Geotechnical Evaluation
Proposed Tonquin Employment Area
Sanitary Sewer Upgrade
Sherwood, Oregon

Prepared for:
City of Sherwood

January 16, 2013
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For

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1.0 Introduction

This report presents Ash Creek Associates, a Division of Apex Companies, L.L.C. (Ash Creek's), geotechnical exploration and recommendations for the proposed Tonquin Employment Area Sanitary Sewer Upgrade in the City of Sherwood, Oregon. The significant aspects of this report have been arranged in the following manner:

- Summary;
- Project understanding;
- Subsurface conditions;
- Engineering conclusions; and
- Appendix addressing field explorations.

2.0 Purpose, Scope, and Project Understanding

2.1 Purpose of Work

The purpose of our work was to provide geotechnical engineering recommendations for the design and construction of the proposed utility. Our recommendations include the following:

- Descriptions of subsurface conditions along the proposed alignment;
- General soil-related recommendations for the project;
- Excavation shoring pressures;
- Open trench and trenchless utility installation; and
- Excavation Dewatering.

2.2 Scope of Services

Our scope of services for this project included the following:

- Surficial reconnaissance;
- Subsurface explorations;
- Environmental analyses;
- Geotechnical engineering analyses; and
- Preparation of this report.

3.0 Limitations of Our Work

This work was performed for the exclusive use of the City of Sherwood, their clients, and their consultants for specific application to this project and site. We performed this work in accordance with generally accepted professional practices in the same or similar localities related to the nature of the work accomplished at the time the services were performed. No other warranty, express or implied, is made.

4.0 Project Understanding and Site Description

Ash Creek's understanding of the project is based upon discussions with the City of Sherwood project engineer and project bid documents. The project consists of upgrading an existing sanitary sewer line to the southeast of the Portland and Western Railroad between Rock Creek to the west and SW Tualatin Sherwood Road to the east. The sewer will be installed through a combination of pipe-bursting and open trenching.

Key geotechnical issues pertaining to the site include trench stability, trench moisture, groundwater level, and the soil strength for pipe-bursting operations. In addition, the proposed alignment crosses a wetland on the southwestern portion of the project area adjacent to Rock Creek. Therefore, geotechnically related issues pertaining to soft, organic, wetland soils and high groundwater will also be evaluated.

According to available information, the property owned by Washington County to the west of Rock Creek has chromium contamination from previous operations on the site. In addition to the geotechnical scope of services, an environmental assessment of the soils at the southwestern end of work area was conducted to determine if contaminated soils are present in the proposed areas of pipe bursting and temporary access road construction.

The site location is indicated on the attached Figure 1. The alignment from SW Tualatin-Sherwood Road to Rock Creek appears to be approximately 1,400 to 1,500 feet in length.

5.0 Geologic Setting

5.1 Geologic Overview

The Portland Basin area was formed by a series of geologic events that included: Cascades Mountains building from a series of large fissures in the earth that emitted the Columbia River Basalt (CRB); torrential erosion of the Cascade Range, generating alluvial deposits now identified as the Troutdale Formation; a second episode of volcanism resulting in a series of Boring Lava volcanic eruption centers; vast deposits of wind-blown silt (loess) – termed the Portland Hills Silt – derived from denuded glacial plains to the east; a series of cataclysmic glacial floods generated in Montana and Idaho that scoured the lowland loess, alluvial

plains, and volcanic cones; and more recent stream and river erosion and sedimentation that has shaped the lowlands as they appear today. Following is a more in-depth description of the deposits and geologic events.

Alluvium. Office review of the geologic studies for the project vicinity indicate that the surficial soil deposits on the lowland areas adjacent to Rock Creek on the southwestern side of the project area consist of unconsolidated alluvium derived from river and stream deposits at the ground surface. Available resources indicate that these materials consist of clay and or peat soils.

Willamette Silts. The Portland Basin was filled with slackwater clayey silts from a series of floods spanning 10,000 to 34,000 years before present (BP; Missoula or Spokane Flood Deposits), commonly known as the Willamette Silt Formation. The Willamette Silts extend the entire length of the Willamette Valley from Eugene to Portland. These deposits are generally found below 350 feet in elevation (El.), but some areas in the southern valley are thought to have exposures up to El. 400.

Missoula (Montana) or Spokane Flood Deposits. The areas of the Portland Basin below about El. 350 are overlain by a series of Pleistocene-aged (ending about 14,000 years ago) catastrophic flood gravels deposited throughout the area (including Vancouver and Lake Oswego) during the great Missoula (or Spokane) floods. The flood deposits consist of large cobbles and boulders near the source areas of maximum velocity, such as the mouth of the Columbia Gorge. The more slackwater deposits of the same flooding events consist of coarse- to fine-grained sands and slightly silty sands. The grain size decreases with distance from the Columbia River Gorge constriction. Typically, these materials are underlain at relatively shallow depths (50 to 60 feet below ground surface [bgs]) by the fine-grained equivalent of the Troutdale Formation. In the vicinity of the site, available resources indicate that coarse flood deposits are present on the upland portions of the alignment and consist of sand, gravel, and boulders at thicknesses of up to 50 feet.

Portland Hills Silt. Between the glacial floods, vast clouds of Aeolian – or windblown – silt (loess) were swept off the Columbia River flood plain and deposited on the hillsides and valleys in the Tualatin Mountains and drainages. This silty aeolian deposit remains above approximately El. 350 and has been named the Portland Hills Silt Formation, aged approximately 34,000 to 700,000 years BP.

Boring Lava. A series of extinct Pliocene- to Pleistocene-aged (5.3 to 0.6 million years BP) volcanic cones (Mt. Tabor, Rocky Butte, Mt. Sylvania, etc.) are present throughout the area. The ash, cinder, and lava flows derived from them have been termed the Boring Lava. They are basaltic in composition and are chemically similar between the flows and eruptive vents in the Portland area. Most of the Boring Lava volcanic centers within the Portland Basin have erupted through the CRB and the conglomerates and sandstones of the Troutdale Formation. In many cases, remnants of the Troutdale Formation have been uplifted onto the flanks of the volcanoes and occasional Troutdale Formation pebbles and cobbles can be found within the Boring Lava.

