.3
P-5	363.5	2.3	361.2	2.3	361.2
P-6	365	1.5	363.5	5.3	359.7
P-7	365	1.3	363.7	4.5	360.5
BH-1	365.1	3	362.1	4	361.1
BH-2	362.8	1.5	361.3	1.5	361.3
BH-3	363.5	2.5	361	11	352.5
BH-4	362.4	2.5	359.9	2.5	359.9

Table 2B. Topsoil Thickness and Depth to Alluvial Gravel Stratum (Raw Water Pump Station Sites)

Location	Ground Elevation (feet)	Topsoil Thickness (feet)	Bottom of Topsoil El. (feet)	Depth to Gravel (feet)	Top of Gravel El. (feet)
TP-29	360.2	1	359.2	8	352.2
TP-30	364.3	0.6	363.7	9.5	354.8
TP-31	361.7	0.8	360.9	9	352.7
TP-32	364.9	0.7	364.2	10.5	354.4
P-8	362.0	0.5	361.5	11.0	351
P-9	362.0	0.5	361.5	n/a	n/a
P-10	362.0	0.5	361.5	n/a	n/a
BH-7	363.1	n/a	n/a	10	353.1

Note: Topsoil thickness in BH-7 could not be determined due to sampling interval.

Table 3B. Topsoil Thickness and Depth to Alluvial Gravel Stratum (Intake and River Drive Undercrossing Sites)

Location	Ground Elevation (feet)	Topsoil Thickness (feet)	Bottom of Topsoil El. (feet)	Depth to Gravel (feet)	Top of Gravel El. (feet)
BH-5	362.2	n/a	n/a	6	356.2
BH-6	367.7	n/a	n/a	10	357.7

Depth Feet	Soil and Rock Description and Comments	Log	Elev. Depth 365.05	Samples	SPT, N-Value / Recovery / Moisture, % / RQD., % (0 – 50 – 100)	Installations/ Water Table
1	Hard, sandy SILT, (ML); brown, moist, low plasticity silt, fine sand, (topsoil/alluvium).		0.0			Morris monument set in concrete
2						
3	Field vane on SH-1-1: S_u: >1.0 tsf at ±2.5 feet.		362.1	SH-1-1		
4	Medium dense silty SAND; brown, moist, low plasticity silt, fine sand, (alluvium).		3.0 361.1 4.0	SS-1-2	44	Bentonite chips
5	Medium dense to dense sandy GRAVEL to gravelly SAND, trace to some silt, (GP to SP); grey-brown, iron-stained, moist, low plasticity silt, fine to coarse sand, fine to coarse, subrounded to rounded gravel, (alluvium).					±1-inch I.D. PVC
6						
7						Ground water (11/7/13)
8	Wet below ±7 feet.			SS-1-3	29	
9						Colorado silica sand
10				SS-1-4	21	0.010 machine slot screen
11						
12						
13				SS-1-5	27	
14						
15	Dense sandy GRAVEL, trace silt, (GP); grey-brown, wet, low plasticity silt, fine to coarse sand, fine to coarse, subrounded to rounded gravel, (alluvium).		350.1 15.0	SS-1-6	41	
16						
17	Very dense below ±17 feet.					
18				SS-1-7	50/3"	
19						
20				SS-1-8	64	Bentonite chips
21						
22	Hard silty CLAY, some sand and gravel; brown, wet, low to medium plasticity, fine to coarse sand, fine to coarse, subrounded to rounded gravel, (alluvium).		343.1 22.0			
23				SS-1-9	41	
24	Very stiff sandy SILT; brown, iron-stained, moist to wet, low plasticity, fine sand, (alluvium)		341.6 23.5			
25				SS-1-10	29	
26						
27						
28						
29	Dense sandy GRAVEL, some silt; grey-brown, wet, low plasticity silt, fine to coarse sand, fine to coarse, subrounded to rounded gravel, (alluvium).		336.1 29.0			
30				SS-1-11	33	
31			333.6 31.5			
	BOTTOM OF BORING					

Project No.: 2131070

Surface Elevation: 365.1 feet (Approx.)

Date of Boring: November 6, 2013

Foundation Engineering, Inc.

Boring Log: BH-1

Water Treatment Plant

Lebanon Water Treatment Plant

Lebanon, Oregon

Depth Feet	Soil and Rock Description and Comments	Elev. Depth (362.82)	Samples	SPT, N-Value / Recovery (0–50)	Moisture, % / RQD., % (50–100)	Installations/ Water Table
1	Medium stiff to stiff SILT; brown, moist, low to medium plasticity, (topsoil/fill).	0.0 / 361.3				Morris monument set in concrete
2–3	Dense, sandy GRAVEL, trace silt; grey-brown, moist, fine to coarse sand, fine to coarse, subrounded to rounded gravel, (alluvium).	1.5	SS-2-1	43		Bentonite chips
4–5	Very dense below ±5 feet.		SS-2-2	50/4"		±1-inch I.D. PVC
6–9	Very dense sandy GRAVEL and COBBLES, trace silt; grey-brown, moist, low plasticity silt, fine to coarse sand, fine to coarse, subrounded to rounded gravel and cobbles, cobbles up to ±6 inches in diameter, (alluvium).	356.8 / 6.0				Ground water (11/7/13)
10–14	Wet below ±10 feet.		SS-2-3	77		Colorado silica sand
15–19			SS-2-4	70		0.010 machine slot screen
20–21	Dense below ±20 feet.		SS-2-5	37		
22–24	Dense sandy, silty GRAVEL to sandy GRAVEL, some silt; grey-brown, wet, low plasticity silt, fine to coarse sand, fine to coarse, subrounded to rounded gravel, (alluvium).	340.8 / 22.0				Bentonite chips
25–26			SS-2-6	42		
27–30	Very dense below ±30 feet.		SS-2-7	55		
31	BOTTOM OF BORING	331.3 / 31.5				

Project No.: 2131070

Surface Elevation: 362.8 feet (Approx.)

Date of Boring: November 6, 2013

Foundation Engineering, Inc.

Boring Log: BH-2

Water Treatment Plant

Lebanon Water Treatment Plant

Lebanon, Oregon

Depth Feet	Soil and Rock Description and Comments	Log	Elev. Depth 363.5	Samples	SPT, N-Value / Recovery / Moisture, % / RQD., % (0 – 50 – 100)	Installations/ Water Table
1–3	Medium stiff SILT, trace to some sand and gravel, scattered organics (ML); brown, damp, low plasticity, fine to coarse sand, fine gravel, subangular to rounded gravel, organics consist of fine roots and wood debris, (topsoil/fill).		0.0			Backfilled with bentonite chips
4–8	Stiff clayey SILT (MH); grey, moist, medium plasticity, (alluvium). Field vane on SH-3-1: S_u = ±1 tsf at ±4.5 feet. Field vane on SH-3-2: S_u = ±0.9 tsf at ±5.5 feet.		360.0 / 3.5	SH-3-1, SH-3-2, SS-3-3	SS-3-3: 3	
9–11	Soft to medium stiff sandy SILT (ML); grey-brown, wet, medium plasticity, fine sand, (alluvium).		355.5 / 8.0	SS-3-4	SS-3-4: 56/11"	
12–19	Very dense sandy GRAVEL, trace silt, scattered cobbles (GP); grey-brown, wet, fine to coarse sand, fine to coarse gravel, subrounded to rounded gravel and cobbles, cobbles up to ±6 inches in diameter, (alluvium).		352.5 / 11.0	SS-3-5, SS-3-6	SS-3-5: 81; SS-3-6: 69	
20–24	Very stiff silty CLAY (CL); brown, damp, low plasticity, (alluvium).		343.5 / 20.0	SS-3-7	SS-3-7: 25	
25–31	Dense sandy GRAVEL, some silt (GP); grey-brown, moist, low plasticity silt, fine to coarse sand, fine to coarse gravel, subrounded to rounded gravel, (alluvium).		338.5 / 25.0	SS-3-8, SS-3-9	SS-3-8: 30; SS-3-9: 75	
32–36	Medium dense silty SAND (SM); brown, wet, low plasticity silt, fine sand, (alluvium).		331.5 / 32.0	SS-3-10	SS-3-10: 21	
	Dense sandy GRAVEL, some silt (GP); grey-brown, wet, low plasticity silt, fine to coarse sand, fine to coarse gravel, subrounded to rounded gravel, (alluvium).		327.3 / 36.3			
	BOTTOM OF BORING		327.0 / 36.5			

Project No.: 2131070-101

Surface Elevation: 363.5 feet (Approx.)

Date of Boring: September 28, 2015

Boring Log: BH- 3

Water Treatment Plant

Lebanon Water Treatment Plant

Lebanon, Oregon

Foundation Engineering, Inc.

Depth Feet	Soil and Rock Description and Comments	Log	Elev. Depth 362.4	Samples	SPT, N-Value / Recovery (0–50)	Moisture, % / RQD., % (50–100)	Installations/ Water Table
1	Stiff SILT (ML); brown, dry to damp, low to medium plasticity, (topsoil/fill).		0.0				Backfilled with bentonite chips
2			359.9				
3	Very dense sandy GRAVEL, trace silt, scattered cobbles (GP); grey-brown, damp, fine to coarse sand, fine to coarse gravel, subrounded to rounded gravel and cobbles, cobbles up to ±6 inches in diameter, (alluvium).		2.5	SH-4-1		85	
4				SS-4-2			
5				SS-4-3		50/1st 5"	
6							
7							
8	Wet below ±7.5 feet.			SS-4-4		50/1st 5"	
9							
10	Dense at ±10 feet.			SS-4-5	37		
11							
12							
13							
14							
15				SS-4-6		50/1st 5"	
16							
17							
18			343.9				
19	Medium dense silty SAND (SM); brown, wet, low to medium plasticity silt, fine sand, (alluvium).		18.5				
20				SS-4-7	11		
21							
22							
23	Very thin scattered silt lenses (<2 inches thick) and fine to medium sand below ±23.5 feet).						
24				SS-4-8	29	●	
25	Very dense silty GRAVEL, some sand (GM); grey-brown, moist, low plasticity silt, fine to coarse sand, fine to coarse gravel, subrounded to rounded gravel, (alluvium).		337.7 24.8	SS-4-9		50/5"	
26							
27							
28							
29							
30				SS-4-10		67	
31							
32							
33							
34							
35				SS-4-11		81	
36							
37							
38							
39							
40	Thin sand lens (±6 inches thick) at ±40 feet.			SS-4-12		68	
41							

Project No.: 2131070-101

Surface Elevation: 362.4 feet (Approx.)

Date of Boring: September 28, 2015

Boring Log: BH- 4

Water Treatment Plant

Lebanon Water Treatment Plant

Lebanon, Oregon

Foundation Engineering, Inc.

Depth Feet	Soil and Rock Description and Comments	Log	Elev. Depth 320.4	Samples	▲ SPT, N-Value ● Moisture, % ⊡ Recovery ⊞ RQD., % (0 – 50 – 100)	Installations/ Water Table
43						
44						
45				SS-4-13	64	
46						
47						
48						
49						
50				SS-4-14	95	
51						
52						
53						
54						
55				SS-4-15	50/5"	
56						
57						
58						
59						
60				SS-4-16	50/5"	
	BOTTOM OF BORING		301.5 60.9			

Project No.: 2131070-101

Surface Elevation: 362.4 feet (Approx.)

Date of Boring: September 28, 2015

Boring Log: BH- 4

Water Treatment Plant

Lebanon Water Treatment Plant

Lebanon, Oregon

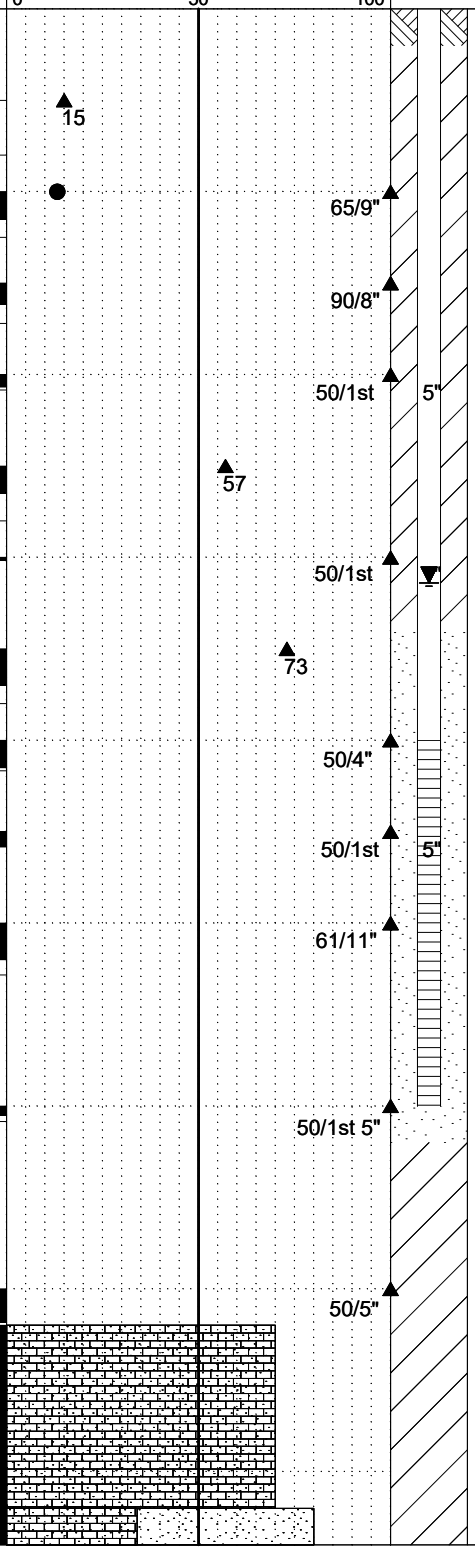
Foundation Engineering, Inc.

Depth Feet	Soil and Rock Description and Comments	Elev. Depth (362.2)
1–2	Medium dense silty GRAVEL AND COBBLES (GM); brown, damp, fine to coarse gravel, subrounded to rounded gravel, subrounded cobbles up to ±5 inches in diameter, (fill).	0.0
3–5	Medium dense silty SAND, scattered organics (SM); brown, damp, low plasticity silt, fine sand, organics consist of tree roots, (alluvium).	359.2 / 3.0
6–10	Very dense sandy GRAVEL, trace silt (GP); grey-brown, dry, fine to coarse sand, fine to coarse gravel, subrounded to rounded gravel, (alluvium). Scattered cobbles (up to ±6 inches in diameter) from ±7.5 to 32 feet.	356.2 / 6.0
10–11	Damp below ±10 feet.	
13–14	Moist below ±12.5 feet.	
15–16	Wet below ±15 feet.	
25–26	Thin sand lens (±8 inches thick) at ±25.3 feet.	
32–37	Extremely weak (R0) VOLCANICLASTIC SANDSTONE; grey, highly weathered, fine to coarse volcanic sand, (Fisher Formation).	330.2 / 32.0
38–41	Very weak (R1) VOLCANICLASTIC SANDSTONE; grey, moderately weathered, very close to close joints, joints are irregular, rough, open, fine to coarse volcanic sand, (Fisher Formation).	324.7 / 37.5

Log

Samples: SS-5-1, SS-5-2, SS-5-3, SS-5-4, SS-5-5, SS-5-6, SS-5-7, SS-5-8, SS-5-9, SS-5-10, SS-5-11, SS-5-12, CS-5-1, CS-5-2

SPT, N-Value ▲ | Recovery ⬚ | Moisture, % ● | RQD., % ⊞ (0 – 50 – 100)



Installations/ Water Table: Morris monument set in concrete; Bentonite chips; ±1-inch I.D. PVC; Ground water (9/30/15); Colorado silica sand; 0.010 machine slot screen; Bentonite chips

Project No.: 2131070-101

Surface Elevation: 362.2 feet (Approx.)

Date of Boring: September 24, 2015

Boring Log: BH- 5

Intake Structure

Lebanon Water Treatment Plant

Lebanon, Oregon

Foundation Engineering, Inc.

