CITY OF Salem
AT YOUR SERVICE

Improvements to the Aquifer Storage & Recovery Facility

Geotechnical Engineering Report

Final
Revision No. 1

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1.0 Introduction

1.1 General

McMillen Jacobs Associates (McMillen Jacobs) has been retained by Murraysmith to provide geotechnical engineering services for the Improvements to the Aquifer Storage & Recovery Facility Project (Project). The Project is in Woodmansee Park in Salem, Oregon, and the City of Salem (City) is the project Owner. This Geotechnical Engineering Report (GER) summarizes the geotechnical analyses and recommendations for the Project. The Project location is shown on the attached Figure 1, Vicinity Map.

1.2 Project Description

The City stores drinking water for backup use at the Aquifer Storage and Recovery (ASR) facility located in Woodmansee Park in Salem, Oregon. The ASR facility receives water diverted from the North Santiam River (the City's primary water source) during the winter, treats the water to drinking water quality, and injects it into a confined aquifer at a depth of about 300 feet below ground, within an interflow zone of the Columbia River Basalt unit. The ASR facility currently consists of five ASR wells (four of which are operational) located in four existing well houses across Woodmansee Park, as well as appurtenant conveyance piping.

As part of the City's commitment to provide safe drinking water, several improvements are proposed for the ASR Facility. Our project understanding is based on our communications with, and the 50% Design Submittal drawings provided by, Murraysmith. The proposed improvements to the ASR Facility include the following:

- A new water treatment facility (WTF) located on the east side of the park, approximately 50 feet north of the existing ASR Well No. 4 pumphouse. The new 46-foot by 47-foot WTF will have a finished floor elevation of 379 feet. To facilitate the construction of the new WTF, cuts up to about 3 feet deep will likely be required. We anticipate the new WTF structure will be lightly loaded and founded on either a conventional perimeter footing with slab-on-grade foundation system or on a mat foundation.
- The new WTF will be situated in the middle of an approximately 180-foot by 110-foot area that will be regraded/flattened and paved with asphalt. In addition, the existing, 15-foot wide, approximately 450-foot long, City-owned easement and gravel access road from Sunnyside Road SE will be re-graded and widened to 35 feet and paved with asphalt; providing access to the new WTF.
- Up to three new ASR wells, each enclosed in an approximately 900 square-foot pumphouse. We anticipate the pumphouses will be lightly loaded and founded on mat foundations.
- We understand that a stormwater infiltration facility will be constructed within the northwest corner of the above-referenced paved area, to offset the increased stormwater volume of the impermeable area resulting from the WTF property development. The stormwater infiltration facility design will be completed by Murraysmith.

- Piping associated with the project includes the following:
    - The Sunnyside Road SE transmission main consists of approximately 2,200 feet of 24-inch Ductile Iron Pipe (DIP).
    - Yard piping consists of approximately 2,600 feet of 24-inch DIP that will convey flows between the WTF, the existing ASR facilities, and the existing City distribution piping.
    - Other piping shown on the 50% Design Submittal drawings includes approximately 425 feet of 10-inch DIP storm drain (SD) piping, 135 feet of 4-inch PVC SD piping, and 200 feet of 4- and 8-inch PVC sanitary sewer (SS) piping.
    - The drawings show a typical 48-inch depth of cover for the 24-inch DIP water piping and an approximate 15- to 48-inch depth of cover for the 10-inch DIP SD piping.
- Based on our communication with Murraysmith, we understand the proposed precast concrete vaults will have typical dimensions of 6-foot wide by 8-foot long by 7-foot deep.

1.3 Purpose and Scope of Work

The purpose of our work is to evaluate the subsurface conditions and to provide geotechnical engineering design and construction recommendations for subsequent use by the design team in support of the Project. Specifically, the scope of our work included the following:

- Subsurface investigation at the Project site including four drilled borings advanced to approximate depths of up to 50 feet below ground surface (bgs);
- Laboratory testing on soil samples obtained from the drilled borings, including moisture content, Atterberg Limits, particle size analysis, and shrink-swell / expansion pressure;
- Characterization of subsurface conditions at the proposed WTF, the ASR well locations, and new pipelines based on geotechnical explorations and laboratory testing;
- Geotechnical engineering assessments and design recommendations for structure and pipeline subgrade properties and settlement potential;
- Shallow foundation design recommendations for the proposed WTF and the pumphouses for the new ASR wells;
- Seismic hazard evaluation results and seismic geotechnical recommendations for the design of new structures;
- Bedding and backfill recommendations for the replacement pipeline and backfill compaction criteria;
- Recommendations for lateral earth pressures on embedded structures (e.g., manholes and vaults);
- Recommendations for structural fill, bedding, backfill, and compaction criteria for foundations, pipelines, and buried structures;
- Recommendations for design and construction of new flexible (e.g., asphalt concrete) pavements.
- Recommendations for subgrade stabilization, if required;
- Preparation of this Geotechnical Engineering Report.