Troutdale Formation. During the Tertiary (Lower Pliocene) Age, cataclysmic uplifting of the CRB, high-velocity floods, and debris flows washed materials into the Portland Basin. The Troutdale Formation is on the order of 5 to 7 million years old and is composed of a variety of sedimentary materials including unconsolidated sand and gravel, cemented gravels and cobbles (conglomerate), and hard sandstones. The formation is in excess of 1,000 feet thick in places and the surface is often deeply weathered to fine-grained sandy silts and clayey silts.

Columbia River Basalt. The Portland metropolitan area is underlain at depth by the CRB, a Tertiary Age series of lava flows as much as 25 million years old. The CRB has been down-warped into the Portland and Tualatin Basins as much as 1,200 to 1,500 feet deep; the basins have subsequently been filled with a fine-grained sedimentary lake deposit termed the Troutdale Formation. Around the basin margins, the CRB is exposed at or near the ground surface. In the vicinity of the project site, available resources indicate that the weathered surface of the CRB is present below the coarse surficial flood gravels.

6.0 Subsurface Conditions

The field explorations for this project were conducted on August 9, 2012. The explorations consisted of a surficial reconnaissance and the drilling of three mud-rotary borings on the Columbia Cascade Distribution, Inc. (Columbia Cascade) property to the southeast of the Portland and Western Railroad. Prior to using mud rotary methods, hollow-stem auger methods were attempted in order to more accurately characterize groundwater conditions on the site; however, shallow auger refusal was encountered on our early attempts and therefore mud rotary methods were used. The approximate boring locations have been indicated on the accompanying Site Exploration Plan (Figure 2).

In addition to the mud rotary borings on the upland portion of the proposed sewer alignment, several shallow hand auger explorations (0 to 6 feet below the ground surface [bgs]) were conducted in the wetland areas on the southwestern portion of the project area adjacent to Rock Creek. These borings were advanced for the purpose of environmental sampling for Chromium in soil, as well as for geotechnical soil characterization in pipe bursting access/egress locations. In addition, three surface soil samples (0 to 1 foot bgs) were conducted along the alignment of a proposed construction access road to the southeast of the proposed sewer alignment through the wetland. The approximate boring locations have been indicated on the accompanying Site Exploration Plan (Figure 2).

Logs of the mud rotary and hand auger subsurface explorations have been included in Appendix A of this report. The attached logs describe soils and various engineering properties of soils encountered during exploration. Descriptions are based upon *in situ* testing, laboratory testing, and field classification of soil samples. A summary of subsurface conditions encountered within our borings has been provided below.

6.1 Upland Subsurface Conditions

Soil conditions were broadly uniform between borings B-1 through B-3 on the upland portion of the site. A summary of soil conditions observed in our borings is provided below.

Surface Fills. Up to approximately 4.5 feet of fill was encountered in our borings. Fill was encountered in Borings B-2 and B-3 on the Cascade Columbia property and consisted primarily of sand with gravel. Based on conversations with facility personnel, anecdotal descriptions indicate that substantial fill construction was used in the development of the Cascade Columbia property due to the difficulty in excavation of the native soils on the parcel. Fill soils consisted of: loose to medium-dense, dry to slightly moist, brown, gravel in a silty sand matrix or silty sand with gravel. On the east portion of the alignment near Tualatin-Sherwood Road, the surface fills were not encountered.

Gravel, Cobbles, and Boulders. Below depths of approximately 4.5 feet bgs, native soils consisted of dense to very dense gravel, cobbles, and boulders. This unit varied somewhat across the three boring locations on the Cascade Columbia property. The westernmost boring (B-3) consisted uniformly of very dense cobbles and boulders. The central boring (B-2) also consisted of very dense boulders and cobbles, but was interbedded with soft to stiff sandy silt units of up to 8 feet thick. The easternmost boring (B-1) consisted of very dense gravel with fewer boulders and cobbles, but was also interbedded with soft to stiff sandy silt units of up to 3 feet thick. On the east portion of the alignment near Tualatin-Sherwood Road, this unit was present below the shallow topsoil. At a depth of approximately 28 feet in borings B-1 and B-2, this unit was underlain by a bedrock unit, most likely the weathered surface of the Columbia River Basalt formation.

Groundwater. Due to the dense subsurface conditions, mud rotary drilling was employed; therefore, seepage and groundwater conditions could not be accurately determined at the time of drilling. Following exploration, water levels in boring B-2 were allowed to equilibrate for a period of approximately 8 hours prior to borehole abandonment. Static groundwater was observed in this boring was approximately 15.8 feet bgs.

The groundwater depth described above was from the time of drilling. Steady-state water levels encountered during trench excavations during late winter and spring months could be several feet higher than those recorded during drilling.

Based upon our review of the proposed sewer line plans, it appears that the proposed excavations will be up to 25 feet below existing surface grades. This will place the trenches and manhole excavations within the groundwater table for most of the project. Based upon our experience with similar soils, excavations into unconsolidated material below the groundwater table will generally lead to extreme sloughing of the trench, resulting in large material overruns and potentially unsafe conditions. We recommend that the site be dewatered to a minimum of 2 feet below proposed excavations.

6.2 Lowland Subsurface Conditions

Soil conditions were broadly uniform between borings HA-1 and HA-2 (as well as surface soil samples) in the wetland adjacent to Rock Creek on the southwestern portion of the site. A summary of soil conditions observed in our explorations is provided below.

Organic Silt and Clay. In the shallow subsurface within the project area, native soils typically consist of multiple feet of soft to very soft, highly organic, wet, fine-grained soils including silt and clay units. These soil units were dark brown to brownish-black in color, and were found in interbedded lenses extending from the surface to the maximum explored depth of approximately 6 feet bgs. In our explorations, the silts and clays became medium stiff at a depth of 3 to 3.5 feet bgs.

These wetland soils present numerous special challenges in construction. These include a very high level of compressibility under even light and moderate loads, poor to impassable "traversability" characteristics by both wheeled and tracked construction equipment, and very shallow groundwater (and/or surface water).