Depth Feet	Soil and Rock Description and Comments	Log	Elev. Depth	Samples	▲ SPT, N-Value ● Moisture, % ⊡ Recovery ⊞ RQD., % (0 – 50 – 100)	Installations/ Water Table
			320.2			
43	Thin and very weak (R1) light brown, moderately weathered tuffaceous claystone interbed from ±41.4 to 42 feet.					
44						
45						
46				CS-5-3		
47						
48						
49	Highly weathered from ±48.5 to 49.6 feet.					
50						
51				CS-5-4		
52	Thin and very weak (R1) light brown, moderately weathered tuffaceous claystone interbed from ±51.3 to 51.8 feet.					
53						
54						
55						
56			306.2			
	BOTTOM OF BORING		56.0			

Project No.: 2131070-101

Surface Elevation: 362.2 feet (Approx.)

Date of Boring: September 24, 2015

Boring Log: BH- 5

Intake Structure

Lebanon Water Treatment Plant

Lebanon, Oregon

Foundation Engineering, Inc.

Depth Feet	Soil and Rock Description and Comments	Log	Elev. Depth 367.7	Samples	SPT, N-Value / Recovery / Moisture, % / RQD., % (0 – 50 – 100)	Installations/ Water Table
	ASPHALTIC CONCRETE (±7 inches).		367.1			Capped with AC cold patch and gravel
1	Dense sandy GRAVEL (GP); grey, damp, fine to coarse sand, fine to coarse gravel, subrounded to rounded gravel, (base rock).		0.6 / 366.1			
2			1.6			
3	Loose silty SAND (SP); brown, damp, low plasticity silt, fine sand, (alluvium).			SS-6-1	7	Backfilled with bentonite chips
4						
5				SS-6-2	6	
6						
7						
8	Trace gravel and scattered organics below ±7.5 feet.			SS-6-3	5	
9						
10	Very dense sandy GRAVEL, trace silt (GP); grey-brown, moist, fine to coarse sand, fine to coarse gravel, subrounded to rounded gravel, (alluvium).		357.7 / 10.0	SS-6-4	54	
11						
12						
13	Scattered cobbles (up to ±6 inches in diameter) from ±12.5 to 35 feet.			SS-6-5	50/1st 5½"	
14						
15	Wet below ±15 feet.			SS-6-6	50/4"	
16						
17						
18				SS-6-7	50/4"	
19						
20	Lost ±300 gallons of drilling fluid from ±20 to 30 feet.			SS-6-8	29	
21						
22						
23						
24						
25				SS-6-9	50/1st 5"	
26						
27						
28						
29						
30				SS-6-10	66	
31						
32						
33						
34						
35	Thin silt lens (±12 inches thick) at ±35 feet.			SS-6-11	27	
36			331.2 / 36.5			
	BOTTOM OF BORING					

Project No.: 2131070-101

Surface Elevation: 367.7 feet (Approx.)

Date of Boring: September 21, 2015

Foundation Engineering, Inc.

Boring Log: BH- 6

Raw Water Pipeline

Lebanon Water Treatment Plant

Lebanon, Oregon

Depth Feet	Soil and Rock Description and Comments	Log	Elev. Depth	Samples	SPT, N-Value / Recovery / Moisture, % / RQD., % (0–50–100)	Installations/ Water Table
			363.1			
1–3	Medium stiff sandy SILT (ML); brown, dry to damp, low plasticity, fine sand, (alluvium).		0.0	SS-7-1	5	Morris monument set in concrete
			359.6			
4–9	Loose SAND, some silt (SP); brown, moist, low plasticity silt, fine to medium sand, (alluvium).		3.5	SS-7-2 SS-7-3	6 5	Bentonite chips ±1-inch I.D. PVC
			353.1			
10–27	Very dense sandy GRAVEL, trace silt, scattered cobbles (GP); grey-brown, wet, fine to coarse sand, fine to coarse gravel, subrounded to rounded gravel and cobbles, cobbles up to ±6 inches in diameter, (alluvium). Lost ±200 gallons of drilling fluid from ±10 to 20 feet.		10.0	SS-7-4 SS-7-5 SS-7-6 SS-7-7 SS-7-8 SS-7-9	90/7" 71 68 71 50/1st 5" 50/5"	Ground water (10/01/15) Colorado silica sand 0.010 machine slot screen
			335.1			
28–40	Extremely weak to very weak (R0 to R1) VOLCANICLASTIC SANDSTONE; grey, highly weathered, fine to coarse sand, (Fisher Formation).		28.0	SS-7-10 SS-7-11 SS-7-12	50/1st 5" 50/1st 5" 50/1st 4"	Bentonite chips
	BOTTOM OF BORING		322.8 40.3			

Project No.: 2131070-101

Surface Elevation: 363.1 feet (Approx.)

Date of Boring: September 25, 2015

Boring Log: BH- 7

Pump Station

Lebanon Water Treatment Plant

Lebanon, Oregon

Foundation Engineering, Inc.

Depth Feet	Soil and Rock Description and Comments	Log	Elev. Depth 365.7	Samples	▲ SPT, N-Value ● Moisture, % ⊡ Recovery ⊞ RQD., % (0 – 50 – 100)	Installations/ Water Table
1 2	Medium dense sandy GRAVEL AND COBBLES (GP); grey-brown, damp, fine to coarse sand, fine to coarse gravel, subrounded to rounded gravel and cobbles, cobbles up to ±5 inches in diameter, (fill).		0.0			Capped with gravel
3 4	Loose silty SAND (SM); brown, damp, low plasticity silt, fine sand, (alluvium).		363.2 2.5	SS-8-1	▲ 4 ●	Backfilled with bentonite chips
5 6 7	Trace gravel and scattered organics below ±5 feet.			SS-8-2	▲ 6 ●	
8 9 10	Medium dense sandy GRAVEL, trace silt (GP); grey-brown, dry to damp, fine to coarse sand, fine to coarse gravel, subrounded to rounded gravel, (alluvium). Very dense below ±10 feet.		358.2 7.5	SS-8-3	▲ 20	
			354.8	SS-8-4	▲ 50/5"	
	BOTTOM OF BORING		10.9			

Project No.: 2131070-101

Surface Elevation: 365.7 feet (Approx.)

Date of Boring: September 21, 2015

Foundation Engineering, Inc.

Boring Log: BH- 8

Raw Water Pipeline

Lebanon Water Treatment Plant

Lebanon, Oregon

Depth Feet	Soil and Rock Description and Comments	Log	Elev. Depth 366.7	Samples	▲ SPT, N-Value ● Moisture, % ⊡ Recovery ⊞ RQD., % (0 – 50 – 100)	Installations/ Water Table
1	Medium dense CRUSHED GRAVEL (GP); grey, dry, ±¾-inch minus, subangular to angular, (fill).		0.0 / 365.7			Capped with gravel
2	Medium dense sandy GRAVEL (GP); grey, damp, fine to coarse sand, fine to coarse gravel, angular to rounded gravel, (fill).		1.0			
3				SS-9-1	▲ 56	Backfilled with bentonite chips
4						
5	Loose silty SAND (SM); brown, damp, low plasticity silt, fine sand, (alluvium).		361.7 / 5.0	SS-9-2	▲ 4 ●	
6						
7						
8				SS-9-3	▲ 7 ●	
9						
10	Medium dense silty SAND (SM); brown, damp, low plasticity silt, fine sand, (alluvium).		356.7 / 10.0	SS-9-4	● ▲ 21	
11			355.2			
	BOTTOM OF BORING		11.5			

Project No.: 2131070-101

Surface Elevation: 366.7 feet (Approx.)

Date of Boring: September 21, 2015

Boring Log: BH- 9

Raw Water Pipeline

Lebanon Water Treatment Plant

Lebanon, Oregon

Foundation Engineering, Inc.

Depth Feet	Soil and Rock Description and Comments	Log	Elev. Depth	Samples	SPT, N-Value ▲ / Recovery ⬚ / Moisture, % ● / RQD., % ⊞ (0 – 50 – 100)	Installations/ Water Table
			363.1			
	ASPHALTIC CONCRETE (±6 inches).		362.6			Capped with AC cold patch and gravel
1	Dense CRUSHED GRAVEL (GP); grey, damp,		0.5			
2	±¾-inch minus, subangular gravel, (base rock).		360.6			
3	Soft SILT, trace sand (ML); brown, damp, low		2.5	SS-10-1	▲ 4 ●	Backfilled with bentonite chips
4	plasticity, fine sand, (alluvium).					
5	Sandy with scattered organics below ±5 feet.			SS-10-2	▲ 4 ●	
6						
7			355.6			
8	Very dense sandy GRAVEL, trace silt (GP);		7.5	SS-10-3	▲ 57	
9	grey-brown, wet, fine to coarse sand, fine to coarse gravel, subrounded to rounded gravel, (alluvium).					
10			353.1			
11	Medium dense SAND, some silt, trace gravel (SM); grey, wet, low plasticity silt, fine to coarse sand, fine gravel, subrounded to rounded gravel, (alluvium).		10.0	SS-10-4	▲ 22	
			351.6			
	BOTTOM OF BORING		11.5			

Project No.: 2131070-101

Surface Elevation: 363.1 feet (Approx.)

Date of Boring: September 22, 2015

Boring Log: BH-10

Raw Water Pipeline

Lebanon Water Treatment Plant

Lebanon, Oregon

Foundation Engineering, Inc.

Depth Feet	Soil and Rock Description and Comments	Log	Elev. Depth	Samples	SPT, N-Value ▲ / Moisture, % ● / Recovery ⊡ / RQD., % ⊞ (0 – 50 – 100)	Installations/ Water Table
			359.4			
	ASPHALTIC CONCRETE (±5 inches).		359.0			Capped with AC cold patch and gravel
1	Dense CRUSHED GRAVEL (GP); grey, damp, ±¾-inch minus, subangular gravel, (base rock).		0.4			
2			356.9			
3	Medium stiff sandy SILT (ML); brown, damp, low plasticity, fine sand, (alluvium).		2.5	SS-11-1	▲ 7 ●	Backfilled with bentonite chips
4						
5	Soft and wet below ±5 feet.			SS-11-2	▲ 2	
6						
7			351.9			
8	Medium dense sandy GRAVEL, trace silt (GP); brown and iron-stained, wet, fine to coarse sand, fine to coarse gravel, subrounded to rounded gravel, (alluvium).		7.5	SS-11-3	▲ 23	
9						
10	Grey below ±10 feet.			SS-11-4	▲ 21	
11			347.9			
	BOTTOM OF BORING		11.5			

Project No.: 2131070-101

Surface Elevation: 359.4 feet (Approx.)

Date of Boring: September 22, 2015

Boring Log: BH-11

Raw Water Pipeline

Lebanon Water Treatment Plant

Lebanon, Oregon

Foundation Engineering, Inc.

Depth Feet	Soil and Rock Description and Comments	Log	Elev. Depth (362.1)	Samples	SPT, N-Value / Recovery / Moisture, % / RQD., % (0–50–100)	Installations/ Water Table
	ASPHALTIC CONCRETE (±7 inches).		361.5			Capped with AC cold patch and gravel
1–4	Very dense sandy GRAVEL, trace silt (GP); grey-brown, damp, fine to coarse sand, fine to coarse gravel, subangular gravel, (fill).		0.6	SS-12-1	50/5½"	Backfilled with bentonite chips
5	Very stiff SILT, trace gravel (ML); brown, damp, low plasticity, fine gravel, subrounded gravel, (alluvium).		357.1 5.0	SS-12-2	28	
6–9	Dense to very dense sandy GRAVEL, trace silt (GP); grey-brown, wet, fine to coarse sand, fine to coarse gravel, subrounded to rounded gravel, (alluvium).		356.1 6.0	SS-12-3	40	
10–12				SS-12-4	63	
13–14	Scattered cobbles (up to ±6 inches in diameter) from ±12.5 to 20 feet.			SS-12-5	32	
15–16				SS-12-6	46	
17–19				SS-12-7	55	
20	Medium dense silty GRAVEL, some sand (GM); brown, moist, low plasticity silt, fine to coarse sand, fine to coarse gravel, subrounded to rounded gravel, (alluvium).		342.1 20.0	SS-12-8	23	
21–24	Very dense below ±24 feet.					
25–29				SS-12-9	71	
30–34				SS-12-10	50/1st 5"	
35–36				SS-12-11	65	
	BOTTOM OF BORING		325.6 36.5			

Project No.: 2131070-101

Surface Elevation: 362.1 feet (Approx.)

Date of Boring: September 21, 2015

Boring Log: BH-12

Raw Water Pipeline

Lebanon Water Treatment Plant

Lebanon, Oregon

Foundation Engineering, Inc.

Depth Feet	Soil and Rock Description and Comments	Log	Elev. Depth	Samples	SPT, N-Value / Recovery / Moisture, % / RQD., % (0–50–100)	Installations/ Water Table
			362.1			
1–4	Very dense silty GRAVEL, some sand, scattered cobbles (GM); brown, moist, low plasticity silt, fine to coarse sand, fine to coarse gravel, subrounded to rounded gravel and cobbles up to ±6 inches in diameter, (alluvium).		0.0	SS-13-1	52	Morris monument set in concrete; Bentonite chips
5–6	Thin silt lens (±12 inches thick) at ±5 feet.		356.1	SS-13-2	16	±1-inch I.D. PVC
6–8	Dense sandy GRAVEL, trace silt (GP); grey, wet, fine to coarse sand, fine to coarse gravel, subrounded to rounded gravel, (alluvium).		6.0	SS-13-3	34	
10–11				SS-13-4	41	Ground water (10/01/15)
13–14	Very dense and scattered cobbles (up to ±6 inches in diameter) below ±12.5 feet.			SS-13-5	59	
15–16				SS-13-6	93/9"	Colorado silica sand
18–19				SS-13-7	56	
20–21				SS-13-8	71	0.010 machine slot screen
25–26	Dense silty GRAVEL, some sand (GM); grey-brown, moist, low plasticity silt, fine to coarse sand, fine to coarse gravel, subrounded to rounded gravel, (alluvium).		337.1 / 25.0	SS-13-9	46	
27–28	Very dense below ±27 feet.					
30–31				SS-13-10	50/5"	Bentonite chips
35			326.7	SS-13-11	50/1st 5"	
	BOTTOM OF BORING		35.4			

Project No.: 2131070-101

Surface Elevation: 362.1 feet (Approx.)

Date of Boring: September 29, 2015

Boring Log: BH-13

Raw Water Pipeline

Lebanon Water Treatment Plant

Lebanon, Oregon

Foundation Engineering, Inc.

Depth Feet	Soil and Rock Description and Comments	Log	Elev. Depth 362.4	Samples	▲ SPT, N-Value ● Moisture, % ⊡ Recovery ⊞ RQD., % (0 – 50 – 100)	Installations/ Water Table
	ASPHALTIC CONCRETE (±6 inches)		361.9			Capped with AC cold patch and gravel
1	Dense CRUSHED GRAVEL, trace silt (GP); grey, wet, ±¾-inch minus, subangular to subrounded gravel, (base rock).		0.5			
2			360.4			
3	Stiff clayey SILT (MH); brown, moist, medium plasticity, (alluvium).		2.0	SS-14-1	▲ 11 ●	Backfilled with bentonite chips
4			358.4			
5	Very dense sandy GRAVEL, trace silt (GP); grey, wet, fine to coarse sand, fine to coarse gravel, subrounded to rounded gravel, (alluvium).		4.0	SS-14-2	▲ 54	
6						
7	Scattered cobbles (up to ±6 inches in diameter) below ±5 feet.					
8				SS-14-3	▲ 50/1st 5"	
9						
10				SS-14-4	▲ 50/5"	
	BOTTOM OF BORING		351.5 10.9			

Project No.: 2131070-101

Surface Elevation: 362.4 feet (Approx.)

Date of Boring: September 29, 2015

Foundation Engineering, Inc.