2.0 Geotechnical Exploration

2.1 Exploratory Borings

The geotechnical exploration for the Project was completed on April 6 and 7, 2020, and included four exploratory borings (B-1 through B-4). Western States Soil Conservation, Inc. of Hubbard, Oregon, completed the borings using a track-mounted CME-55 drill rig. The borings were advanced to depths ranging from 20 feet to 50 feet bgs, using hollow-stem auger and mud-rotary drilling techniques. The approximate locations of the exploratory borings are shown in Figure 2.

In addition, we completed four dynamic cone penetrometer (DCP) tests within the proposed pavements areas around the new WTF on June 26, 2020. Details of the DCP tests are provided in Section 2.3 below and the approximate locations of the tests are shown in Figure 3.

The explorations were completed under the supervision of a McMillen Jacobs geologist who maintained continuous observation, collected soil, and maintained a full-depth descriptive log of the soil materials penetrated in each borehole.

2.2 Soil Classification & Sampling

The soil samples collected in the borings were classified in accordance with the Visual-Manual Procedure (ASTM D2488). Sample depths, stratigraphy, groundwater observations, and soil engineering characteristics were also noted. The stratigraphic contacts, indicated on the exploration logs in Appendix A, represent the approximate boundaries between soil types; actual transitions between soil units might be more gradational than shown.

Disturbed soil samples were collected at selected intervals using a standard 2-inch diameter split-barrel sampler and automatic safety hammer system. In each test, the sampler was advanced 18 inches by dropping a 140-pound hammer 30-inches for each strike in accordance with ASTM D1586. The number of hammer-blows for each 6 inches of penetration was recorded. The standard penetration resistance (designated as the "N-value") of the soil is calculated as the sum of the number of blows required for the final 12 inches of sampler penetration. SPT N-values of 50 or more blows per 6 inches or less of penetration is defined as "refusal." The N-value is an indication of the relative density of granular soils and the relative consistency of cohesive soils. The drill rig was equipped with an automatic hammer to obtain Standard Penetration Test (SPT) samples. Western States provided an automatic hammer calibration Report of SPT Hammer Energies (GeoDesign, 2018), showing that the drill rig used for our site investigation (Rig #2, CME-55) has an energy transfer ratio of 81.2 (correction factor = 1.353). N-values reported on our boring logs are, however, uncorrected field-recorded values (i.e., no corrections have been applied).

2.3 Dynamic Cone Penetrometer Tests

McMillen Jacobs performed four dynamic cone penetrometer (DCP) tests within the future pavement subgrade around the proposed WTF to depths up to about 2 feet bgs. DCP testing was performed in general accordance with ASTM D6951, and consists of driving a 20-mm diameter, hardened steel cone on 16-mm diameter steel rods into the ground using a 8-kg drop hammer with a 460-mm, free-fall height.

The number of hammer blows required to drive the DCP tip is typically recorded in 10-mm increments. The DCP index (defined as the amount of penetration per blow) is calculated by dividing the incremental penetration by the number of blows. The DCP index can be correlated to subgrade resilient modulus (M_R) using published correlation equations (ODOT, 2011). DCP test results are presented in Appendix C.

It should be noted that it is standard practice to perform DCP testing in conjunction with exploratory borings, and to obtain physical samples of the pavement subgrade soils for laboratory testing and physical examination. However, due to cultural resources constraints on obtaining physical soil samples from the Project site, it was decided that only DCP testing would be permissible. Therefore, no soil samples were collected at the DCP testing locations.