Groundwater. Groundwater was present at the ground surface in all of our lowland explorations, with up to 2 feet of surface water noted during our reconnaissance to be present across the wetland area. As such, the proposed access road and pipe burst access/egress locations in this wetland area were inundated with water at the time of our reconnaissance and explorations. These groundwater depth observations were made during late summer 2012. During the wet season (late fall through late spring) the groundwater and surface water levels may be expected to rise above the elevations observed at the time of our explorations.

7.0 Chemical Analyses, Results and Screening

For the purpose of environmental characterization of soils within the wetland on the southwestern portion of the site adjacent to Rock Creek, Ash Creek advanced two hand auger borings (HA-1 and HA-2) in locations where excavation is planned for access/egress of the existing pipeline for deployment of pipe bursting equipment. Soil samples were collected from the 0-2 foot bgs and 2-6 foot bgs intervals in each hand auger boring. In addition, surface soil samples (0-1 foot depth) at three locations within the footprint of the proposed temporary construction access road.

Chemical Analyses. Chemical analyses were performed to determine the magnitude of potential chemicals of interest (COI) in the soil on the southwestern portion of the proposed sewer alignment. Sediment samples were containerized and transferred to Apex Laboratories of Tigard, Oregon, by chain-of-custody for chemical analysis.

Based on historical activities and environmental cleanup conducted on the adjacent property to the west of Rock Creek, chromium was determined to be a constituent of interest (COI) for this project. Samples were

analyzed for total chromium using EPA 6000/7000-series methods. Laboratory analytical reports, chain-of-custody documentation, and a quality assurance review are provided in Appendix B.

Results and Screening. Soil analytical results are listed in the following Table 1. Concentrations of chromium in soil were compared to Oregon Department of Environmental Quality (DEQ) Clean Fill Screening Criteria (Clean Fill) for Unrestricted Upland Disposal (DEQ, 2009). The basis for the DEQ Clean Fill screening concentration for chromium is the default regional background concentration for this metal.

Table 1 – Chromium Concentrations in Soil

Boring Number & Depth of Boring (feet)	Chromium Concentration in Soil (mg/kg)	DEQ Clean Fill Screening Criteria (mg/kg)
HA-1 (0-2)	34.4	42
HA-1 (2-6)	30.1	42
HA-2 (0-2)	24.5	42
HA-2 (2-6)	25.2	42
SS-1 (0-1)	31.0	42
SS-2 (0-1)	25.9	42
SS-3 (0-1)	22.7	42

Based on the results of our soil sampling and analysis, the samples are below applicable screening levels. In addition, the chromium were broadly uniform and below regional background concentrations. Therefore, it is our opinion that the soil is suitable for unrestricted excavation and re-use, and that no special handling would be required for excavation and construction within these soils.

8.0 Conclusions and Recommendations

Our recommendations are based on our current understanding of the project. If the nature or location of the planned construction changes, Ash Creek should be contacted so we may confirm or revise our recommendations.

8.1 Excavations

Native soils may stand in near-vertical slopes for short periods of time; however, they may collapse suddenly and without warning. Precautions in excavations will be required due to the potential for caving/sloughing within native soils underlying the site. Any excavations deeper than 4 feet should be sloped or shored in accordance with Occupational Safety and Health Act (OSHA) regulations. Normally, shoring systems are contractor-designed and -installed items.

Subgrade soil at the base of excavations should be firm or dense prior to the placement of bedding or base material. The base of excavations should also be free of mud and muck, and should be sufficiently stable to remain firm and intact under the feet of the workers. Where necessary, a layer of clean gravel should be placed at the base of excavations of sufficient strength and thickness to withstand subsequent construction activity; this will require installation below the specified subgrade elevation and thus will entail additional excavation below design subgrade elevations. Base stabilization gravel should conform to Oregon Department of Transportation (ODOT) Class-B granular fill material (1-inch or 3/4-inch [minus] crushed rock material, or Class-D pit-run material, if directed).

Temporary Excavation Slopes and Shoring. In general, temporary excavation slopes may be suitable in areas where adjacent improvements, roads, utilities, or structures are not located within a horizontal distance equal to the depth of the excavation (measured from the top of the excavation). Unsupported temporary excavation slopes within native fine-grained soils or fill soils should not exceed slopes of 1H:1V. Actual slopes used during construction should be determined by the contractor on a case-by-case basis.

It is possible that cantilever or braced shoring may be necessary to limit the excavation area, and we have provided recommendations for the design of this shoring.

Shoring Deflection. Numerous studies have shown that shoring pressures are directly related to lateral movement of the shoring. An average lateral deflection at the top of a wall of approximately 1/1,000 of the wall height should be adequate to mobilize the internal soil strength, thereby reducing the total lateral pressure to a semi-active state of stress (conventional design approach). With this level of deflection, the stress is distributed in a roughly parabolic shape, normally approximated as a rectangle. If lateral deflections are allowed to increase to within the range of 1/150 to 1/75 of the wall height, the pressure distribution starts to become triangular, with the greatest stress at the bottom of the wall. Quality construction procedures usually result in shoring deflections less than this.

Vertical deflections (settlements) immediately behind the wall may approach 2 to 3 inches, with settlements dissipating further from the wall. This assumes that good construction procedures are used. If unfilled voids are left behind the wall, or if walls are allowed to slough or cave before lagging is installed, the settlements can be far greater.

Design Shoring Pressures. Cantilever shoring should be designed for a triangular lateral earth pressure derived from an equivalent fluid weight of 31 pounds per cubic foot (pcf). Braced excavations should be designed for a uniform (rectangular) lateral earth pressure of H X 26 pcf, where H is the depth of the excavation in feet. These shoring pressures represent our best estimate of actual pressures that may develop against the shoring and do not contain a factor of safety. Adequate factors of safety must be incorporated in the design method.

These design pressures do not include seismic effects due to the low probability of a major seismic event occurring during the relatively short construction period.

Soldier Piles. Soldier piles must be designed for bending, vertical loads, and for passive kick-out at the pile toe. Toe kick-out can be resisted by passive pressure against the base of the pile. For a horizontal ground slope at the base of the wall, passive pressures may be designed as a 500-pcf equivalent fluid weight. In the case of isolated soldier piles (center-to-center spacing greater than three pile diameters), these pressures may be applied to a width equal to three pile diameters. This pressure is our best estimate of actual pressures that can be developed and does not contain a factor of safety. We recommend using a safety factor of at least 1.5 in design against kick-out.