Boring Log: BH-14

Water Transmission Main and Sanitary Sewer Main

Lebanon Water Treatment Plant

Lebanon, Oregon

Depth Feet	Soil and Rock Description and Comments	Log	Elev. Depth 360	Samples	▲ SPT, N-Value ● Moisture, % ⊡ Recovery ⊞ RQD., % (0 – 50 – 100)	Installations/ Water Table
	ASPHALTIC CONCRETE (±6 inches)		359.5			Capped with AC cold patch and gravel
1	Dense CRUSHED GRAVEL (GP); grey, damp, ±¾-inch minus, subangular to subrounded gravel, (base rock).		0.5			
2			358.0			
3	Stiff SILT (ML); dark brown, damp, low plasticity, (alluvium).		2.0	SS-15-1	▲ 9 ●	Backfilled with bentonite chips
4						
5	Some sand and brown below ±5 feet.			SS-15-2	▲ 5 ●	
6			354.0			
7	Loose silty SAND (SM); brown, moist, low plasticity silt, fine to medium sand, (alliuvium).		6.0			
8	Medium dense and wet with trace gravel below ±7.5 feet.			SS-15-3	▲ 13 ●	
9						
10			350.0			
11	Very dense sandy GRAVEL, trace silt (GP); brown, wet, fine to coarse sand, fine to coarse gravel, subrounded to rounded gravel, (alluvium).		10.0	SS-15-4	▲ 86	
			348.5			
	BOTTOM OF BORING		11.5			

Project No.: 2131070-101

Surface Elevation: 360.0 feet (Approx.)

Date of Boring: September 29, 2015

Foundation Engineering, Inc.

Boring Log: BH-15

Water Transmission Main and Sanitary Sewer Main

Lebanon Water Treatment Plant

Lebanon, Oregon

Comments	Depth, Feet	Sample #	Location	Class Symbol	Water Table	C, TSF	Symbol	Soil and Rock Description
Surface: grass. Fine roots extend to ±5 inches.	1	S-1-1						Stiff SILT, scattered organics (ML); brown, moist, low to medium plasticity, organics consist of fine roots, (topsoil).
	2	S-1-2				0.9 tsf 0.75 tsf		Stiff clayey SILT (MH); light brown, damp to moist, medium plasticity, (alluvium).
	3	S-1-3						
	4							
	5							
	6							
Slow seepage at ±7 feet.	7	S-1-4						Loose to medium dense silty SAND (SM); brown, wet, low plasticity silt, fine to medium sand, (alluvium).
	8							
	9							
	10							Dense sandy GRAVEL and COBBLES (GP); brown, wet, fine to coarse sand, fine to coarse gravel, subrounded to rounded gravel and cobbles, cobbles up to ±7 inches in diameter, (alluvium).
	11							
	12							BOTTOM OF TEST PIT
	13							
	14							

Project No.: 2131070

Surface Elevation: 363.8 feet (Approx.)

Date of Test Pit: November 1, 2013

Test Pit Log: TP- 1
Water Treatment Plant
Lebanon Water Treatment Plant
Lebanon, Oregon

Comments	Depth, Feet	Sample #	Location	Class Symbol	Water Table	C, TSF	Symbol	Soil and Rock Description
Surface: grass. Fine roots extend to ±6 inches.		S-2-1						Stiff SILT, scattered organics (ML); brown, moist, low to medium plasticity, (topsoil/fill).
	1					0.6 tsf		
	2	S-2-2				0.8 tsf		Stiff clayey SILT (MH); light brown, moist, medium plasticity, (alluvium).
	3							
	4							
	5							Dense sandy GRAVEL and COBBLES (GP); brown, moist, fine to coarse sand, fine to coarse gravel, subrounded to rounded gravel and cobbles, cobbles up to ±8 inches in diameter, (alluvium).
	6							
No seepage or groundwater encountered to the limit of excavation.	7							BOTTOM OF TEST PIT
	8							
	9							
	10							
	11							
	12							
	13							
	14							

Project No.: 2131070

Surface Elevation: 364.6 feet (Approx.)

Date of Test Pit: November 1, 2013

Test Pit Log: TP- 2
Water Treatment Plant
Lebanon Water Treatment Plant
Lebanon, Oregon

Comments	Depth, Feet	Sample #	Location	Class Symbol	Water Table	C, TSF	Symbol	Soil and Rock Description
Surface: grass.								
	1	S-3-1				0.3 tsf		Medium stiff SILT, trace to some sand and gravel, scattered organics (ML); brown, moist, low plasticity, fine to coarse sand, fine to coarse gravel, subangular to rounded gravel, organics consist of fine roots and wood debris, (topsoil/fill).
Fine roots extend to ±1.7 feet.	2							
	3							
	4	S-3-2				>1 tsf		Stiff, clayey SILT (MH); grey, iron-stained, moist, medium plasticity, (alluvium).
	5							No iron-staining below ±5 feet.
	6	S-3-3						
	7							
	8	S-3-4						Soft to medium stiff sandy SILT (ML); brown, wet, low plasticity, fine sand, (alluvium).
Slow seepage at ±9 feet.	9							
	10							
	11							
	12							Dense sandy GRAVEL and COBBLES (GP); brown, wet, fine to coarse sand, fine to coarse gravel, subrounded to rounded gravel and cobbles, cobbles up to ±7 inches in diameter, (alluvium).
	13							
	14							BOTTOM OF TEST PIT

Project No.: 2131070

Surface Elevation: 363.5 feet (Approx.)

Date of Test Pit: November 1, 2013

Test Pit Log: TP- 3
Water Treatment Plant
Lebanon Water Treatment Plant
Lebanon, Oregon

Comments	Depth, Feet	Sample #	Location	Class Symbol	Water Table	C, TSF	Symbol	Soil and Rock Description
Surface: grass.		S-4-1						Medium stiff SILT, some organics, trace gravel (ML); brown, moist, low plasticity, fine to coarse, rounded gravel, organics consist of wood debris and fine roots, (topsoil/fill).
	1							
Fine roots extend to ±1-foot.	2	S-4-2				0.55 tsf		Stiff clayey SILT (MH); brown, iron-stained, moist, medium plasticity, (alluvium).
	3					>1 tsf		
	4	S-4-3				0.75 tsf		Becomes blue-grey and iron-stained below ±4 feet.
	5							No iron-staining below ±5 feet.
	6							
	7	S-4-4						Medium dense silty SAND (SM); blue-grey, moist to wet, low plasticity silt, fine to medium sand, (alluvium).
	8							Dense sandy GRAVEL and COBBLES (GP); blue-grey, moist to wet, fine to coarse sand, fine to coarse gravel, subrounded to rounded gravel and cobbles, cobbles up to ±8 inches in diameter, (alluvium).
No seepage or groundwater encountered to the limit of excavation.	9							
	10							BOTTOM OF TEST PIT
	11							
	12							
	13							
	14							

Project No.: 2131070

Surface Elevation: 362.9 feet (Approx.)

Date of Test Pit: November 1, 2013

Test Pit Log: TP- 4
Water Treatment Plant
Lebanon Water Treatment Plant
Lebanon, Oregon

Comments	Depth, Feet	Sample #	Location	Class Symbol	Water Table	C, TSF	Symbol	Soil and Rock Description
Surface: grass. Brick debris at ±0.5 feet. Fine roots extend to ±1.5 feet.	1 2 3 4 5 6	S-5-1 S-5-2				0.35 tsf		Medium stiff SILT, some gravel, scattered organics (ML); moist, low plasticity, fine to coarse, subrounded to rounded gravel, organics consist of fine roots, (topsoil/fill).
								Dense GRAVEL and COBBLES, some sand and silt (GP); brown, moist, low plasticity silt, fine to coarse sand, fine to coarse gravel, subrounded to rounded gravel and cobbles, cobbles up to ±9 inches in diameter, (alluvium). Sandy below ±3.5 feet.
No seepage or groundwater encountered to the limit of excavation.	7 8 9 10 11 12 13 14							BOTTOM OF TEST PIT

Project No.: 2131070

Surface Elevation: 361.9 feet (Approx.)

Date of Test Pit: November 1, 2013

Test Pit Log: TP- 5
Water Treatment Plant
Lebanon Water Treatment Plant
Lebanon, Oregon

Comments	Depth, Feet	Sample #	Location	Class Symbol	Water Table	C, TSF	Symbol	Soil and Rock Description
Surface: grass. Tree limbs (±4 inches in diameter) at ±6 inches. Fine roots extend to ±7 inches. Rubber belt debris at ±1-foot.	1 2 3 4	S-6-1 S-6-2				0.6 tsf		Medium stiff SILT, scattered organics (ML); brown, moist, low to medium plasticity, organics consist of fine roots and wood debris, (topsoil/fill).
								Stiff clayey SILT (MH); brown, damp, medium plasticity, (alluvium).
								Dense sandy GRAVEL and COBBLES (GP); brown, moist, fine to coarse sand, fine to coarse gravel, subrounded to rounded gravel and cobbles, cobbles up to ±8 inches in diameter, (alluvium).
No seepage or groundwater encountered to the limit of excavation.	5 6 7 8 9 10 11 12 13 14							BOTTOM OF TEST PIT

Project No.: 2131070

Surface Elevation: 364.4 feet (Approx.)

Date of Test Pit: November 1, 2013

Test Pit Log: TP- 6
Water Treatment Plant
Lebanon Water Treatment Plant
Lebanon, Oregon

Comments	Depth, Feet	Sample #	Location	Class Symbol	Water Table	C, TSF	Symbol	Soil and Rock Description
Surface: grass. Fine roots extend to ±8 inches. Tree limbs (±4 inches in diameter) at ±1-foot. No seepage or groundwater encountered to the limit of excavation.	1 2 3 4 5 6 7 8 9 10 11 12 13 14	S-7-1 S-7-2				0.6 tsf 0.9 tsf		Medium stiff SILT, some organics (ML); dark brown, damp to moist, low plasticity, organics consist of fine roots and wood debris, (topsoil/fill). Stiff clayey SILT (MH); light brown, damp, medium plasticity, (alluvium). Dense sandy GRAVEL and COBBLES (GP); grey, moist, fine to coarse sand, fine to coarse gravel, subrounded to rounded gravel and cobbles, cobbles up to ±6 inches in diameter, (alluvium). BOTTOM OF TEST PIT

Project No.: 2131070

Surface Elevation: 364.7 feet (Approx.)

Date of Test Pit: November 1, 2013

Test Pit Log: TP- 7
Water Treatment Plant
Lebanon Water Treatment Plant
Lebanon, Oregon

Comments	Depth, Feet	Sample #	Location	Class Symbol	Water Table	C, TSF	Symbol	Soil and Rock Description
Surface: grass. Fine roots extend to ±6 inches. No seepage or groundwater encountered to the limit of excavation.	1 2 3 4 5 6 7 8 9 10 11 12 13 14	SH-8-1 S-8-2				0.9 tsf >1 tsf		Medium stiff SILT, scattered organics (ML); brown, damp to moist, low to medium plasticity, organics consist of fine roots and wood debris, (topsoil/fill). Stiff clayey SILT (MH); light brown, damp, medium plasticity, (alluvium). Moist below ±3 feet. Dense sandy GRAVEL and COBBLES (GP); brown, moist, fine to coarse sand, fine to coarse gravel, subrounded to rounded gravel and cobbles, cobbles up to ±6 inches in diameter, (alluvium). BOTTOM OF TEST PIT

Project No.: 2131070

Surface Elevation: 364.8 feet (Approx.)

Date of Test Pit: November 1, 2013

Test Pit Log: TP- 8
Water Treatment Plant
Lebanon Water Treatment Plant
Lebanon, Oregon

Comments	Depth, Feet	Sample #	Location	Class Symbol	Water Table	C, TSF	Symbol	Soil and Rock Description
Surface: grass. Fine roots extend to ±6 inches.	1 2 3	S-9-1				0.4 tsf		Medium stiff SILT, trace to some gravel, scattered cobbles and organics (ML); dark brown, damp, low plasticity, fine to coarse gravel, subrounded gravel and cobbles, cobbles up to ±5 inches in diameter, organics consist of fine roots, (topsoil/fill).
	4 5 6 7	S-9-2				0.9 tsf		Stiff clayey SILT (MH); blue-grey, iron-stained, moist, medium plasticity, (alluvium). No iron-staining below ±5 feet.
Slow seepage observed at ±9 feet.	8 9 10 11 12	S-9-3 S-9-4						Medium dense silty SAND (SM); grey-brown, wet, low plasticity silt, fine to medium sand, (alluvium).
	13 14							Dense sandy GRAVEL and COBBLES (GP); brown, wet, fine to coarse sand, fine to coarse gravel, subrounded to rounded gravel and cobbles, cobbles up to ±6 inches in diameter, (alluvium). BOTTOM OF TEST PIT

Project No.: 2131070

Surface Elevation: 363.9 feet (Approx.)

Date of Test Pit: November 1, 2013

Test Pit Log: TP- 9
Water Treatment Plant
Lebanon Water Treatment Plant
Lebanon, Oregon

Comments	Depth, Feet	Sample #	Location	Class Symbol	Water Table	C, TSF	Symbol	Soil and Rock Description
Surface: grass. Fine roots extend to ±8 inches.	1 2 3	S-10-1 S-10-2				0.4 tsf >1 tsf >1 tsf		Medium stiff SILT, some gravel, scattered cobbles and organics (ML); dark brown, damp, low plasticity, fine to coarse gravel, subrounded to rounded gravel and cobbles, cobbles up to ±4 inches in diameter, organics consist of fine roots, (topsoil/fill). Medium stiff silty CLAY, trace gravel (CL); grey-brown, moist, medium plasticity, fine, subrounded gravel, (fill).
	4 5	S-10-3						Stiff clayey SILT (MH); light brown, iron-stained, moist, medium plasticity, (alluvium).
	6							Medium stiff sandy SILT (ML); blue-grey and iron-stained, moist, low plasticity, fine sand, (alluvium).
	7							Medium dense silty SAND (SM); blue-grey, moist to wet, low plasticity silt, fine to medium sand, (alluvium).
No seepage or groundwater encountered to the limit of excavation.	8 9 10 11 12 13 14							Dense sandy GRAVEL and COBBLES (GP); blue, wet, fine to coarse sand, fine to coarse gravel, subrounded to rounded gravel and cobbles, cobbles up to ±4 inches in diameter, (alluvium). BOTTOM OF TEST PIT

Project No.: 2131070

Surface Elevation: 363.1 feet (Approx.)

Date of Test Pit: November 1, 2013

Test Pit Log: TP-10
Water Treatment Plant
Lebanon Water Treatment Plant
Lebanon, Oregon

Comments	Depth, Feet	Sample #	Location	Class Symbol	Water Table	C, TSF	Symbol	Soil and Rock Description
Surface: grass.		S-11-1						Medium stiff SILT, trace gravel, scattered organics (ML); dark brown, low plasticity, fine gravel, subrounded gravel, organics consist of fine roots and wood debris, (topsoil/fill).
Fine roots extend to ±12 inches.	1							
Plastic sheet debris and rebar at ±1-foot.	2							Dense sandy GRAVEL and COBBLES (GP); brown, moist, fine to coarse sand, fine to coarse gravel, subrounded to rounded gravel and cobbles, cobbles up to ±8 inches in diameter, (alluvium).
Glass fragments at ±1.3 feet.	3							
No seepage or groundwater encountered to the limit of excavation.	4							BOTTOM OF TEST PIT
	5							
	6							
	7							
	8							
	9							
	10							
	11							
	12							
	13							
	14							

Project No.: 2131070

Surface Elevation: 362.4 feet (Approx.)

Date of Test Pit: November 1, 2013

Test Pit Log: TP-11
Water Treatment Plant
Lebanon Water Treatment Plant
Lebanon, Oregon

Comments	Depth, Feet	Sample #	Location	Class Symbol	Water Table	C, TSF	Symbol	Soil and Rock Description
Surface: grass.								Medium stiff SILT, scattered organics (ML); brown, moist, low to medium plasticity, organics consist of fine roots (topsoil).
Fine roots extend to ±7 inches.	1							
	2	S-12-1				0.65 tsf		Stiff clayey SILT (MH); light brown, moist, medium plasticity, (alluvium).
	3							
	4	S-12-2				>1 tsf		Medium dense silty SAND (SM); brown, moist, low plasticity silt, fine to medium sand, (alluvium).
	5							
	6							
Rapid seepage at ±7 feet.	7							Dense sandy GRAVEL and COBBLES (GP); brown, wet, fine to coarse sand, fine to coarse gravel, subrounded to rounded gravel and cobbles, cobbles up to ±6 inches in diameter, (alluvium).
	8							BOTTOM OF TEST PIT
	9							
	10							
	11							
	12							
	13							
	14							

Project No.: 2131070

Surface Elevation: 364.1 feet (Approx.)