2.4 Laboratory Testing

Soil samples were delivered to the McMillen Jacobs Portland office for further examination and storage. Each of the samples was re-examined and compared to the field boring log description to confirm the field classifications and maintain consistency. Representative samples were then selected for the following laboratory testing:

- Water (Moisture) Content of Soil and Rock by Mass (ASTM D2216);
- Amount of Material Finer than a No. 200 Sieve (i.e., ‘Fines Content’) (ASTM D1140);
- Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils (ASTM D4318); and
- Standard Test Methods for One-Dimensional Expansion, Shrinkage, and Uplift Pressure of Soil-Lime Mixtures (ASTM D3877), with an explanation provided below.

Moisture contents, Atterberg limits, and Percent fines are indicated on the boring logs in Appendix A. Individual laboratory test reports are included in Appendix B.

2.4.1 One-Dimensional Expansion, Shrinkage, and Uplift Pressure Tests

Three, one-dimensional expansion, shrinkage, and uplift pressure tests (referred to hereafter as shrink-swell / expansion pressure tests) were performed on remolded composite SPT samples. The expansion pressure test is the first step, in which the sample is placed in a consolidometer and inundated. As the sample tries to expand, the vertical pressure is increased just enough to keep the sample from expanding. This process is continued until the applied vertical load maintains a constant-volume condition of the soil sample. The vertical pressure to maintain this condition is the uplift or expansion pressure.

After the uplift pressure phase of the test, the expansion-shrinkage phase of the test is performed. The sample is removed from the consolidometer, weighed, and allowed to air-dry for typically three days. Then, the sample dimensions and weight are measured and put in an oven. After oven-dried, the sample dimensions and weight are measured again. The volume change vs. moisture content is then plotted; an increase in volume corresponds to expansion and a decrease in volume corresponds to shrinkage. A summary of these test results is presented in Table 2-1.

Table 2-1: Results Summary of Shrink-Swell & Expansion Pressure (ASTM D3877) Tests

Sample Designation	Sample Depth (feet)	In-Situ Moisture Content (%)	Expansion Pressure Phase		Shrink-Swell Phase	
			Expansion Pressure (psf)	Change in Height (%) (positive=swell), (negative=shrink)	Volume Change[3] (%)	Range of Moisture Contents[4] (%)
Boring B-1, Sample S-1/S-2	2.5 to 6.5	33.1	7,381[1]	1.32	-5.1	32.7 to 0.0
Boring B-1, Sample S-3/S-4	7.5 to 11.5	59.3	N/A[2]	-0.3% at 100 psf load	-26.1	59.9 to 0.0
Boring B-4, Sample S-1/S-2	2.5 to 6.5	39.9	N/A[2]	-0.4% at 100 psf load	-16.6	40.2 to 0.0

Notes:

1. Expansion pressure based on inundating soil from air-dried condition.
2. Expansion pressure based on inundating soil from in-situ/field moisture content.
3. Volume change measured in one dimension (e.g., height of sample), from saturated to oven-dry. Negative values indicate shrinkage.
4. Higher moisture content value represents saturated condition and 0 percent moisture represents oven-dry conditions.

3.0 Site Conditions

3.1 Surface Conditions

The Project site is in Woodmansee Park, approximately 30-acre City park located on the west side of Sunnyside Road SE and about ½ mile north of Kuebler Boulevard. The Project site is bordered by Sunnyside Road SE to the east, residential developments to the north and south, and the north-flowing Pringle Creek to the west. Existing features within Woodmansee Park include two tennis courts, a basketball court, restroom facilities, an access road, three parking areas, a soccer/sports field, a covered picnic area, and a frisbee golf course. The southern portion of the park is generally grass-surfaced and includes the tennis courts, basketball court and the soccer/sports field, while the northern and western areas of the park are generally more vegetated with coniferous and deciduous trees. Pringle Creek flows along the western boundary of the park. In general, topography within the park slopes gently downhill to the north, with an overall vertical relief of about 15 to 20 feet. To the west of Pringle Creek, the topography rises to the west, with an overall vertical relief of about 10 feet, and flattens out at the neighboring Judson Middle School property.

3.2 Site Geology

The Project site is situated in Salem Heights, the name given to the rolling upland area of indefinite extent near the southern City of Salem limits and is included in the northeastern part of the more extensive Salem Hills area (Foxworthy, 1970). The approximately 60-square-mile Salem Hills area rises to the south from the City of Salem, from an elevation of approximately 200 feet to slightly more than 1,100 feet at Prospect Hill, located about 4 miles southwest of the Project site.