Lagging. Lagging is not normally structurally designed, and thicknesses are usually based on empirical values. Soil arching at the soldier pile will substantially reduce stresses on the lagging. If lagging is to be designed, we recommend design pressures be the same as those already presented except that, due to arching, no stress need be included on the portion of lagging that is within one pile diameter of the soldier piles.

Excavation Backfill. In order to minimize the potential for post-construction backfill settlement, we recommend that all backfill within road alignment areas or other settlement-sensitive areas consist of clean, imported granular fill. We recommend that backfill material conform to ODOT specifications for Class-B granular fill material (1-inch or 3/4-inch [minus] crushed rock material).

Compaction from excavation subgrade to within 18 inches below the bottom of the pavement base section should be a minimum of 92 percent of the material's maximum dry density as determined by ASTM D 1557 or AASHTO T-180. Compaction of the final 18 inches of backfill should be at least 95 percent of the material's maximum dry density as determined by ASTM D 1557 or AASHTO T-180.

Issues associated with meeting compaction specifications are most often directly associated with lift thickness, compaction equipment being employed, and application of moisture to the backfill material. Addition of water to dry granular backfill prior to its placement into excavations, use of heavy vibratory plate compactors, as well as maintaining lift thickness to less than 8 or 16 inches (thickness dependent upon actual compaction method), will typically result in satisfactory backfill density and minimize issues associated with backfill settlement.

8.2 Site Preparation

We have provided recommendations for temporary haul road construction and working pad installation, as well as other geotechnical concerns and issues relative to the project site. Because of the moisture-sensitive near-surface soils in the lowland areas and the presence of shallow groundwater and surface water over much of the site, Ash Creek strongly recommends site grading and utility trenching be

conducted during dry weather conditions. Even during dry weather conditions, substantial dewatering will be required for construction of the sewer along this alignment. The optimum time for site grading and trench work generally falls between late June and late September.

Temporary Haul Roads and Working Pad Installation. Construction within the wetland areas adjacent to Rock Creek is expected to present special challenges from the perspective of equipment access. Standing water is present at the ground surface in this area.

We anticipate that access to construction areas will require use of stabilized temporary haul roads and granular working blankets. These stabilized work areas will require grid or geo-web stabilized crushed rock fills. We recommend that in areas of stabilized haul roads and working blanket construction, the topsoil layer not be stripped.

The root mat mantling the soft wetland soils will provide some level of temporary subgrade strength during the initial haul road and working blanket installation. The vegetation and grass over these areas should be mowed prior to construction of the working blanket (if possible) but actual topsoil stripping should be avoided wherever possible.

Stabilized working areas should be constructed whenever possible by lighter weight CATs and smaller trackhoes pushing material off of an advancing stabilized pad. We recommend that construction activities in this area be limited to summer time grading and site work.

8.3 Pipe Bursting

Based on our current understanding of the project, we understand that pipe bursting has been proposed for extensive sections of this project. Pipe bursting is a trenchless technique for replacing existing pipelines. Pipe bursting consists of breaking up the existing pipe, pushing aside the fractured pipe pieces, and pulling or jacking a new pipe in place. Subject to site conditions, the method can be used to maintain the existing pipe size or to install a new pipe size up to 100 percent larger than the original pipe diameter. Pipe bursting is a specialized technique that should only be performed by contractors with a demonstrated record of experience.

The soil conditions most conducive to pipe bursting are those that allow for the deflection of the burst pipe segments through compaction or consolidation of the soils. Stiff soils can result in significant surface heave or impacts to adjacent utilities. Cohesive soils can be preferable in that they will maintain the open hole formed by the pipe-bursting while the new pipe is being pulled into place; this limits the friction on the new pipe and reduces tensile stresses. As such, the most favorable soils would be soft clays, preferably above the groundwater table.

Pipe bursting below the groundwater table can be quite difficult. The pipe-bursting process can increase pore water pressures, reducing effective stresses; this can result in a "quick" condition where soil strengths are seriously reduced and the soil behaves as a fluid. The result is that pipe buoyancy forces are increased and soils in the pipe zone can flow into the pipe and into the receiving pit. It may be necessary to dewater the project for pipe-bursting processes.

This site presents a number of geotechnical challenges relative to pipe bursting. These challenges include:

- High groundwater levels (above planned pipeline invert depths and required access pits);
- Porous and permeable subsurface soils (sands and gravels);
- Dense native soils and potentially trench backfill that may limit or stop pipe bursting progress;
- Loose, flowing, granular, non-cohesive soil with little to no vertical standup time (e.g., native sands, gravels, and trench backfill) and impact on access pit shoring; and
- Large native materials (e.g., cobbles and boulders) within native soils and fills and backfills and impact on pipe bursting and access pit construction.

For this project, the existing pipeline to be replaced consists of a 10-inch concrete sewer pipe. The replacement pipeline will be 18-inch high-density polyethylene (HDPE). Burst lengths will range from 300 to 370 feet, with existing sewer invert depths ranging between approximately 5 and 30 feet below present ground surface.

The International Pipe Bursting Association (IPBA) classifies pipe bursting installations into three categories based on the depth of the existing pipe, existing and new pipe diameters, and burst length (IPBA, 2004). The IPBA Pipe Bursting Classification System is presented in the table below.

IPBA Classification	Degree of Difficulty	Depth of Pipe (feet)	Existing Pipe ID (inches)	New Pipe Diameter Compared to Existing Pipe	Burst Length (feet)
A	Minimal	<12	2-12	Size on Size	0-350
B	Moderate	>12 to <18	12-18	Single Upsize	350-500
C	Comprehensive	>18+	20-36	Double/Triple Upsize	200-1,000

We have highlighted the applicable criteria for this project. The upsize proposed for this project (10 inches to 18 inches) is considered by the IPBA to be a Triple upsize. The pipe depths, upsizing, and soil conditions (dense granular formations), and shallow groundwater all combine to result in a classification for this project of Challenging to Extremely Challenging. Pipe bursting for this project should only be attempted by contractors with a demonstrated record of experience on similar projects with similar pipe sizes, depths, and upsizing.