Date of Test Pit: November 1, 2013

Test Pit Log: TP-12
Water Treatment Plant
Lebanon Water Treatment Plant
Lebanon, Oregon

Comments	Depth, Feet	Sample #	Location	Class Symbol	Water Table	C, TSF	Symbol	Soil and Rock Description
Surface: grass.		S-13-1				0.45 tsf		Medium stiff SILT, scattered organics (ML); brown, moist, low to medium plasticity, organics consist of fine roots and wood debris (topsoil).
Fine roots extend to ±12 inches.	1							
	2	S-13-2				0.9 tsf		Stiff clayey SILT (MH); light brown, moist, medium plasticity, (alluvium).
	3					>1 tsf		
	4							
	5							Dense sandy GRAVEL and COBBLES (GP); brown, moist, fine to coarse sand, fine to coarse gravel, subrounded to rounded gravel and cobbles, cobbles up to ±6 inches in diameter, (alluvium).
No seepage or groundwater encountered to the limit of excavation.	6							
	7							BOTTOM OF TEST PIT
	8							
	9							
	10							
	11							
	12							
	13							
	14							

Project No.: 2131070

Surface Elevation: 364.4 feet (Approx.)

Date of Test Pit: November 1, 2013

Test Pit Log: TP-13
Water Treatment Plant
Lebanon Water Treatment Plant
Lebanon, Oregon

Comments	Depth, Feet	Sample #	Location	Class Symbol	Water Table	C, TSF	Symbol	Soil and Rock Description
Surface: grass.								Soft SILT, trace gravel, scattered organics (MH); dark brown, moist, low plasticity, fine, subrounded gravel, (topsoil/fill).
Fine roots extend to ±4 inches.	1	S-14-1						
		S-14-2				0.75 tsf		Stiff SILT, scattered organics (ML); grey-brown, moist, low to medium plasticity, organics consist of fine roots and wood debris, (topsoil/fill).
	2							
	3							Stiff clayey SILT (MH); light brown, moist, medium plasticity, (alluvium).
	4							
No seepage or groundwater encountered to the limit of excavation.	5							Dense sandy GRAVEL and COBBLES (GP); brown, moist, fine to coarse sand, fine to coarse gravel, subrounded to rounded gravel and cobbles, cobbles up to ±6 inches in diameter, (alluvium).
	6							
	7							BOTTOM OF TEST PIT
	8							
	9							
	10							
	11							
	12							
	13							
	14							

Project No.: 2131070

Surface Elevation: 364.3 feet (Approx.)

Date of Test Pit: November 1, 2013

Test Pit Log: TP-14
Water Treatment Plant
Lebanon Water Treatment Plant
Lebanon, Oregon

Comments	Depth, Feet	Sample #	Location	Class Symbol	Water Table	C, TSF	Symbol	Soil and Rock Description
Surface: grass. Fine roots extend to ±4 inches. Metal cable (±¼-inch in diameter) at ±1.5 feet. Cobble (±5 inches in diameter) at ±2.6 feet. Slow seepage at ±8 feet.	1 2 3 4 5 6 7 8 9 10 11 12 13 14	S-15-1 S-15-2 S-15-3 S-15-4				0.45 tsf 0.9 tsf		Medium stiff SILT, trace gravel and cobbles, scattered organics (ML); dark brown, moist, low plasticity, fine to coarse gravel, subrounded gravel and cobbles, cobbles up to ±6 inches in diameter, organics consist of fine roots and wood debris, (topsoil/fill). Medium stiff SILT, trace sand and gravel, scattered organics (ML); brown, moist, fine to coarse sand, fine to coarse gravel, subrounded gravel and cobbles, organics consist of wood debris, (fill). Stiff clayey SILT (MH); grey, moist, medium plasticity silt, (alluvium). Medium dense silty SAND (SM); blue-grey, wet, low to medium plasticity, fine to medium sand, (alluvium). Dense sandy GRAVEL and COBBLES (GP); blue-grey, wet, fine to coarse sand, fine to coarse gravel, subrounded to rounded gravel and cobbles, cobbles up to ±6 inches in diameter, (alluvium). BOTTOM OF TEST PIT

Project No.: 2131070

Surface Elevation: 363.4 feet (Approx.)

Date of Test Pit: November 1, 2013

Test Pit Log: TP-15
Water Treatment Plant
Lebanon Water Treatment Plant
Lebanon, Oregon

Comments	Depth, Feet	Sample #	Location	Class Symbol	Water Table	C, TSF	Symbol	Soil and Rock Description
Surface: grass. Fine roots extend to ±5 inches. Slow seepage at ±8.5 feet.	1 2 3 4 5 6 7 8 9 10 11 12 13 14	S-16-1 S-16-2 S-16-3 S-16-4				0.4 tsf 0.65 tsf		Medium stiff SILT, some organics, trace gravel, scattered cobbles (ML); brown, moist, low plasticity, fine to coarse gravel, subrounded gravel and cobbles, cobbles up to ±4 inches in diameter, organics consist of fine roots, (topsoil/fill). Medium stiff silty CLAY, trace gravel (CL); grey-brown, iron-stained, moist, low to medium plasticity, fine to coarse gravel, subrounded gravel, blocky structure, (fill). Stiff clayey SILT (MH); brown, iron-stained, moist, medium plasticity, (alluvium). Medium stiff sandy SILT (ML); blue-grey and iron-stained, moist, low plasticity, fine sand, (alluvium). Wet below ±7 feet. Dense sandy GRAVEL and COBBLES (GP); blue-grey, wet, fine to coarse sand, fine to coarse gravel, subrounded to rounded gravel and cobbles, cobbles up to ±5 inches in diameter, (alluvium). BOTTOM OF TEST PIT

Project No.: 2131070

Surface Elevation: 362.9 feet (Approx.)

Date of Test Pit: November 1, 2013

Test Pit Log: TP-16
Water Treatment Plant
Lebanon Water Treatment Plant
Lebanon, Oregon

Comments	Depth, Feet	Sample #	Location	Class Symbol	Water Table	C, TSF	Symbol	Soil and Rock Description
Surface: grass. Fine roots extend to ±12 inches.	1	S-17-1						Very stiff gravelly SILT, scattered organics (ML); brown, dry, low plasticity, fine to coarse gravel, subrounded to rounded gravel, organics consist of fine roots, (topsoil/fill).
	2	S-17-2				>1.0		Very stiff clayey SILT (MH); light brown and iron-stained, damp, medium plasticity, (alluvium).
	3							
	4							
	5							
	6	S-17-3						Medium stiff sandy SILT to medium dense silty SAND (ML-SM); grey and iron-stained, moist, low plasticity, fine sand, (alluvium).
	7							
	8							
	9	S-17-4						Very dense sandy GRAVEL and COBBLES, trace silt (GP); grey, wet, fine to coarse sand, fine to coarse gravel, subrounded to rounded gravel and cobbles, cobbles up to ±10 inches in diameter, (alluvium).
No seepage or ground water encountered to the limit of excavation.	10							
	11							BOTTOM OF TEST PIT
	12							
	13							
	14							

Project No.: 2131070-101

Surface Elevation: 363.5 feet (Approx.)

Date of Test Pit: October 1, 2015

Test Pit Log: TP-17
Water Treatment Plant
Lebanon Water Treatment Plant
Lebanon, Oregon

Comments	Depth, Feet	Sample #	Location	Class Symbol	Water Table	C, TSF	Symbol	Soil and Rock Description
Surface: grass. Fine roots extend to ±8 inches.	1							Very stiff SILT, trace gravel, scattered organics (ML); brown, dry, low plasticity, fine gravel, subrounded gravel, organics consist of fine roots, (topsoil/fill).
	2	S-18-1				>1.0		Very stiff clayey SILT (MH); light brown and iron-stained, damp, medium plasticity, (alluvium).
	3							
	4	S-18-2						Dense sandy GRAVEL, trace silt (GP); brown, moist, fine to medium sand, fine to coarse gravel, subrounded to rounded gravel, (alluvium).
No seepage or ground water encountered to the limit of excavation.	5							
	6							BOTTOM OF TEST PIT
	7							
	8							
	9							
	10							
	11							
	12							
	13							
	14							

Project No.: 2131070-101

Surface Elevation: 363.0 feet (Approx.)

Date of Test Pit: October 1, 2015

Test Pit Log: TP-18
Water Treatment Plant
Lebanon Water Treatment Plant
Lebanon, Oregon

Comments	Depth, Feet	Sample #	Location	Class Symbol	Water Table	C, TSF	Symbol	Soil and Rock Description
Surface: grass. Fine roots extend to ± 12 inches.	1	S-19-1						Very stiff SILT, trace to some gravel, scattered organics (ML); brown, dry, low plasticity, fine to coarse gravel, subrounded to rounded gravel, organics consist of fine roots, (topsoil/fill).
	2	S-19-2						Dense silty GRAVEL and COBBLES, some sand (GM); brown, damp, low plasticity silt, fine to coarse sand, fine to coarse gravel, subrounded to rounded gravel and cobbles, cobbles up to ±4 inches in diameter, (alluvium).
	3							
	4							
No seepage or ground water encountered to the limit of excavation.	5							BOTTOM OF TEST PIT
	6							
	7							
	8							
	9							
	10							
	11							
	12							
	13							
	14							

Project No.: 2131070-101

Surface Elevation: 362.5 feet (Approx.)

Date of Test Pit: October 1, 2015

Test Pit Log: TP-19
Water Treatment Plant
Lebanon Water Treatment Plant
Lebanon, Oregon

Comments	Depth, Feet	Sample #	Location	Class Symbol	Water Table	C, TSF	Symbol	Soil and Rock Description
Surface: grass. Fine roots extend to ±9 inches.	1	S-20-1						Very stiff SILT, some organics, trace gravel (ML); brown, dry, low plasticity, fine gravel, subrounded gravel, organics consist of wood debris and fine roots, (topsoil/fill).
	2							
	3	S-20-2				>1.0		Very stiff clayey SILT (MH); brown and iron-stained, damp, medium plasticity, (alluvium).
	4							
	5							
	6	S-20-3						Medium stiff sandy SILT (ML); blue-grey and iron-stained, moist, low plasticity, fine sand, (alluvium).
	7							
	8							
	9							Very dense sandy GRAVEL and COBBLES, some silt (GP); brown, wet, low plasticity silt, fine to coarse sand, fine to coarse gravel, subrounded to rounded gravel and cobbles, cobbles up to ±5 inches in diameter, (alluvium).
No seepage or ground water encountered to the limit of excavation.	10							
	11							BOTTOM OF TEST PIT
	12							
	13							
	14							

Project No.: 2131070-101

Surface Elevation: 364.2 feet (Approx.)

Date of Test Pit: October 1, 2015

Test Pit Log: TP-20
Water Treatment Plant
Lebanon Water Treatment Plant
Lebanon, Oregon

Comments	Depth, Feet	Sample #	Location	Class Symbol	Water Table	C, TSF	Symbol	Soil and Rock Description
Surface: grass.								
Fine roots extend to ±8 inches.	1	S-21-1						Medium stiff SILT, some sand and gravel, scattered cobbles and organics (ML); brown, dry, low plasticity, fine to medium sand, fine to coarse gravel, subrounded to rounded gravel and cobbles, cobbles up to ±5 inches in diameter, organics consist of wood debris and fine roots, (topsoil/fill).
	2							
	3	S-21-2				>1.0		Very stiff clayey SILT (MH); brown and iron-stained, damp, medium plasticity, (alluvium).
	4							
	5							
	6	S-21-3						Medium stiff sandy SILT (ML); grey, moist, low plasticity, fine sand, (alluvium).
	7							
	8							
	9							
	10							Medium dense silty SAND (SM); brown, wet, low plasticity silt, fine to medium sand, (alluvium).
No seepage or ground water encountered to the limit of excavation.	11							Very dense sandy GRAVEL and COBBLES, trace silt (GP); brown, wet, fine to coarse sand, fine to coarse gravel, subrounded to rounded gravel and cobbles, cobbles up to ±6 inches in diameter, (alluvium).
	12							
	13							BOTTOM OF TEST PIT
	14							

Project No.: 2131070-101

Surface Elevation: 363.6 feet (Approx.)

Date of Test Pit: October 1, 2015

Test Pit Log: TP-21
Water Treatment Plant
Lebanon Water Treatment Plant
Lebanon, Oregon

Comments	Depth, Feet	Sample #	Location	Class Symbol	Water Table	C, TSF	Symbol	Soil and Rock Description
Surface: grass.								
Fine roots extend to ±8 inches.	1	S-22-1						Very stiff SILT, some sand and gravel, scattered organics (ML); brown, dry to damp, low plasticity, fine sand, fine to coarse gravel, subrounded to rounded gravel, organics consist of wood debris and fine roots, (topsoil/fill).
	2							
	3							
	4	S-22-2				>1.0		Very stiff clayey SILT (MH); grey and iron-stained, damp, medium plasticity, (alluvium).
	5							
	6							Medium stiff sandy SILT (ML); grey, moist, low plasticity, fine sand, (alluvium).
	7	S-22-3						
	8							
	9	S-22-4						Medium dense silty SAND (SM); brown, wet, low plasticity silt, fine sand, (alluvium).
	10							
No seepage or ground water encountered to the limit of excavation.	11							Very dense sandy GRAVEL and COBBLES, trace silt (GP); brown, wet, fine to coarse sand, fine to coarse gravel, subrounded to rounded gravel and cobbles, cobbles up to ±5 inches in diameter, (alluvium).
	12							
	13							BOTTOM OF TEST PIT
	14							

Project No.: 2131070-101

Surface Elevation: 364.4 feet (Approx.)

Date of Test Pit: October 1, 2015

Test Pit Log: TP-22
Water Treatment Plant
Lebanon Water Treatment Plant
Lebanon, Oregon

Comments	Depth, Feet	Sample #	Location	Class Symbol	Water Table	C, TSF	Symbol	Soil and Rock Description
Surface: grass.								
Fine roots extend to ±12 inches.	1	S-23-1						Very stiff SILT, trace sand and gravel, scattered organics (ML); brown, dry, low plasticity, fine sand, fine to coarse gravel, subrounded to rounded gravel, organics consist of fine roots, (topsoil/fill).
	2	S-23-2				>1.0		Very stiff clayey SILT (MH); brown and iron-stained, damp, medium plasticity, (alluvium).
	3							
	4							Medium stiff to stiff sandy SILT (ML); grey and iron-stained, moist to wet, low plasticity, fine sand, (alluvium).
	5	S-23-3						
	6							
	7							
	8	S-23-4						Medium dense silty SAND (SM); brown, wet, low plasticity silt, fine to medium sand, (alluvium).
	9							Very dense sandy GRAVEL and COBBLES, trace silt (GP); brown, wet, fine to coarse sand, fine to coarse gravel, subrounded to rounded gravel and cobbles, cobbles up to ±6 inches in diameter, (alluvium).
No seepage or ground water encountered to the limit of excavation.	10							
	11							
	12							BOTTOM OF TEST PIT
	13							
	14							

Project No.: 2131070-101

Surface Elevation: 363.4 feet (Approx.)

Date of Test Pit: October 1, 2015

Test Pit Log: TP-23
Water Treatment Plant
Lebanon Water Treatment Plant
Lebanon, Oregon

Comments	Depth, Feet	Sample #	Location	Class Symbol	Water Table	C, TSF	Symbol	Soil and Rock Description
Surface: grass.								
Fine roots extend to ±12 inches.	1	S-24-1						Stiff SILT, trace to some sand and gravel, scattered organics (ML); brown, dry, low plasticity, fine sand, fine to coarse gravel, subrounded gravel, organics consist of wood debris and fine roots, (topsoil/fill).
	2							
	3	S-24-2						Very stiff clayey SILT (MH); brown and iron-stained, damp, medium to high plasticity, (alluvium).
	4							
	5							Medium stiff sandy SILT (ML); grey and iron-stained, wet, low plasticity, fine sand, (alluvium).
	6	S-24-3						
	7							
	8							Medium dense silty SAND (SM); brown, wet, low plasticity silt, fine sand, (alluvium).
	9							Very dense sandy GRAVEL and COBBLES, trace silt (GP); brown, wet, fine to coarse sand, fine to coarse gravel, subrounded to rounded gravel and cobbles, cobbles up to ±6 inches in diameter, (alluvium).
No seepage or ground water encountered to the limit of excavation.	10							
	11							
	12							BOTTOM OF TEST PIT
	13							
	14							

Project No.: 2131070-101

Surface Elevation: 362.8 feet (Approx.)