The oldest rock unit in the Project site area is the Eocene-Oligocene sedimentary bedrock, typically consisting of sandstone, siltstone, and mudstone, with lesser amounts of conglomerate and interspersed localized volcanic rocks (Wang and Leonard, 1996). The middle Miocene (about 17 to 6 million years before the present time) Columbia River Basalt Group (CRBG) overlies the sedimentary bedrock, and typically consists of weathered and fresh basaltic lava flows with interflow zones characterized by vesicular flow-top breccia, ash, and baked soils (Wang and Leonard, 1996). The maximum CRBG thickness ranges from about 400 to 600 feet, generally thickest in the Salem Hills and thins to the north towards Salem. Well logs drilled in Woodmansee Park as part of the original City of Salem ASR study indicated a CRBG thickness of about 270 feet (Foxworthy, 1970).

The upper region of the basalt has been deeply weathered to laterite and saprolite soils; red-brown clayey soils, collectively referred to in this report as residual soil. The residual soils extend to depths on the order of 200 feet in the Salem Hills and generally thin out to the north towards Salem; typically, several tens of feet thick in the Salem Heights and Salem Hills areas. The previously-referenced ASR study encountered residual soils to depths of up to 65 feet (Foxworthy, 1970) at the Project site.

Overlying the bedrock units are Quaternary lower terrace deposits of alluvial bottomlands; land that is typically adjacent to a river or creek (e.g., Pringle Creek in this case) and subject to overflow during floods. These deposits primarily consist of low to medium plasticity, clayey soils with minor amounts of sand and typically extend to depths of about 4 to 12 feet along bottomlands of interior drainages (Bela, 1981).

3.3 Subsurface Conditions

Subsurface conditions were generally consistent between the four borings advanced as part of our geotechnical exploration. Except for an upper 1-foot topsoil zone, we encountered residual soil to the maximum depths explored, up to 50 feet bgs. We did not encounter Quaternary alluvium (i.e., lower terrace deposits) that is shown on geologic mapping; likely because this unit is locally present in the vicinity of Pringle Creek and our explorations were at least 300 feet from Pringle Creek. The residual soil unit has been defined by its geologic origin, stratigraphic position, engineering properties, and its distribution in the subsurface. Variations in subsurface conditions may exist between the locations of the borings. Contacts between the various soil types within the residual soil unit are approximate and may be more gradational than shown on the boring logs in Appendix A.

3.3.1 Residual Soil of Columbia River Basalt Group

Residual soil of the Columbia River Basalt (CRB) group was encountered below topsoil in each of the borings, at an approximate depth of 1 foot bgs, and extended to the maximum depth explored of approximately 50 feet bgs. The residual soil is a product of deep, in-place weathering of the CRB.

The residual soil is typically red-brown with frequent varicolored mottled zones, ranging from red to yellow to black. The overall structure of the residual soil is typically heterogeneous, with a clayey, silty matrix containing frequent angular gravel clasts of predominately weathered to decomposed basalt, with the parent rock fabric being apparent.

Forty-two SPT N-values were recorded in the residual soil, ranging from 2 to 53 blows per foot (bpf) and averaging 17 bpf; indicating soft to hard consistencies. Seven Atterberg Limits tests indicated liquid limits (LL) ranging from 48 to 112 and plasticity indices (PI) ranging from 24 to 51; resulting in lean clay (CL), fat clay (CH), and elastic silt (MH) soil classifications. Thirty-six moisture contents test ranged from 32 to 83 percent and generally increased with depth.

In the upper 7 to 12 feet, soil moisture content typically ranges from 32 to 40 percent, and increases from 53 to 83 percent below these depths. Soils in the upper 7 to 12 feet typically exhibit transitional-type behavior, plotting around the CL-CH-MH transition zone on the PI versus LL graph. An exception to this trend was observed in boring B-2, in which non-plastic silt with sand (ML) and moisture content above 50 percent was observed in the upper 16 feet.

3.4 Groundwater

The borings were advanced using mud rotary drilling methods, thereby precluding the observation of groundwater conditions during drilling. However, in boring B-4 (at the proposed WTF), the borehole was flushed clean and allowed to stay open for about two hours, allowing for the groundwater level to stabilize to the extent possible. Using an electronic groundwater level indicator, we measured a groundwater level of 5.6 feet bgs at this location.