Even with an experienced contractor, due to the length, depth, degree of upsize, and soil conditions for this project, there is a risk of ground heave or refusal of the bursting tools (i.e., pipe-bursting equipment getting stuck part way through a reach). The project specifications should include language to the effect that the contractor will be responsible for the repair of any damages related to their choice of construction method and will be responsible for bearing the cost of switching methods should their original choice of construction method prove unsuccessful (e.g., switching from pipe bursting to conventional open-cut removal and replacement).

Pipe-Bursting Equipment. The contractor's choice of pipe replacement method and equipment should be compatible with the project requirements and subsurface conditions as described herein. It is particularly crucial that the Contractor select equipment that it capable of completing the proposed burst lengths given the subsurface conditions, depth, and upsize.

Native soils near the pipe elevation are generally dense to very dense cobbly soils. Trench backfill soils are unknown at this time.

Given the generally dense condition of native soils; the suspected presence of coarse gravel, cobbles, and boulders; the length of proposed pipeline segments; and degree of upsize proposed for several pipeline segments, only pneumatic pipe-bursting equipment should be specified for this project.

Lubricant. A primary component of the installation load applied to pipelines installed using pipe bursting methods is the friction generated between the new pipe and the burst pipe/soil matrix as the new pipe is pulled into place. In many cases, friction (and, therefore, the installation load) on the installed pipe can be reduced through the use of pipe lubricants. According to the IPBA (2004), the use of lubricants should be considered when:

- The new pipe diameter is equal to or greater than 2 times the existing pipe diameter;
- The burst length exceeds 300 feet;
- The new pipe diameter exceeds 12 inches;
- The host pipe is below groundwater;
- The ground conditions are unstable (i.e., flowing ground); and
- Recommended by the bursting equipment manufacturer.

Ultimately, all of these conditions apply to this project.

Resistance Forces. The majority of the project will be constructed within dense, cobbly soils. Passive pressures against native soils are likely to be used to develop forces necessary to accomplish the pipe bursting. For the majority of the project, a passive pressure developed from an equivalent fluid pressure of

450 pcf may be used. The exception is the westernmost pipe burst run (MH-1 to MH-2). For that run, lateral forces based on an equivalent fluid pressure of 150 pcf would be appropriate.

8.4 Dewatering

Intensive excavation dewatering will be required for trenching and excavation work. Groundwater depths encountered in August 2012 are detailed in the Subsurface Conditions Section 5.0 of this report. The depths represent groundwater levels (and surface water presence) were observed in our lowland explorations, as well as in one of our mud rotary boreholes on the upland portion of the project. Please note that the groundwater conditions observations noted during our explorations were derived from a single mud rotary borings. Due to the water-intensive nature of the mud rotary methods, the accuracy of groundwater interpretations based on these explorations is limited.

It should also be emphasized that the groundwater levels observed in our August 2012 drilling work represent seasonally low water levels. Groundwater elevations in August are typically at their yearly low point. Groundwater levels invariably rise during winter and spring months and thus can be 10 or more feet higher than water levels observed during summer months.

Regardless of the time of year in which site work is conducted, we expect that dewatering via well points along the sewer alignment upgrade will be required prior to starting excavations. Attempting excavation within the project boundaries without dewatering prior to the commencement of excavation work will result in extreme caving and heaving. This can translate to significant project delays, large amounts of additional excavation work, as well as the requirement for increased quantities of backfill. In addition, significant caving and heaving can result in the undermining and settlement of adjacent structures, pavements, roads, sidewalks, utilities, etc.

For all excavations, the groundwater table should be maintained at a minimum of 2 feet below the bottom of excavations and trenches. Drawdown of the water level should be maintained from before subgrade excavation until after the excavation has been backfilled. In the case of trenches, the dewatering should be conducted by means of well points or other acceptable means until the pipe has been laid and backfill has been completed to a stage where danger from flotation or base heave is eliminated via the downward pressure exerted by the trench backfill.

The disposal of the wastewater from dewatering operations should be tight-lined to existing disposal locations (i.e., ditches, channels, or drains) subject to permit approval. Settlement of suspended silt or sand will likely be required prior to discharge of wastewater from trench and excavation dewatering.

Surface water and rainwater runoff should be diverted to prevent it from entering trenches and excavations to the greatest extent possible without damage to adjacent property from dikes, ditches, or impounded water.

From the perspective of potentially reducing the amount of project dewatering, groundwater levels will typically be at their deepest levels from mid-summer through early fall months; however, even during summer months significant dewatering will still be required in trenches and excavations.

Based upon the soil conditions, it appears that drilled, pumped wells would be feasible for this project. The wells should be a minimum of 4 inches in diameter and should be installed by a licensed well driller with experience in dewatering.

Typically, a continuous line of wells would be advanced adjacent to the proposed trench alignment. Wells will require some amount of embedment below trench base elevations or manhole excavations. Well tip penetration below the base of excavations is required to draw down groundwater levels such that the potential for trench bottom and excavation base heave is minimized during trenching and pipe laying.