Date of Test Pit: October 1, 2015

Test Pit Log: TP-24
Water Treatment Plant
Lebanon Water Treatment Plant
Lebanon, Oregon

Comments	Depth, Feet	Sample #	Location	Class Symbol	Water Table	C, TSF	Symbol	Soil and Rock Description
Surface: grass. Fine roots extend to ±10 inches. No seepage or ground water encountered to the limit of excavation.	1 2 3 4 5 6 7 8 9 10 11 12 13 14	S-25-1						Stiff SILT, some sand and gravel, scattered organics (ML); brown, dry, low plasticity, fine sand, fine to coarse gravel, subangular to rounded gravel, organics consist of wood debris and fine roots, (topsoil/fill). Very dense sandy GRAVEL AND COBBLES, trace silt (GP); grey-brown, damp, fine to coarse sand, fine to coarse gravel, subrounded to rounded gravel and cobbles, cobbles up to ±6 inches in diameter, (alluvium). BOTTOM OF TEST PIT

Project No.: 2131070-101

Surface Elevation: 362.7 feet (Approx.)

Date of Test Pit: October 1, 2015

Test Pit Log: TP-25
Water Treatment Plant
Lebanon Water Treatment Plant
Lebanon, Oregon

Comments	Depth, Feet	Sample #	Location	Class Symbol	Water Table	C, TSF	Symbol	Soil and Rock Description
Surface: grass. Fine roots extend to ±5 inches. No seepage or ground water encountered to the limit of excavation.	1 2 3 4 5 6 7 8 9 10 11 12 13 14	S-26-1						Stiff SILT, some sand and gravel, scattered organics (ML); brown, dry, low plasticity, fine sand, fine to coarse gravel, subrounded to rounded gravel, organics consist of fine roots, (topsoil/fill). Very stiff SILT, some sand, trace gravel (ML); brown, damp, low plasticity, fine sand, fine gravel, subrounded to rounded gravel, (alluvium). Very dense silty GRAVEL, some sand (GM); brown, moist, low plasticity silt, fine to coarse sand, fine to coarse gravel, subrounded to rounded gravel, (alluvium). BOTTOM OF TEST PIT

Project No.: 2131070-101

Surface Elevation: 363.8 feet (Approx.)

Date of Test Pit: October 1, 2015

Test Pit Log: TP-26
Water Treatment Plant
Lebanon Water Treatment Plant
Lebanon, Oregon

Comments	Depth, Feet	Sample #	Location	Class Symbol	Water Table	C, TSF	Symbol	Soil and Rock Description
Surface: grass. Fine roots extend to ±8 inches.	1 2 3 4 5	S-27-1				>1.0		Stiff SILT, some sand and gravel, scattered organics (ML); brown, dry, low plasticity, fine to coarse sand, fine to coarse gravel, subangular to rounded gravel, organics consist of fine roots, (topsoil/fill). Very stiff to medium dense sandy SILT to silty SAND (ML-SM); brown, damp, low plasticity, fine sand, (alluvium).
No seepage or ground water encountered to the limit of excavation.	6 7 8 9 10 11 12 13 14							Very dense silty GRAVEL and COBBLES, some sand (GM); brown, damp, low plasticity silt, fine to coarse sand, fine to coarse gravel, subrounded to rounded gravel and cobbles, cobbles up to ±8 inches in diameter, (alluvium). BOTTOM OF TEST PIT

Project No.: 2131070-101

Surface Elevation: 363.1 feet (Approx.)

Date of Test Pit: October 1, 2015

Test Pit Log: TP-27
Water Treatment Plant
Lebanon Water Treatment Plant
Lebanon, Oregon

Comments	Depth, Feet	Sample #	Location	Class Symbol	Water Table	C, TSF	Symbol	Soil and Rock Description
Surface: grass. Fine roots extend to ±10 inches.	1 2 3 4 5	S-28-1						Stiff SILT, some sand and gravel, scattered organics (ML); brown, dry, low plasticity, fine sand, fine gravel, subrounded to rounded gravel, organics consist of fine roots, (topsoil/fill). Very stiff to medium dense sandy SILT to silty SAND, trace gravel (ML-SM); brown, damp, low to medium plasticity, fine sand, fine gravel, subrounded gravel, (alluvium).
No seepage or ground water encountered to the limit of excavation.	6 7 8 9 10 11 12 13 14							Very dense silty GRAVEL AND COBBLES, some sand (GM); brown, damp, low plasticity silt, fine to coarse sand, fine to coarse gravel, subrounded to rounded gravel and cobbles, cobbles up to ±8 inches in diameter, (alluvium). BOTTOM OF TEST PIT

Project No.: 2131070-101

Surface Elevation: 364.3 feet (Approx.)

Date of Test Pit: October 1, 2015

Test Pit Log: TP-28
Water Treatment Plant
Lebanon Water Treatment Plant
Lebanon, Oregon

Comments	Depth, Feet	Sample #	Location	Class Symbol	Water Table	C, TSF	Symbol	Soil and Rock Description
Surface: weeds.								Stiff SILT, some sand, scattered organics (ML); brown, dry, low plasticity, fine sand, organics consist of roots up to ±½ inch in diameter, (topsoil).
	1							Loose silty SAND (SM); brown, damp, low plasticity silt, fine sand, (alluvium).
	2	S-29-1						
	3	S-29-2						Loose SAND, some silt (SP-SM); brown, damp, low plasticity silt, fine sand, (alluvium).
	4							
	5							
	6							
	7							
	8							Very dense sandy GRAVEL AND COBBLES, trace silt (GP); brown, moist, fine to medium sand, fine to coarse gravel, subrounded to rounded gravel and cobbles, cobbles up to ±10 inches in diameter, (alluvium).
No seepage or ground water encountered to the limit of excavation.	9							
	10							
	11							BOTTOM OF TEST PIT
	12							
	13							
	14							

Project No.: 2131070-101

Surface Elevation: 360.2 feet (Approx.)

Date of Test Pit: October 1, 2015

Test Pit Log: TP-29
Pump Station
Lebanon Water Treatment Plant
Lebanon, Oregon

Comments	Depth, Feet	Sample #	Location	Class Symbol	Water Table	C, TSF	Symbol	Soil and Rock Description
Surface: weeds.								Medium stiff SILT, some sand, scattered organics (ML); brown, dry, low plasticity, fine sand, organics consist of tree roots up to ±1 inch in diameter, (topsoil).
	1							Loose silty SAND (SM); brown, damp, low plasticity silt, fine sand, (alluvium).
	2	S-30-1						
Tree roots extend to ±3 feet.	3							
	4							
	5							Loose SAND, some silt (SP-SM); brown, damp, low plasticity silt, fine sand, (alluvium).
	6	S-30-2						
	7							
	8							
	9							
	10							Dense sandy GRAVEL, trace silt (GP); brown, moist, fine to medium sand, fine to coarse gravel, subrounded to rounded gravel, (alluvium).
No seepage or ground water encountered to the limit of excavation.	11							
	12							BOTTOM OF TEST PIT
	13							
	14							

Project No.: 2131070-101

Surface Elevation: 364.3 feet (Approx.)

Date of Test Pit: October 1, 2015

Test Pit Log: TP-30
Pump Station
Lebanon Water Treatment Plant
Lebanon, Oregon

Comments	Depth, Feet	Sample #	Location	Class Symbol	Water Table	C, TSF	Symbol	Soil and Rock Description
Surface: weeds.								Medium stiff SILT, scattered organics (ML); brown, dry, low plasticity, organics consist of tree roots up to ±1½ inches in diameter, (topsoil).
	1	S-31-1						
	2							Medium stiff sandy SILT, scattered organics (ML); brown, dry to damp, low plasticity, fine sand, organics consist of tree roots up to ±¼ inch in diameter, (alluvium).
	3	S-31-2						
	4							Loose SAND, some silt (SP-SM); brown, dry to damp, low plasticity silt, fine sand, (alluvium).
	5							
	6	S-31-3						
	7							
	8							
	9							Dense sandy GRAVEL, trace silt (GP); brown, moist, fine to medium sand, fine to coarse gravel, subrounded to rounded gravel, (alluvium).
No seepage or ground water encountered to the limit of excavation.	10							
	11							BOTTOM OF TEST PIT
	12							
	13							
	14							

Project No.: 2131070-101

Surface Elevation: 361.7 feet (Approx.)

Date of Test Pit: October 1, 2015

Test Pit Log: TP-31
Pump Station
Lebanon Water Treatment Plant
Lebanon, Oregon

Comments	Depth, Feet	Sample #	Location	Class Symbol	Water Table	C, TSF	Symbol	Soil and Rock Description
Surface: weeds.								Medium stiff SILT, some sand, scattered organics (ML); brown, dry, low plasticity, fine sand, organics consist of tree roots up to ±¼ inch in diameter, (topsoil).
	1							
	2	S-32-1						Loose silty SAND (SM); brown, dry to damp, low plasticity silt, fine sand, (alluvium).
	3	S-32-2						
	4							Damp below ±4 feet.
	5							
	6							
	7							
	8	S-32-3						Loose SAND, some silt (SP-SM); brown, damp, fine sand, (alluvium).
	9							
	10							
	11							Dense sandy GRAVEL, trace silt (GP); brown, moist, fine to medium sand, fine to coarse gravel, subrounded to rounded gravel, (alluvium).
No seepage or ground water encountered to the limit of excavation.	12							
	13							BOTTOM OF TEST PIT
	14							

Project No.: 2131070-101

Surface Elevation: 364.9 feet (Approx.)

Date of Test Pit: October 1, 2015

Test Pit Log: TP-32
Pump Station
Lebanon Water Treatment Plant
Lebanon, Oregon

Comments	Depth, Feet	Sample #	Location	Class Symbol	Water Table	C, TSF	Symbol	Soil and Rock Description
Surface: grass. Fine roots extend to ±4 inches.	1 2							Medium stiff SILT, some sand and gravel, scattered organics (ML); brown, moist, low plasticity, fine sand, fine to coarse gravel, subrounded to rounded gravel, organics consist of fine roots, (topsoil/fill).
	3 4	P-5-1						Dense silty sandy GRAVEL and COBBLES (GM); brown, moist to wet, fine to coarse sand, fine to coarse gravel, subrounded to rounded gravel and cobbles, cobbles up to ±6 inches in diameter, (alluvium).
Slow seepage at ±4.6 feet.	5 6 7 8 9 10 11 12 13 14							BOTTOM OF TEST PIT

Project No.: 2131070-101

Surface Elevation: 363.5 feet (Approx.)

Date of Test Pit: December 7, 2015

Test Pit Log: P- 5
Water Treatment Plant
Lebanon Water Treatment Plant
Lebanon, Oregon

Comments	Depth, Feet	Sample #	Location	Class Symbol	Water Table	C, TSF	Symbol	Soil and Rock Description
Surface: grass. Fine roots extend to ±4 inches.	1							Soft to medium stiff SILT, scattered organics (ML); brown, moist, low plasticity, (topsoil/fill).
	2 3 4 5							Stiff clayey SILT (MH); brown, moist, low to medium plasticity, (alluvium).
	6 7	P-6-1						Dense sandy GRAVEL and COBBLES, trace silt (GP); brown, moist, fine to coarse sand, fine to coarse gravel, subrounded to rounded gravel and cobbles, cobbles up to ±5 inches in diameter, (alluvium).
No seepage or ground water encountered to the limit of excavation.	8 9 10 11 12 13 14							BOTTOM OF TEST PIT

Project No.: 2131070-101

Surface Elevation: 365.0 feet (Approx.)

Date of Test Pit: December 7, 2015

Test Pit Log: P- 6
Water Treatment Plant
Lebanon Water Treatment Plant
Lebanon, Oregon

Comments	Depth, Feet	Sample #	Location	Class Symbol	Water Table	C, TSF	Symbol	Soil and Rock Description
Surface: grass.								Soft SILT (ML); brown, moist to wet, low plasticity, (topsoil/fill).
Fine roots extend to ±4 inches.	1							
	2							Stiff clayey SILT (MH); light brown, moist, medium plasticity, (alluvium).
	3							
	4							
	5							Dense silty sandy GRAVEL and COBBLES (GM); moist, fine to coarse sand, fine to coarse gravel, subrounded to rounded gravel and cobbles, cobbles up to ±4 inches in diameter, (alluvium).
	6	P-7-1						
No seepage or ground water encountered to the limit of excavation.	7							BOTTOM OF TEST PIT
	8							
	9							
	10							
	11							
	12							
	13							
	14							

Project No.: 2131070-101

Surface Elevation: 365.0 feet (Approx.)

Date of Test Pit: December 7, 2015

Test Pit Log: P- 7
Water Treatment Plant
Lebanon Water Treatment Plant
Lebanon, Oregon

Comments	Depth, Feet	Sample #	Location	Class Symbol	Water Table	C, TSF	Symbol	Soil and Rock Description
Surface: bare soil.								Soft SILT, some sand, scattered organics (ML); brown, wet, low plasticity, fine sand, organics consist of tree roots up to ±2 inches in diameter, (topsoil).
	1							
	2							Loose silty SAND (SM); brown, moist to wet, low plasticity silt, fine sand, (alluvium).
	3							
	4							Loose SAND to silty SAND (SP-SM); brown, damp, low plasticity silt, fine sand, (alluvium).
	5							
	6							
	7							
	8							
	9							
	10							
Moderate seepage at ±10.2 feet.	11	P-8-1						Dense sandy GRAVEL and COBBLES (GP); brown, wet, fine to coarse sand, fine to coarse gravel, subrounded to rounded gravel and cobbles, cobbles up to ±9 inches in diameter, (alluvium).
	12							
	13							BOTTOM OF TEST PIT
	14							

Project No.: 2131070-101

Surface Elevation: 362.0 feet (Approx.)

Date of Test Pit: December 7, 2015

Test Pit Log: P- 8
Pump Station
Lebanon Water Treatment Plant
Lebanon, Oregon

Comments	Depth, Feet	Sample #	Location	Class Symbol	Water Table	C, TSF	Symbol	Soil and Rock Description
Surface: bare soil.								Soft SILT, some sand, scattered organics (ML); brown, wet, low plasticity, fine sand, organics consist of tree roots up to ±1 inch in diameter, (topsoil).
	1							Loose silty SAND (SM); brown, moist to wet, low plasticity silt, fine sand, (alluvium).
	2							
	3							
	4							Loose SAND to silty SAND (SP-SM); brown, damp, low plasticity silt, fine sand, (alluvium).
	5							
	6	P-9-1						
No seepage or ground water encountered to the limit of excavation.	7							BOTTOM OF TEST PIT
	8							
	9							
	10							
	11							
	12							
	13							
	14							

Project No.: 2131070-101

Surface Elevation: 362.0 feet (Approx.)

Date of Test Pit: December 7, 2015

Test Pit Log: P- 9
Pump Station
Lebanon Water Treatment Plant
Lebanon, Oregon

Comments	Depth, Feet	Sample #	Location	Class Symbol	Water Table	C, TSF	Symbol	Soil and Rock Description
Surface: bare soil.								Soft SILT, some sand, scattered organics (ML); brown, wet, low plasticity, fine sand, organics consist of tree roots up to ±1 inch in diameter, (topsoil).
	1							Loose silty SAND (SM); brown, moist to wet, low plasticity silt, fine sand, (alluvium).
	2							
	3							
	4							Loose SAND to silty SAND (SP-SM); damp, low plasticity silt, fine sand, (alluvium).
	5	P-10-1						
No seepage or ground water encountered to the limit of excavation.	6							BOTTOM OF TEST PIT
	7							
	8							
	9							
	10							
	11							
	12							
	13							
	14							

Project No.: 2131070-101

Surface Elevation: 362.0 feet (Approx.)

Date of Test Pit: December 7, 2015

Test Pit Log: P-10
Pump Station
Lebanon Water Treatment Plant
Lebanon, Oregon

Appendix C

Field and Laboratory Test Results

Professional Geotechnical Services

Foundation Engineering, Inc.

Table 1C. Atterberg Limits, Natural Water Contents, and Percent Fines

Exploration	Sample Number	Sample Depth (feet)	Moisture Content (percent)	LL	PL	PI	USCS Classification	Percent Fines
BH-1	SH-1-1	2.5 - 4.0	34.0					
	SS-1-9	22.5 - 24.0	54.4					
	SS-1-10	25.0 - 26.5	47.8					47.0
BH-3	SH-3-1	2.5 - 4.0	43.9					
	SH-3-2	5.0 - 7.0	41.8	67	41	26	MH	
	SS-3-3	7.0 - 8.5	48.2					
	SS-3-4	10.0 - 11.4	51.8					
BH-4	SS-4-8	23.5 - 25.0	45.2					39.3
BH-5	SS-5-2	5.0 - 6.5	13.2					
BH-6	SS-6-1	2.5 - 4.0	21.3					
	SS-6-2	5.0 - 6.5	19.0					25.8
	SS-6-3	7.5 - 9.0	24.7					
BH-7	SS-7-1	2.5 - 4.0	28.9					54.2
	SS-7-2	5.0 - 6.5	24.1					10.4
	SS-7-3	7.5 - 9.0	27.5					16.7
BH-8	SS-8-1	2.5 - 4.0	15.8					
	SS-8-2	5.0 - 6.5	11.8					32.9
BH-9	SS-9-2	5.0 - 6.5	13.0					
	SS-9-3	7.5 - 9.0	11.0					14.6
	SS-9-4	10.0 - 11.5	11.6					
BH-10	SS-10-1	2.5 - 4.0	28.2	46	37	9	ML	
	SS-10-2	5.0 - 6.5	28.5					
BH-11	SS-11-1	2.5 - 4.0	31.6					53.1
BH-12	SS-12-2	5.0 - 6.5	22.0					
BH-14	SS-14-1	2.5 - 4.0	32.2					
BH-15	SS-15-1	2.5 - 4.0	23.5					
	SS-15-2	5.0 - 6.5	32.4					
	SS-15-3	7.5 - 9.0	24.6					16.4

Table 1C. Atterberg Limits, Natural Water Contents, and Percent Fines

Exploration	Sample Number	Sample Depth (feet)	Moisture Content (percent)	LL	PL	PI	USCS Classification	Percent Fines
TP-7	S-7-1	1.0 - 2.0	34.5					
	S-7-2	2.5 - 3.5	31.5					
TP-8	SH-8-1	2.0 - 3.0	32.1					
	S-8-2	3.0 - 4.0	39.8					
TP-9	S-9-1	1.0 - 2.0	28.8					
	S-9-2	4.0 - 4.5	42.7	62	39	23	MH	
	S-9-3	7.5 - 8.0	35.8					21.0
TP-10	S-10-1	1.0 - 1.5	23.6					
	S-10-2	2.0 - 3.0	38.1					
	S-10-3	4.0 - 5.0	31.8					53.5
TP-11	S-11-1	0 - 1.5	20.5					
TP-13	S-13-2	2.0 - 3.0	37.5					
TP-14	S-14-1	0.5 - 1.0	26.2					
	S-14-2	1.0 - 2.0	40.1					
TP-15	S-15-1	0 - 0.8	31.6					
	S-15-2	1.0 - 2.0	34.3					
	S-15-3	3.5 - 4.0	30.3					
	S-15-4	7.0 - 8.0	31.6					16.0
TP-16	S-16-1	0.5 - 1.0	26.7					
	S-16-2	1.5 - 2.5	37.1	68	38	30	MH	
	S-16-3	4.0 - 4.5	37.8					
TP-17	S-17-1	1.0 - 2.0	14.0					
	S-17-2	2.0 - 3.0	28.4					
	S-17-3	5.5 - 6.0	34.3					44.8
TP-18	S-18-1	2.0 - 3.0	22.5	51	35	16	MH	
TP-19	S-19-1	0.5 - 1.5	12.9					
TP-20	S-20-1	1.0 - 2.0	16.4					
	S-20-2	2.5 - 3.5	34.4					
	S-20-3	5.5 - 6.5	42.2					

Table 1C. Atterberg Limits, Natural Water Contents, and Percent Fines

Exploration	Sample Number	Sample Depth (feet)	Moisture Content (percent)	LL	PL	PI	USCS Classification	Percent Fines
TP-21	S-21-1	1.0 - 2.0	14.6					
	S-21-2	3.0 - 4.0	35.2					
	S-21-3	6.0 - 7.0	41.1					55.3
TP-22	S-22-1	1.0 - 2.0	18.7					
	S-22-2	3.5 - 4.0	28.8					
	S-22-3	6.5 - 7.0	38.6					
	S-22-4	8.5 - 9.0	37.2					
TP-23	S-23-1	1.0 - 1.5	17.7					
	S-23-2	2.0 - 3.0	31.2					
	S-23-3	4.5 - 5.5	40.8					65.4
	S-23-4	7.5 - 8.0	35.0					
TP-24	S-24-1	1.0 - 2.0	14.0					
	S-24-2	2.5 - 3.5	31.5	70	38	32	MH	
	S-24-3	5.5 - 6.0	38.2	48	36	12	ML	
TP-26	S-26-1	1.5 - 2.5	23.1					
TP-27	S-27-1	2.0 - 3.0	21.1					42.4
TP-28	S-28-1	2.0 - 3.0	23.2					49.2
TP-29	S-29-1	1.5 - 2.5	10.0					19.4
	S-29-2	3.0 - 4.0	9.1					13.0
TP-30	S-30-1	2.0 - 3.0	13.7					
TP-31	S-31-1	1.0 - 2.0	13.0					
	S-31-2	3.0 - 4.0	9.3					10.3
TP-32	S-32-1	2.0 - 3.0	13.6					
	S-32-2	3.0 - 4.0	14.1					
	S-32-3	7.5 - 8.0	10.6					
Pump Station	P-1	5.0 - 5.5	44.1					33.2
	P-2	2.0 - 2.5	37.4					34.0

Table 2C. Summary of Unconfined Compression Tests

Sample Number	Sample Depth (ft)	Compressive Strength (psi)
CS-5-1	38.4 – 39.1	513
CS-5-2	43.9 – 45.0	219

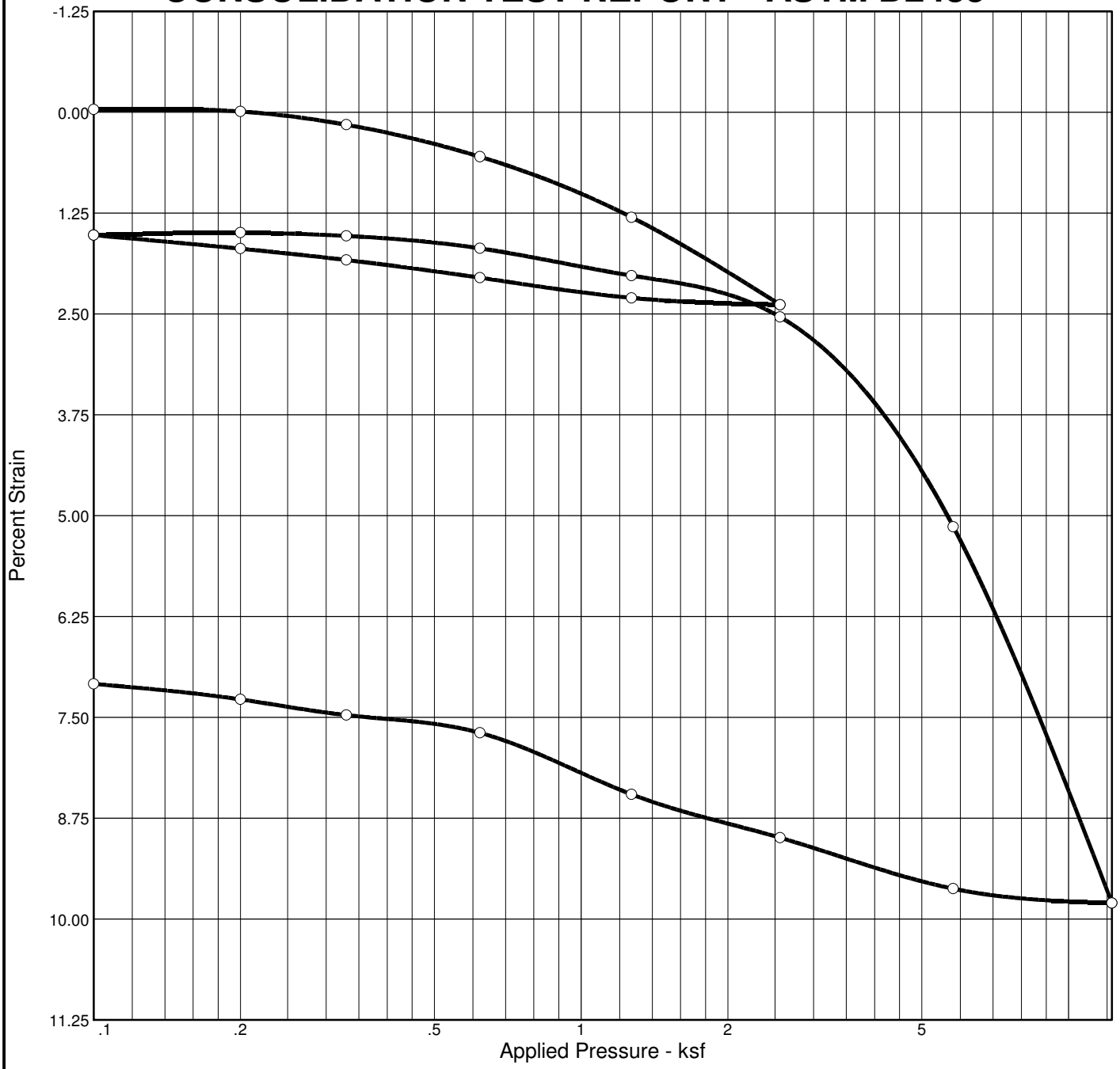
Table 3C. pH Test Results (ASTM G51)

Sample Number	Sample Depth (ft)	Sample Description	pH
S-1-3	3.0 – 4.0	Medium plasticity clayey SILT	6.2
S-5-2	2.0 – 3.0	GRAVEL and COBBLES, some sand and silt	6.0
S-8-1	2.0 – 3.0	Medium plasticity clayey SILT	6.4
S-9-3	7.5 – 8.0	Medium plasticity clayey SILT	6.1
S-16-3	4.0 – 4.5	Medium plasticity clayey SILT	6.3

Table 4C. Summary of Resistivity Testing (ASTM G57)

Location	Pin Spacing (ft.)	Resistivity (Ω-cm)
WTP Between TP-3 and TP-9	2	4,596
	4	4,596
	8	3,064
Pump Station (South of BH-7)	6	18,384
	8	22,980
	10	22,980

CONSOLIDATION TEST REPORT - ASTM D2435



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (ksf)	P_c (ksf)	C_c	C_r	Swell Press. (ksf)	Swell %	e_o
Sat.	Moist.											
84.6 %	36.5 %	77.2										

MATERIAL DESCRIPTION	USCS	AASHTO
Grey, medium plasticity, clayey SILT	MH	

Project No. 2156001-619 **Client:** Foundation Engineering, Inc.; Project #2131070-101

Project: Lebanon Water Treatment Plant

Source: 5862 **Sample No.:** SH-3-2 **Elev./Depth:** 5.0-7.0

CONSOLIDATION TEST REPORT - ASTM D2435

FEI Testing & Inspection, Inc.
Corvallis, OR

Remarks:

Figure 1C

Appendix D

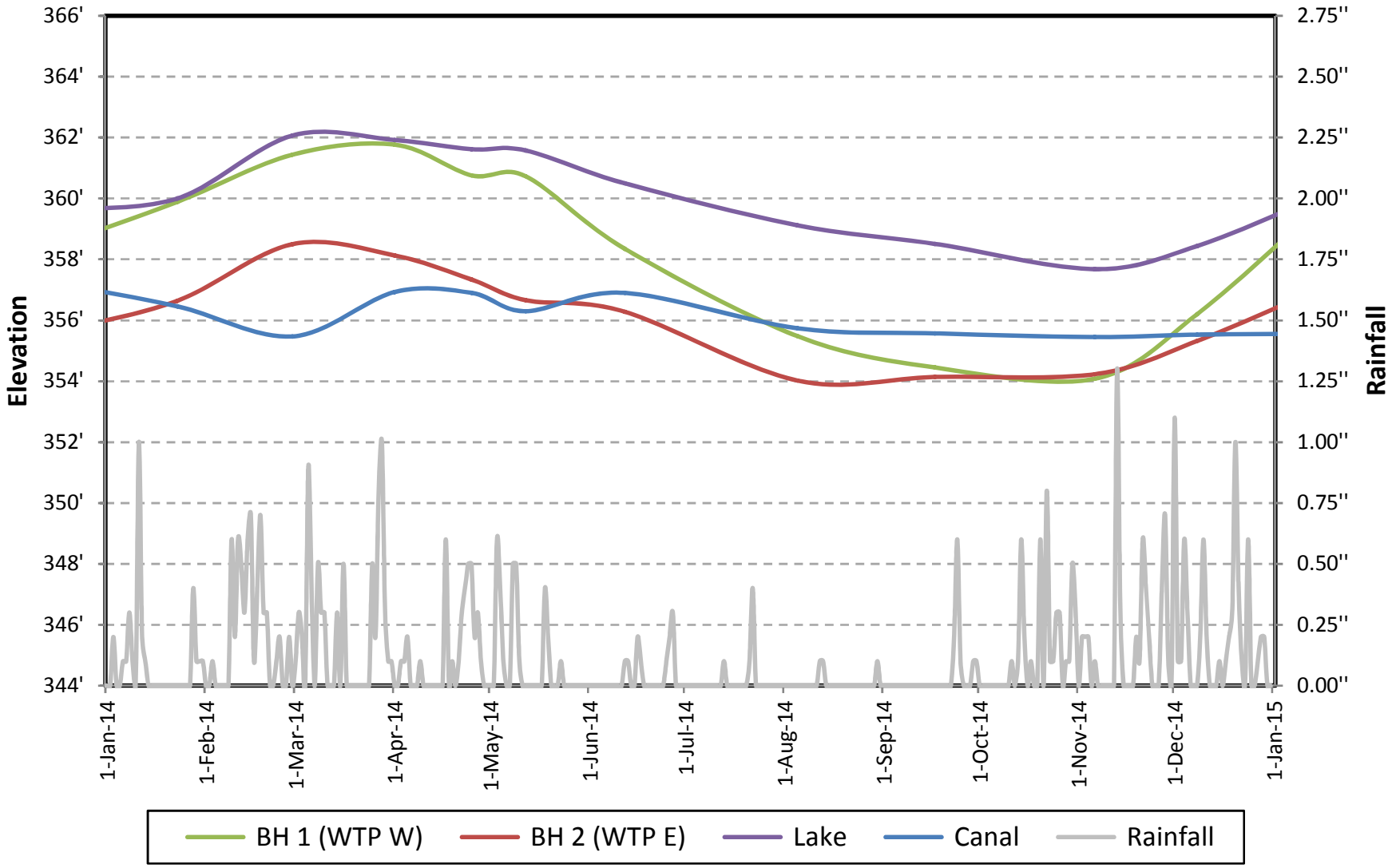
Water Level Data

Professional
Geotechnical
Services

Foundation Engineering, Inc.