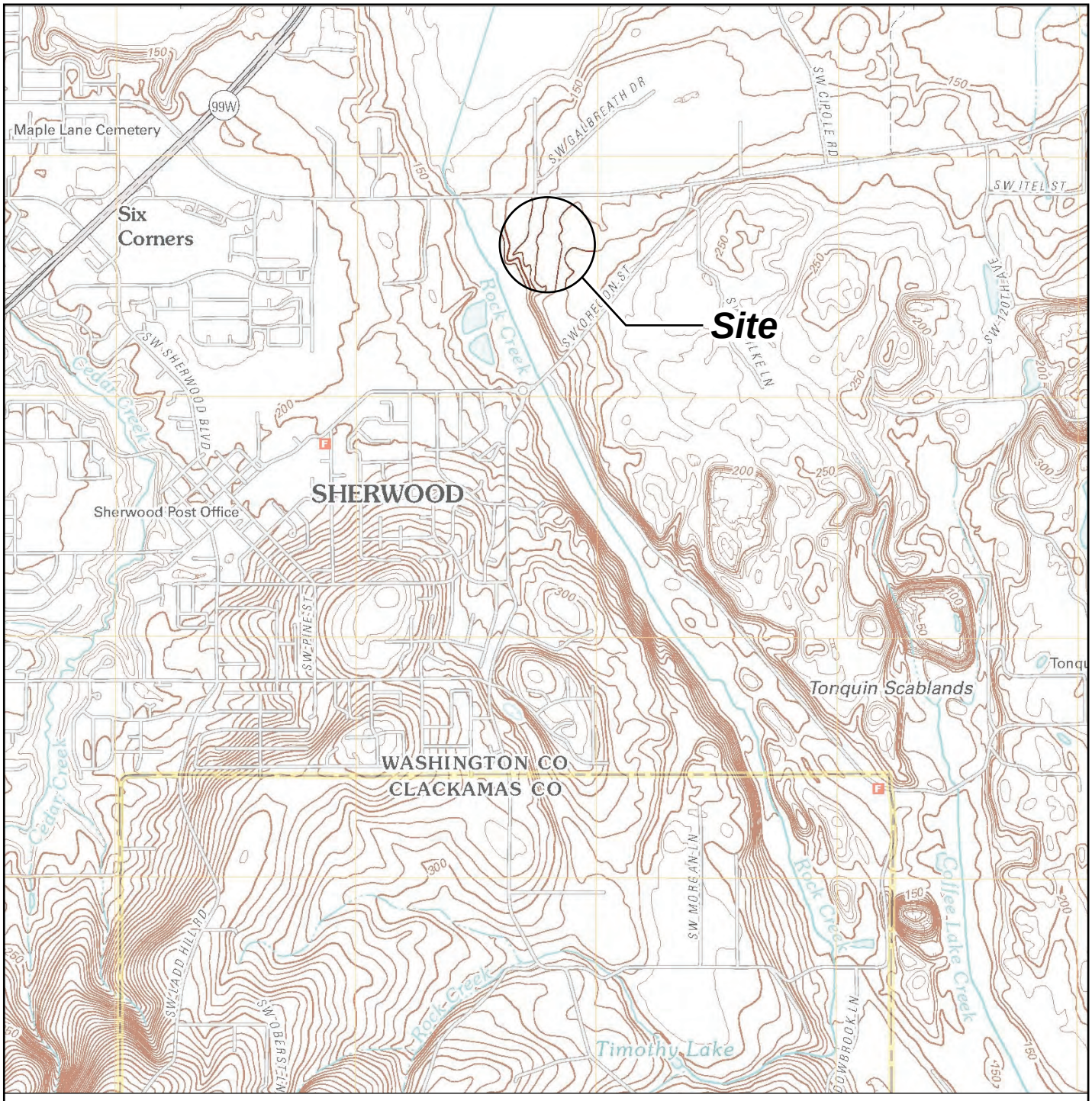
Trench and excavation dewatering may entail the generation of relatively large volumes of water during dewatering. Discharge of water drawn from dewatering wells will require phase planning prior to the dewatering plan being implemented. Discharge could potentially involve some method to remove silt from the effluent and routing to nearby native drainages.

9.0 Additional Geotechnical Services

Prior to construction, we recommend that Ash Creek reviews the final design plans and specifications. This review will allow us to evaluate whether any change in concept may affect the validity of our recommendations, and whether our recommendations have been correctly interpreted. In order to correlate preliminary soil data with the actual soil conditions encountered during construction, and to assess construction conformance to our report, we recommend Ash Creek be retained for construction observation of site preparation activities including excavation and compaction.

10.0 Closing

This report presented Ash Creek's Geotechnical Engineering Evaluation and recommendations for the proposed Tonquin Employment Area Sanitary Sewer Upgrade project in Sherwood, Oregon. We trust this report meets your needs. If you have any questions, or if we can be of further assistance, please call. We look forward to working with you in the future.



Note: Base map prepared from USGS 7.5-minute quadrangle of Sherwood, OR, dated 2011 as provided by USGS.gov.

0 2,000 4,000

Approximate Scale in Feet

N

Site Location Map

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Tonquin Employment Area Sanitary Sewer Upgrade
City of Sherwood, Oregon

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Project Number	1963-00	Figure
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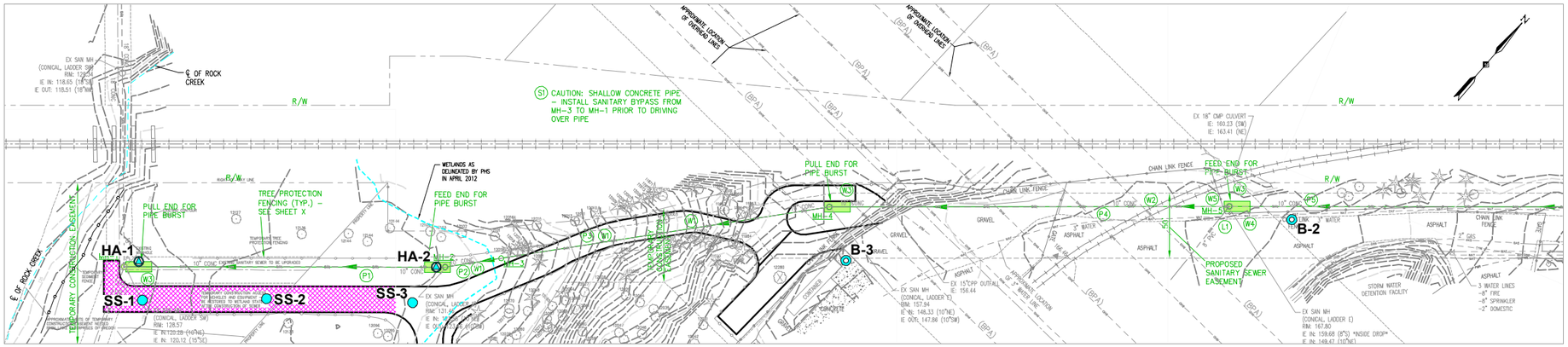
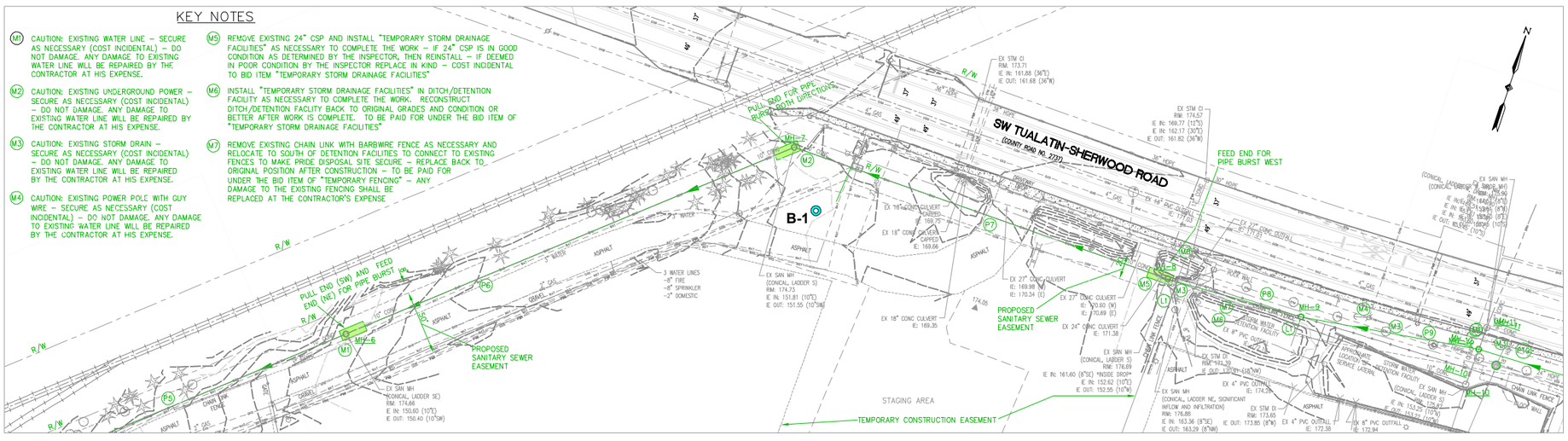
SANITARY SEWER PLAN AND PROFILE - SOUTHWEST