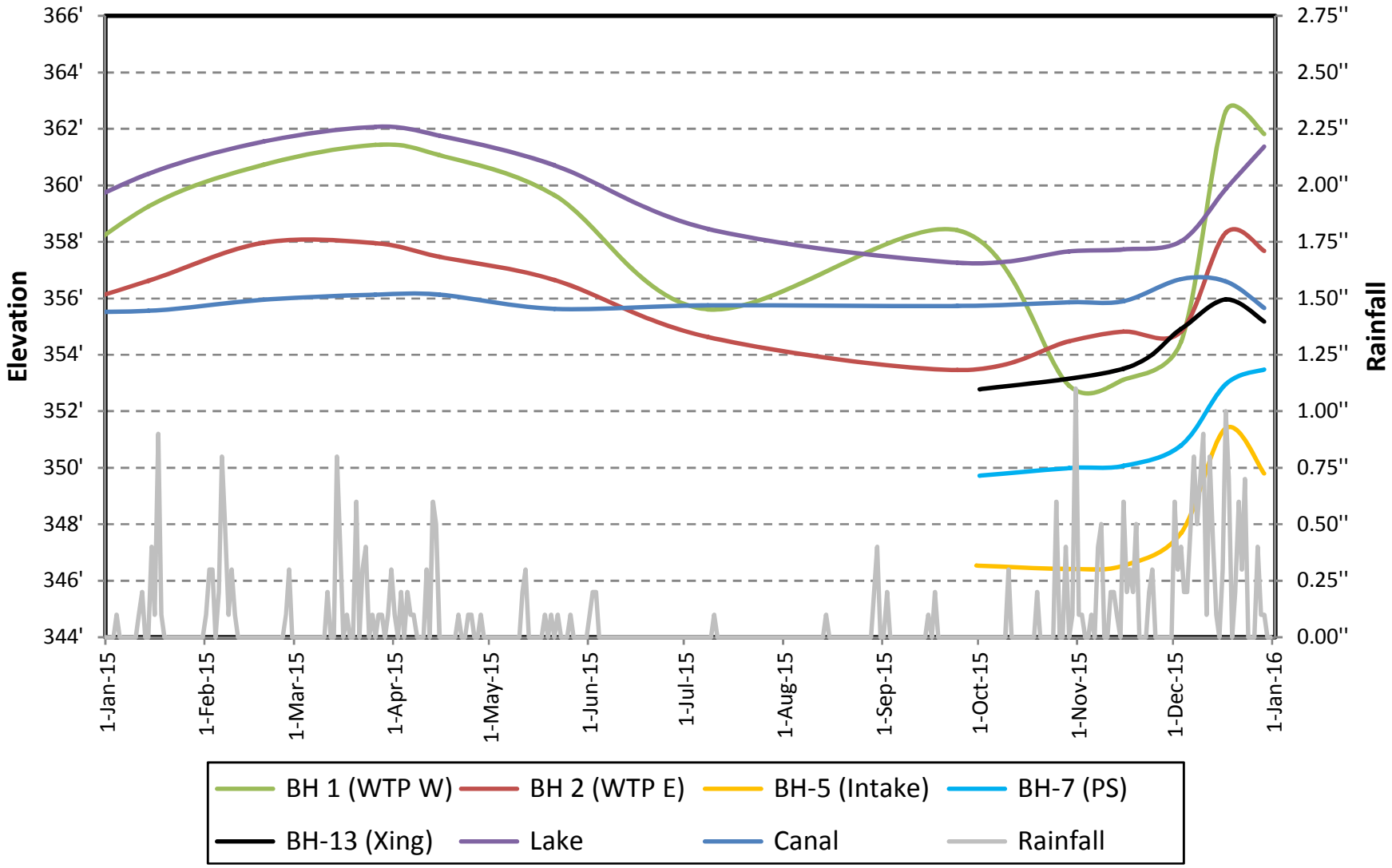
Lebanon Water Treatment Plant - Project No. 10706

Property Groundwater, Canal and Lake Levels: 2014



Lebanon Water Treatment - Project No. 10706

WTP, PS & Canal Xing Groundwater, Canal and Lake Levels: 2015



Appendix E

Seismic Hazard Study

Professional
Geotechnical
Services

Foundation Engineering, Inc.

SEISMIC HAZARD STUDY

LEBANON WATER TREATMENT PLANT
LEBANON, OREGON

INTRODUCTION

The seismic hazard study was completed to identify potential geologic and seismic hazards and evaluate the effect those hazards may have on the proposed project. The study fulfills the requirements presented in the 2014 Oregon Structural Specialty Code (OSSC), Section 1803 for site-specific seismic hazard reports for essential and hazardous facilities and major and special-occupancy structures (OSSC, 2014).

The following sections provide a discussion of the regional geology, seismic and tectonic setting, earthquakes and seismic hazards. A detailed discussion of the subsurface conditions at the project location including exploration logs is provided in the main report. Since this project contains a large facility, buildings and associated pipelines that are not all located in the same parcel, a mid-point location was used for several of the data and information searches.

LITERATURE REVIEW

Available geologic and seismic publications and maps were reviewed to characterize the local and regional geology and evaluate relative seismic hazards at the site. Local water well logs available from the Oregon Water Resources Department (OWRD) website and information from other geotechnical and seismic hazard investigations previously conducted by Foundation Engineering in the Lebanon area were also reviewed.

Regional Geology

The site is located within the southern Willamette Valley, a broad, north-south trending basin separating the Coast Range to the west from the Cascade Range to the east. Lebanon is located within the transition between the eastern Willamette Valley and the western foothills of the Western Cascades. In the early Eocene (±50 to 58 million years ago), the Willamette Valley was part of a broad continental shelf extending from the Western Cascades west beyond the present coastline (Yeats et al., 1996; Orr and Orr, 1999). Basement rock underlying most of the Willamette Valley includes the Siletz River Volcanics which erupted as part of a submarine oceanic island-arc. The thickness of the basement volcanic rock is unknown; however, it is estimated to be ±3 to 4 miles thick (Yeats et al., 1996).

The island-arc collided with and was accreted to the western margin of the converging North American Plate near the end of the early Eocene. Volcanism subsided and a fore arc basin was created and infilled (primarily to the south) with marine sediments of the Flournoy, Yamhill, Spencer and Eugene Formations throughout the late Eocene and Oligocene and terrestrial sedimentary deposits of the Fisher Formation and Little Butte Volcanics of the Oligocene (Orr and Orr, 1999).

These marine sediments typically overlie, but are also interbedded with, basalt and volcanics of the Siletz River Volcanics and younger Tertiary volcanics.

After emerging from a gradually shallowing ocean, the marine and volcanic formations were covered by the terrestrial Columbia River Basalt (CRB). The CRB poured through the Columbia River Gorge from northeastern Oregon and southeastern Washington during the Miocene (±17 to 10 million years ago) (Tolan et al., 2000), spreading as far south as Salem with some flows reaching west to the Pacific Ocean (Crenna and Yeats, 1994). Uplift and tilting of the Coast Range and the Western Cascades formed the trough-like configuration of the Willamette Valley during the late Miocene. Thick layers of Pleistocene and Holocene fluvial and floodplain deposits blanket the Columbia River Basalt and older Tertiary deposits (Orr and Orr, 1999; Tolan et al., 2000).

Catastrophic flood deposits later appeared during the Pleistocene (over 15,000 years ago) and now mantle the Willamette Valley floor as far south as Eugene (Hampton, 1972; Yeats et al., 1996). These deposits originated from a series of glacial-outburst floods that periodically drained Glacial Lake Missoula in western Montana. The older deposits, typically found within the Portland Basin, usually consist of layers of cobbles/boulders, gravel and sand during a period of time when the river(s) had sufficiently high flow to move large boulders (i.e., erratics). In the Southern Willamette Valley, turbid floodwater eventually settled, depositing a relatively thick (±50 to 100-foot) layer of silt and clay (i.e. Willamette Silt) (Orr and Orr, 1999; Wiley, 2006).

Tectonic Setting

The southern Willamette Valley lies ±130 miles inland from the surface expression of the Cascadia Subduction Zone (CSZ) (Goldfinger et al., 1992). The CSZ is a converging, oblique plate boundary where the Juan de Fuca plate is being subducted beneath the western edge of the North American continent (Geomatrix Consultants, 1995). The CSZ extends from central Vancouver Island in British Columbia, Canada, through Washington and Oregon to Northern California (Atwater, 1970). Available information indicates the CSZ is capable of generating earthquakes within the descending Juan de Fuca plate (intraplate), along the inclined interface between the two plates (interface or subduction zone), or within the overriding North American Plate (crustal) (Weaver and Shedlock, 1996). Therefore, western Oregon is located in an area of potentially high seismic activity due to its proximity to the CSZ.

Local Faulting

Numerous concealed and inferred crustal faults are located within ±10 miles of the project site (Beaulieu et al., 1974; Yeats et al., 1996; Ferns and McClaughry, 2008; McClaughry et al., 2010). However, none of these faults show any evidence of movement in the last ±1.6 million years (Geomatrix Consultants, 1995; USGS, 2006). Five potentially active Quaternary (<1.6 million years or less) crustal fault zones have been mapped within ±40 miles of the site (Geomatrix Consultants, 1995; Personius et al., 2003; USGS, 2006). These faults are listed in Table 1E.

Figure 1E shows the approximate locations of these faults. Additional fault information can be found in the literature (Personius et al., 2003; USGS, 2006; USGS, 2015).

Table 1E. Potentially Active Quaternary Crustal Faults within ±40 miles of Lebanon[1]

Fault Name	Length (miles)	Distance from Site (miles) [2]	Most Recent Estimated Deformation	Slip Rate (mm/yr)
Owl (#870)	±9	±15 W	<750,000 years	<0.20
Mill Creek (#871)	±11	±21 N-NW	<1.6 million years	<0.20
Corvallis (#869)	±25	±22 W	<1.6 million years	<0.20
Waldo Hills (#872)	±8	±25 N-NW	<1.6 million years	<0.20
Mount Angel (#873)	±19	±34 NE	<15,000 years	0.067[3]

Notes: 1. Fault data based on Personius et al., 2003, USGS, 2006 and USGS, 2015.
2. Distance from site to nearest surface projection of the fault.
3. From Table H-1 (Petersen et al., 2008).

All faults above are considered Class A faults by the United States Geologic Survey (USGS) except the Corvallis fault (Class B). Class A faults have geologic evidence supporting tectonic movement in the Quaternary, known or presumed to be associated with large-magnitude earthquakes (Personius et al., 2003). Class B faults are of non-tectonic origin (e.g. volcanic activity) or demonstrate less evidence of tectonic displacement.

Historic Earthquakes

No significant interface (subduction zone) earthquakes have occurred on the CSZ in historic times; however, several large-magnitude (>M ~8.0, where M = unspecified magnitude scale) subduction zone earthquakes are thought to have occurred in the past few thousand years. This is evidenced by recently discovered tsunami inundation deposits, combined with evidence for episodic subsidence along the Oregon and Washington coasts (Peterson et al., 1993; Atwater et al., 1995).

The Oregon Department of Geology and Mineral Industries (DOGAMI) estimates the maximum magnitude of an interface subduction zone earthquake ranges from moment magnitude (M_w) 8.5 to M_w 9.0 (Wang and Leonard, 1996; Wang et al., 1998; Wang et al., 2001). The fault rupture may occur along a portion or the entire length of the CSZ (Weaver and Shedlock, 1996).

The last CSZ interface event occurred ±315 years ago (January 26, 1700) (Nelson et al., 1995; Satake et al., 1996). Turbidite deposits in the Cascadia Basin have been investigated to help develop a paleoseismic record for the CSZ and estimate

recurrence intervals for interface earthquakes (Adams, 1990; Goldfinger et al., 2012). A recent study of turbidites from the last ±10,000 years (Goldfinger et al., 2012) suggests the return period for interface earthquakes varies with location and rupture length. That study estimated an average recurrence interval of ±240 years for an interface earthquake on the southern portion of the CSZ and an average recurrence interval of ±500 to 530 years for an interface earthquake extending the entire length of the CSZ (Goldfinger et al., 2012). Older deep-sea cores were recently re-examined. The findings may indicate greater Holocene stratigraphy variability along the Washington coast (Atwater et al., 2014). This complicated variability suggests possible uncertainty in the previous turbidite correlations.

Intraplate (Benioff Zone) earthquakes occur within the Juan de Fuca Plate at depths of ±28 to 37 miles (Weaver and Shedlock, 1996). The maximum estimated magnitude of an intraplate earthquake is about M_w 7.5 (Wang et al., 2001). No intraplate earthquakes have been recorded in Oregon in modern times. However, the Puget Sound region of Washington State has experienced three intraplate events in the last ±66 years, including a surface wave magnitude (M_s) 7.1 event in 1949 (Olympia), a M_s 6.5 event in 1965 (Seattle/Tacoma) (Wong and Silva, 1998), and a M_w 6.8 event in 2001 (Nisqually) (Dewey et al., 2002).

Crustal earthquakes dominate Oregon's seismic history. Crustal earthquakes occur within the North American plate, typically at depths of ±6 to 12 miles. The estimated maximum magnitude of a crustal earthquake in the Willamette Valley and adjacent physiographic regions is about M_w 6.5 (Wang and Leonard, 1996; Wang et al., 1998; Wang et al., 2001). Only two major crustal events in Oregon have reached Richter local magnitude (M_L) 6 (the 1936 Milton-Freewater M_L 6.1 earthquake and the 1993 Klamath Falls M_L 6.0 earthquake) (Wong and Bott, 1995). The majority of Oregon's larger crustal earthquakes are in the M_L 4 to 5 range (Wong and Bott, 1995).

Table 2E summarizes earthquakes with a M of 4.0 or greater or Modified Mercalli Intensities (MM) of V or greater that have occurred within a ±50-mile radius of the project site in the last 182 years (Johnson et al., 1994; ANSS, 2015).

Table 2E. Historic Earthquakes within ±50-mile Radius of Lebanon Latitude 44.520041 and Longitude -122.887605

Year	Month	Day	Hour	Minute	Latitude	Longitude	Depth (miles)	Magnitude or Intensity
1921	02	25	20	00	44.4	-122.4	unknown	MM = V
1930	07	19	02	38	45.0	-123.2	unknown	MM = VI
1942	05	13	01	52	44.5	-123.3	unknown	MM = V
1944	03	05	13	00	45.0	-123.4	unknown	MM = V
1961	08	19	04	56	44.7	122.5	unknown	M = 4.5
1963	03	07	23	53	44.9	-123.5	29.2	M_b = 4.6
1993	03	25	13	34	45.0	-122.607	6.2	M_c = 5.6

Notes: M = unspecified magnitude, M_b = compressional body wave magnitude, M_c = primary coda magnitude, and M_L = local Richter magnitude

It should be noted that seismic events in Oregon were not comprehensively documented until the 1840's (Wong and Bott, 1995). According to Wong and Bott (1995), seismograph stations sensitive to smaller earthquakes ($M_L \leq 4$ to 5) were not implemented in northwestern Oregon until 1979 when the University of Washington expanded their seismograph network to Oregon. Prior to 1979, few seismograph stations were installed in Oregon. OSU (Corvallis) likely had the first station installed in 1946 (Wong and Bott, 1995). The local Richter magnitude (M_L) of events occurring prior to the establishment of seismograph stations have been estimated based on correlations between magnitude and MM intensities. Some discrepancy exists in the correlations.

Table 3E summarizes distant strong earthquakes felt in the Lebanon area (Noson et al., 1988; Bott and Wong, 1993; Stover and Coffman, 1993; Dewey et al., 1994; Wong and Bott, 1995; Black, 1996; Dewey et al., 2002). None of these events caused significant, reportable damage in Lebanon.

Table 3E. Distant Earthquakes Felt in the Lebanon Area

Earthquake	Modified Mercalli Intensities (MM)
2001 Nisqually, Washington	II - III
1993 Scotts Mills, Oregon	IV - V
1981 Elk Lake, Washington	II - IV
1965 Seattle – Tacoma, Washington	I - IV
1962 Portland, Oregon	I - IV
1961 Lebanon/Albany, Oregon	IV
1949 Olympia, Washington	V
1873 Crescent City, California	V

Seismic Hazards

Section 1803.7 of the 2014 OSSC requires the evaluation of risks from a range of seismic hazards, including ground motion amplification, ground rupture, earthquake-induced landslides, liquefaction and lateral spread, and tsunami/seiche.

We have developed conclusions regarding the seismic hazards based on our on-site explorations, the soil/bedrock profiles encountered in local water well logs, our knowledge of the site geology, and a review of previous geotechnical and seismic studies performed in the area, and available geologic hazard maps.

DOGAMI has completed geologic and seismic hazard studies which include Linn County and the City of Lebanon (Burns et al., 2008). DOGAMI also provides online hazard information through HazVu, LiDAR and SLIDO (Madin and Wang, 1999; Burns et al., 2008; DOGAMI, 2015c; DOGAMI, 2015b; DOGAMI, 2015a). These studies

are only a guide and do not have precedence over site-specific evaluations. In the following sections, information from the available seismic hazard maps is provided along with our site-specific evaluations for comparison.

The relative earthquake hazard is based on the combined effects of ground shaking amplification and earthquake-induced landslides with a range in hazard from Zone A (highest hazard) to Zone D (lowest hazard). Based on the DOGAMI mapping, the entire project is located within Zone C (low to intermediate hazard) for the overall, relative earthquake hazard (Madin and Wang, 1999).

Ground Motion Amplification

Ground motion amplification is the influence of a soil deposit on the earthquake motion. As seismic energy propagates up through the soil strata, the ground motion is typically increased (i.e., amplified) or decreased (i.e., attenuated) to some extent.

The planned WTP site is underlain by a relatively thin veneer of silt and sand underlain by a deep deposit of dense to very dense sandy gravel. Similarly, the intake and pump station sites are underlain by sand and gravel (fill) over dense to very dense alluvial sandy gravel. Bedrock underlies the gravel at depths of ±28 to 32 feet. Similar soil profiles are anticipated along the associated pipeline routes.

Based on the stiffness and density of the alluvial soil and the relatively shallow bedrock depth, it is our opinion the amplification hazard is low to moderate and is consistent with an OSSC/IBC Site Class C soil profile at the intake site and a Site Class D at the WTP and pump station sites. Burns et al. (2008) indicates the amplification susceptibility for the site is moderate (NEHRP Site Class C).

We estimated a peak bedrock acceleration of ±0.36g for the project area based on modified (risk-targeted) USGS 2008 maps with a 1% probability of exceedance in 50 years (i.e., a 4,975-year return period) (USGS, 2014). The PGA at the ground surface at the intake site (±0.37g) was calculated by multiplying the PGA on bedrock by an F_{pga} factor of 1.04, interpolated from Table 11.8-1 of ASCE 7-10 (ASCE, 2010) assuming a Site Class C. The PGA at the ground surface at the WTP and pump station sites (±0.41g) was calculated by multiplying the PGA on bedrock by an F_{pga} factor of 1.13, interpolated from Table 11.8-1 of ASCE 7-10 (ASCE, 2010) assuming a Site Class D. The HazVu website (DOGAMI, 2015b) also indicates very strong ground shaking.

Ground Rupture

The risk of ground rupture is expected to be low due to the lack of known faulting beneath the site . However, hidden and/or deep-seated active faults could remain undetected. Additionally, recent crustal seismic activity cannot always be tied to observable faults. In the event of a catastrophic earthquake with a large seismic moment, inactive faults could potentially be reactivated.

Liquefaction, Lateral Spread and Settlement

The following sections provide brief discussions of the liquefaction hazard at each of the component sites.

Water Treatment Plant. Liquefiable soils typically consist of loose, fine-grained sand and non-plastic or low plasticity silt (i.e., silt with a plasticity index (PI) less than 8) below the ground water table. The WTP site is typically capped with medium stiff to stiff silt and clayey silt, which are not considered liquefiable due to their stiffness and plasticity. Additionally, the site is underlain by predominantly medium dense to very dense sandy gravel and cobbles that are not considered liquefiable due to their density and coarse gradation.

Relatively thin (±1 to 4.5-foot thick) lenses of medium stiff sandy silt and loose to medium dense silty sand were observed between the upper fine-grained soil and the gravel stratum in test pits at scattered locations across the WTP site, including the footprints of the water storage tank, the solids lagoons, future UV reactors and the west side of the process building depths. Medium dense silty sand lenses were also noted from ±32 to 36 feet in BH-3 and from ±18.5 to 24.8 feet in BH-4.

The loose to medium dense silty sand is potentially susceptible to liquefaction, provided it is saturated at the time of a large earthquake. However, based on the limited thickness of the potentially liquefiable soil, we believe the surface settlement, if any, associated with liquefaction would be modest (±1 inch or less). The site is relatively flat and there is no indication of a continuous liquefiable sand layer. Therefore, the risk of lateral spread is considered very low.

Intake Structure. The intake site is capped with medium dense silty sand underlain by very dense sandy gravel and bedrock. The foundation excavation for the structure will remove the silty sand and the structure will bear on very dense sandy gravel. Therefore, the risk of liquefaction impacting the intake structure is negligible. The structure is located on a slope. However, based on the absence of liquefiable soils, the risk of lateral spread is also considered negligible.

Raw Water Pump Station. The pump station site is capped by ±8 to 11 feet of loose silty sand and sand underlain by very dense sandy gravel and bedrock. Construction of the wet well and associated underground piping will require a large excavation extending below the sand layer beneath the center of the pump station building area. To provide uniform support for the building's shallow foundations, we recommend excavating any loose sand that extends below the building footprint and placing it back in compacted lifts or replacing it with imported granular fill. The trenches for the raw water transmission lines entering and exiting the site will also bypass the loose sand. Therefore, the liquefaction risk for the pump station building and associated piping is negligible.

Loose sand will remain beneath the pavements and drainage swales outside the building area. Based on the available ground water data, we anticipate the loose sand stratum is above the ground water level in the summer. However, comparing

the available ground water data at the pump station site to the seasonal data from the WTP site and the estimated 100-year flood level on the river, we anticipate ground water could rise as high as ±El. 361 at the pump station site during the 100-year flood. This ground water elevation would extend ±6 to 8 feet above the bottom of the loose sand stratum at the site. Therefore, it is possible liquefaction could occur in the lower portion of the loose sand stratum, if a large earthquake struck in the winter or spring during a period of extended heavy rainfall and/or flooding. Based on the thickness of the saturated soil, the associated liquefaction-induced settlement could be ±1 to 3 inches, depending on the water ground level and level of ground shaking. The site is relatively flat. Therefore, the risk of lateral spread is considered negligible.

Because the liquefaction risk is limited to the pavements and drainage swales, we assume mitigation will not be necessary and these areas will be repaired, if needed, following an earthquake.

Transmission Lines. Loose sand was also encountered in the borings along the alignments for the raw water transmission line and the finished water and sewer mains. The sand is likely above the ground water level through much of the year. Therefore, we anticipate an earthquake would have to occur coincident with a period of extended heavy rainfall and/or flooding, when the ground water level is elevated, to develop saturated conditions in the potentially liquefiable sandy soil. Even then, it is likely saturated soil conditions will be limited to the lower portion of the sand stratum.

The trenching for the sewer main is expected to extend through the loose sand stratum. The liquefaction risk for this component is considered negligible. Depending on location, the trenching for the shallower finished water main and raw water transmission line is expected to remove most or all of the loose sand. Where loose sand remains below the bottom of the trench, the compaction of the pipe bedding material and trench backfill is likely to densify a portion of the sand, leaving a relatively thin layer of loose sand. Therefore, we anticipate the liquefaction risk is relatively low and the liquefaction-induced settlement, if any, is expected to be less than ±1 inch. The transmission line alignments are relatively flat. Therefore, the risk of lateral spread is considered negligible.

Landslides and Earthquake-Induced Landslides

The proposed WTP and pump station sites and associated pipelines are located in a relatively flat terrain. The intake site is located in sloping terrain on south of the South Santiam River, but the site is underlain by predominantly dense to very dense sandy gravel over bedrock. We did not observe any landslide topography or slope instability features on site. Based on our observations, we believe the risk of landslides or earthquake-induced landslides is low. DOGAMI also indicates a low landslide susceptibility with no mapped landslides or slope instability features at the site (Burns et al., 2008; DOGAMI, 2015c; DOGAMI, 2015b).

Tsunami / Seiche

Tsunami inundation is not applicable to this site since Lebanon is not on the Oregon Coast. Seiche (the back and forth oscillations of a water body during a seismic event) is also not a concern due to the absence of large bodies of water near the site.

SEISMIC DESIGN

Design Earthquakes

The 2014 OSSC, Section 1803.3.2.1, requires the design of structures classified as essential or hazardous facilities, and major and special-occupancy structures address, at a minimum, the following earthquakes:

Crustal: A shallow crustal earthquake on a real or assumed fault near the site with a minimum M_w of 6.0 or the design earthquake ground motion acceleration determined in accordance with the 2014 OSSC Section 1613.

Intraplate: A deep subduction earthquake (Benioff Zone earthquake) with a minimum M_w of 7.0 on the seismogenic part of the subducting plate (Juan de Fuca) of the CSZ.

Interface: A subduction earthquake with a minimum M_w of 8.5 on the seismogenic part of the interface between the Juan de Fuca and the North American Plates on the CSZ.

The design maximum considered earthquake ground motion maps provided in the 2014 OSSC are based on modified (risk-targeted) 2008 maps prepared by USGS for an earthquake with a 1% probability of exceedance in 50 years (i.e., a ±4,975-year return period) for design spectral accelerations. The modifications include factors to adjust the spectral accelerations to account for directivity and risk.

The interactive deaggregation search tool on the USGS National Earthquake Hazard Mapping website indicates the seismic hazard at the project site is dominated by the CSZ. Crustal sources were included in the development of the spectral acceleration maps, but crustal earthquakes are estimated to comprise less than 10% of the total hazard at the site.

The following earthquake mean magnitudes and source-to-site distances were listed as principal sources for the USGS spectral acceleration maps (USGS, 2008):

Crustal: A shallow, ±M_w 6.1 crustal earthquake located ±5.5 miles from the site, representing ±8.5% of the hazard.

Intraplate: A 50-km deep, ±M_w 6.9 intraplate subduction earthquake located ±36 miles from the site, representing ±25% of the hazard.

Interface: A M_w 8.3 to 8.7 floating interface subduction earthquake located ±50 miles from the site, representing ±17% of the hazard and a M_w 9.0 megathrust earthquake located ±51 miles from the site, representing ±47% of the hazard.

The earthquake magnitudes and source-to-site distances used to generate the 2008 USGS maps satisfy the requirements of 2014 OSSC. Seismic design parameters and an OSSC (2014) design response spectrum are discussed in the Seismic Design section of the main report and shown on Figures 8A and 9A (Appendix A).

CONCLUSION

Based on the findings presented herein, the seismic hazard is limited to a modest liquefaction risk, which can be addressed. It is our opinion there are no major geologic or seismic hazards that would preclude the design and construction of the proposed water treatment plant and associated facilities. This site-specific hazard investigation for the Lebanon WTP Final Design project in Lebanon, Oregon, was prepared by Brooke Running, R.G., C.E.G.

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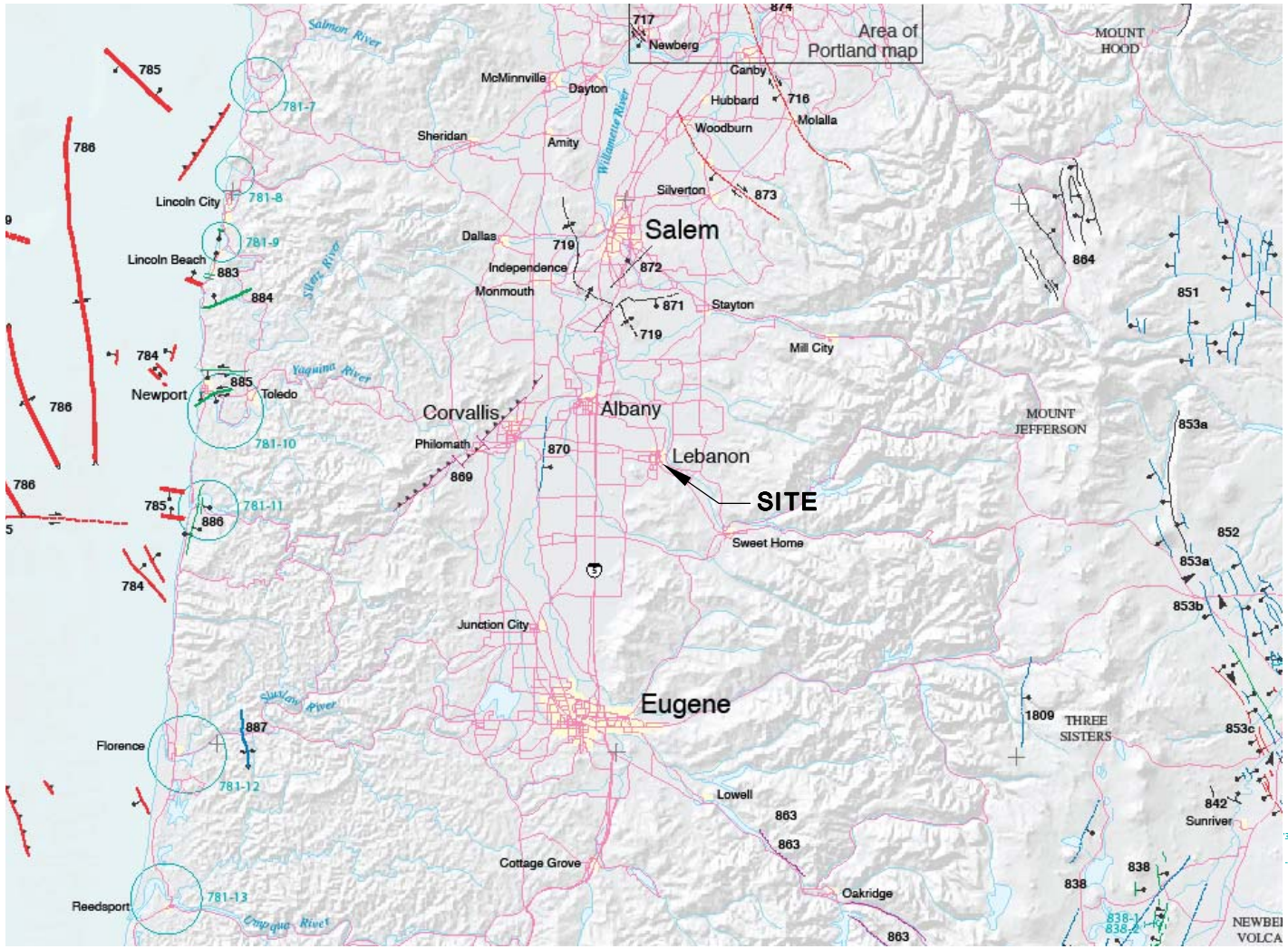

NOT TO SCALE

LEGEND

TIME OF MOST RECENT SURFACE RUPTURE
- Holocene (<10,000 years) or post last glaciation (<15,000 years; 15 ka); no historic ruptures in Oregon to date
- Late Quaternary (<130,000; post penultimate glaciation)
- Late and middle Quaternary (<750,000 years; 750 ka)
- Quaternary, undifferentiated (<1,600,000 years; <1.6 Ma)
- Class B structure (age or origin uncertain)

SLIP RATE
- >5 mm/year
- 1.0-5.0 mm/year
- 0.2-1.0 mm/year
- <0.2 mm/year

TRACE
- Mostly continuous at map scale
- Mostly discontinuous at map scale
- Inferred or concealed

STRUCTURE TYPE AND RELATED FEATURES
- Normal or high-angle reverse fault
- Strike-slip fault
- Thrust fault
- Anticlinal fold
- Synclinal fold
- Monoclinal fold
- Plunge direction of fold
- Fault section marker

DETAILED STUDY SITES
- Trench site
- Subduction zone study site

CULTURAL AND GEOGRAPHIC FEATURES
- Divided highway
- Primary or secondary road
- Permanent river or stream
- Intermittent river or stream
- Permanent or intermittent lake

NOTES:
1. PORTION OF MAP BASED ON MAP OF QUATERNARY FAULTS AND FOLDS IN OREGON (PERSONIUS ET AL., 2003).
2. SEE SITE SPECIFIC HAZARD STUDY FOR A DISCUSSION OF LOCAL FAULTING.
3. FAULTS: #869= CORVALLIS; #870= OWL; #871= MILL CREEK; #872= WALDO HILLS; AND #873= MOUNT ANGEL.

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REVIS.
PROJECT NO. 2131070-101

QUATERNARY CRUSTAL FAULT MAP
CENTRAL WILLAMETTE VALLEY
LEBANON WATER TREATMENT PLANT
LEBANON, OREGON

FIGURE NO.
1E

FILE NAME:FIGURE 1D