SANITARY SEWER PLAN AND PROFILE - WEST T-S ROAD



Legend:

HA-1 Hand Auger Location

B-1 Boring Location

SS-1 Soil Sample Location

0 100 200

Scale in Feet

Base map prepared from 8073 - Tonquin Employment Area Sanitary Upgrade - Sheets - Plan and Profile provided by the City of Sherwood.

Site Vicinity Plan

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City of Sherwood, Oregon

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Appendix A

Field Exploration Logs

Sample Descriptions

Classification of soils in this report is based on visual field and laboratory observations which include density/consistency, moisture condition, and grain size, and should not be construed to imply field nor laboratory testing unless presented herein. Visual-manual classification methods of ASTM D 2488 were used as an identification guide.

Soil descriptions consist of the following:
MAJOR CONSTITUENT with additional remarks; color, moisture, minor constituents, density/consistency.

Density/Consistency

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

SAND and GRAVEL Density	Standard Penetration Resistance in Blows/Foot
Very loose	0 - 4
Loose	4 - 10
Medium dense	10 - 30
Dense	30 - 50
Very dense	>50

SILT or CLAY Density	Standard Penetration Resistance in Blows/Foot
Very soft	0 - 2
Soft	2 - 4
Medium stiff	4 - 8
Stiff	8 - 15
Very Stiff	15 - 30
Hard	>30

Moisture

Dry	Little perceptible moisture.
Sl. Moist	Some perceptible moisture, probably below optimum.
Moist	Probably near optimum moisture content.
Wet	Much perceptible moisture, probably above optimum.

Minor Constituents	Estimated Percentage
Not identified in description	0 - 5
Slightly (clayey, silty, etc.)	5 - 12
Clayey, silty, sandy, gravelly	12 - 30
Very (clayey, silty, etc.)	30 - 50

Sampling Symbols

BORING AND PUSH-PROBE SYMBOLS

- ☒ Split Spoon
- ⧄ Sonic
- ⧅ Tube (Shelby, Push-Probe)
- ▥ Cuttings
- ◫ Core Run
- * No Sample Recovery
- SSA Solid Stem Auger
- HSA Hollow Stem Auger
- MR Mud Rotary

TEST PIT SOIL SAMPLES

- ☒ Grab
- ⧄ Bag
- ⧅ Shelby Tube

Key to Exploration Logs

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Project Number	1963-00	Figure
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Geotechnical Site Assessment
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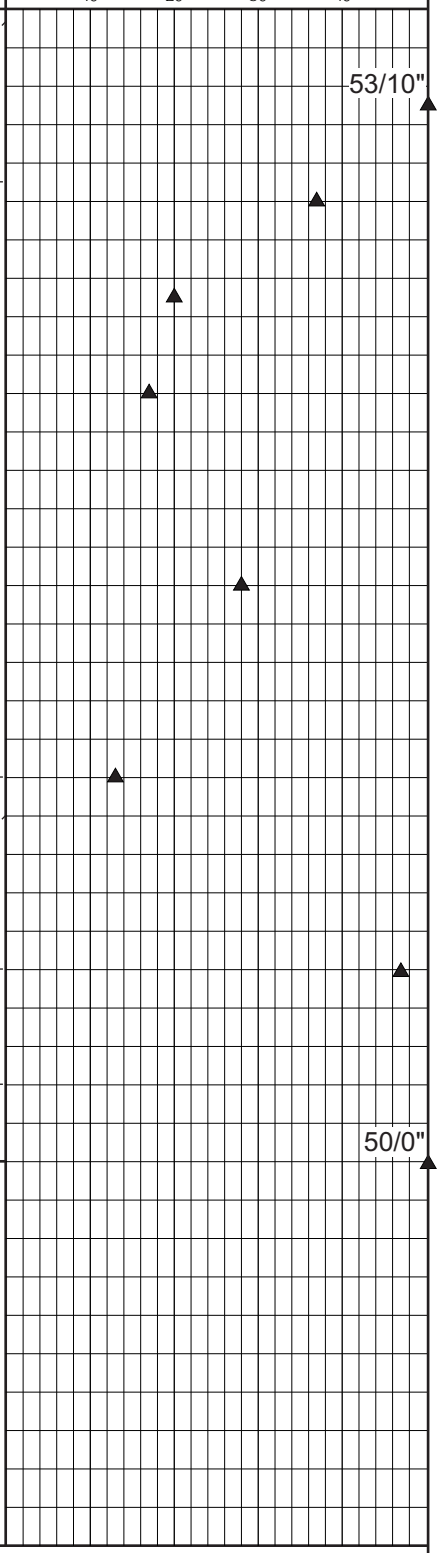
Boring Number: **B-1**

Project Number: **1963-00**
Logged By: A. Reese
Date: August 9, 2012
Site Conditions: --
Drilling Contractor: Western States
Drilling Equipment: Truck-mounted CME-75, Mud Rotary
Depth to Water (ATD): 15.8'
Surface Elevation: --

Depth, feet	Sample ID	Sample	Lithologic Description
			Grass/topsoil surface (4").
		☒	COBBLES in sandy, silty gravel matrix; brown (10YR 4/3), slightly moist, medium dense to dense.
5		☒	GRAVEL (to 3" diameter) in a silty, sand matrix; pale brown (10YR 6/1), moist, stiff.
		☒	
10		☒	
15		☒	
20		☒	Slightly silty SAND with some gravel; pale brown (10YR 6/1); moist, medium dense.
			Sandy, clayey SILT; soft to stiff. (Logged from cuttings)
25		☒	GRAVEL (to 3" diameter) in a silty sand matrix; brown (10YR 4/3), moist, stiff.
			BEDROCK (siltstone); very dense. (No sample recovery; logged from cuttings).
30			Bottom of Boring at 30.0' BGS.
			Note: Due to subsurface conditions, mud rotary drilling was employed; seepage and groundwater conditions could not, therefore, be accurately determined at the time of drilling.
35			



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Geotechnical Site Assessment
Tonquin Employment Area Sanitary Sewer Upgrade
City of Sherwood, Oregon

Boring Number: **B-2**

Project Number: **1963-00**
Logged By: A. Reese
Date: August 9, 2012
Site Conditions: --
Drilling Contractor: Western States
Drilling Equipment: Truck-mounted CME-75, Mud Rotary
Depth to Water (ATD): 15.8'
Surface Elevation: --

Depth, feet	Sample ID	Sample	Lithologic Description	Standard Penetration Resistance (Blows per Foot)
			Asphalt concrete surface (4"). Gravel subgrade (4").	
		X	Sandy SILT (Fill) with some fine to medium gravel; brown (10YR 4/3), slightly moist, medium dense.	
5		X	COBBLES and BOULDERS with silty, sandy gravel matrix; dark gray (10YR 4/1), dry, very dense.	50/3"
		X		
10		X		
15		X	Sandy SILT with trace gravel; brown (10YR 4/3), moist, stiff.	
20		X		
			COBBLES and BOULDERS with silty, sandy gravel matrix; dark gray (10YR 4/1), dry, very dense.	50/1"
25			Sandy SILT; soft to stiff. (Logged from cuttings)	
30		X	BEDROCK; reddish siltstone with vertically-oriented coarse sandstone veins, moist, very dense.	90/11"
			Bottom of Boring at 31.0' BGS.	
35				

Note: Due to subsurface conditions, mud rotary drilling was employed; seepage and groundwater conditions could not, therefore, be accurately determined at the time of drilling.

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Geotechnical Site Assessment
Tonquin Employment Area Sanitary Sewer Upgrade
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Boring Number: **B-3**
Project Number: **1963-00**
Logged By: A. Reese
Date: August 9, 2012
Site Conditions: --
Drilling Contractor: Western States
Drilling Equipment: Truck-mounted CME-75, Mud Rotary
Depth to Water (ATD): 15.8'
Surface Elevation: --

Depth, feet	Sample ID	Sample	Lithologic Description	Standard Penetration Resistance (Blows per Foot)
			Gravel surface/subgrade (8").	
			Sandy, silty GRAVEL (Fill); brown (10YR 4/3), moist, loose to medium dense.	
		X		
5		X		56/9"
			COBBLES and BOULDERS (to 6" diameter) with silty, sandy gravel matrix; dark gray (10YR 4/1), dry to wet, very dense.	
		X	Moderate mud loss.	63/8"
10		X	Moderate mud loss.	61/9"
15		—		50/1"
20		—		50/1"
25		—		50/2"
			Bottom of Boring at 25.2' BGS.	
30				
35				

Note: Due to subsurface conditions, mud rotary drilling was employed; seepage and groundwater conditions could not, therefore, be accurately determined at the time of drilling.

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Geotechnical Site Assessment
Tonquin Employment Area Sanitary Sewer Upgrade
City of Sherwood, Oregon

Hand Auger Number: **HA-1**
Project Number: **1963-00**
Logged By: A. Reese
Date: August 9, 2012
Site Conditions: --
Drilling Contractor: Western States
Drilling Equipment: Hand Auger, Stainless Steel, 3.25" ID
Depth to Water (ATD): --
Surface Elevation: --

Depth, feet	Sample ID	Core Interval/Recovery	Lithologic Description
		Hand Auger	Slightly clayey SILT; dark brown, wet, very soft, highly organic. (Standing water at the ground surface)
			SILT with some gravel; dark brown/very dark brown mottled, wet, soft, some organics.
			Clayey SILT; very dark brown, wet, medium stiff, moderate to high plasticity, some organics.
5			
			Bottom of Boring at 6.0' BGS.
10			

Hand Auger Number: **HA-2**
Logged By: A. Reese
Date: August 9, 2012
Site Conditions: --
Drilling Contractor: Western States
Drilling Equipment: Hand Auger, Stainless Steel, 3.25" ID
Depth to Water (ATD): --
Surface Elevation: --

Depth, feet	Sample ID	Core Interval/Recovery	Lithologic Description
		Hand Auger	Slightly clayey SILT; dark brown, wet, very soft, highly organic. (Standing water at the ground surface)
			Slightly sandy/clayey, organic SILT; dark brown/very dark brown mottled, soft, wet.
			Clayey SILT; very dark brown, wet, medium stiff, moderate to high plasticity, some organics.
5			
			Bottom of Boring at 6.0' BGS.
10			