We researched available well logs located within Section 10, Township 8 South, Range 3 West, Willamette Meridian on the Oregon Water Resources Department (OWRD) website. Our research resulted in five well logs drilled in Woodmansee Park as part of the City of Salem ASR study. Our review indicated groundwater levels at the site ranged from about 8 feet to 135 feet bgs. It should be noted that groundwater levels reported on the OWRD logs often reflect the purpose of the well, so water well logs may only report deeper, confined groundwater, while geotechnical or environmental borings will often report any groundwater encountered, including shallow, unconfined groundwater. Therefore, the levels reported on the OWRD well logs referenced above are considered generally indicative of local water levels and may not reflect actual groundwater levels at the project site. However, well logs for ASR wells No. 4 and ASR No. 6 indicated groundwater was first encountered at depths of 10 feet and 8 feet bgs, which is consistent with our findings.

Groundwater levels may vary with precipitation, the time of year, site utilization, and/or other factors. Generally, groundwater highs occur near the end of the wet season in late spring or early summer and groundwater lows occur near the end of the dry season in the early fall. We believe that that water levels encountered during the current investigation are close to the seasonal maximum.

4.0 Seismic and Geologic Hazard Evaluation

We performed a seismic hazards evaluation in general accordance with the 2019 Oregon Structural Specialty Code (OSSC, 2019) and ASCE's Minimum Design Loads for Buildings and Other Structures, 2016 Edition (ASCE/SEI 7-16). The OSSC requires evaluating the seismic hazards for the Maximum Credible Earthquake (MCE) having a 2-percent probability of exceedance in a 50-year period (2,475-year return period).

4.1 Regional Seismicity

The Pacific Northwest is a seismically active region that has three principle seismic sources: (1) the Cascadia Subduction Zone (CSZ) megathrust, which represents the interface between the subducting Juan de Fuca plate and the overriding North American plate; (2) faults located within the Juan de Fuca plate (referred to as CSZ intraplate or intraslab sources); and (3) crustal faults principally within the North American plate (Wong and Silva, 1998). Faulting and seismicity associated with Cascade volcanoes are also potential sources of seismicity, though they generally do not impact sites in the Willamette Valley.

4.1.1 Crustal Sources

Crustal sources typically occur at depths ranging from approximately 14 to 40 kilometers bgs (Geomatrix Consultants, 1995). A search was performed on the U.S. Geological Survey (USGS) website (USGS, 2020) to identify known crustal seismic sources within 20 kilometers (about 12.5 miles) of the project alignment. These faults are presented in Table 4-1.

Table 4-1. Known Faults Within 20 km of the Project Site

USGS Fault ID.	Fault Name	Char. Mag	Type of Fault	USGS Fault Class[1]	Approx. Earthquake Depth (km)	Distance (km) & Direction from Site	Notes
719	Salem-Eola Hills Homocline	6.00	Homocline (Normal)	A	15 to 40 km	6.3 km West	3
871	Turner and Mill Creek faults	6.59	Normal	A	15 to 40 km	9 km South	2
872	Waldo Hills fault	6.00	Normal	A	15 to 40 km	4.5 km Southwest	3

Notes:

1. USGS Fault Classes from USGS Earthquake Hazards Program, 2014 National Seismic Hazard Maps
    - Class A: Fault with convincing evidence of Quaternary activity (ACTIVE)
    - Class B: Fault that requires further study in order to confidently define their potential as possible sources of earthquake-induced ground motion (POTENTIALLY ACTIVE)
2. Characteristic earthquake magnitude from USGS Earthquake Hazards Program, 2014 National Seismic Hazard Maps – Fault Parameters.
3. Characteristic earthquake magnitude from Section 1803.3.2.1 of the 2019 OSSC - Design Earthquake.

4.1.2 Cascadia Subduction Zone Seismic Sources

The Cascadia Subduction Zone (CSZ) is an approximate 1,000-kilometer-long zone of active tectonic convergence where oceanic crust of the Juan de Fuca Plate is subducting beneath the North American continental plate at a rate of about 3 to 4 centimeters per year (DeMets, et al., 1990). The fault trace is located off the coast of southern British Columbia, Washington, Oregon, and northern California; approximately 325 kilometers west of the site.

There are two primary seismicity sources associated with the CSZ: 1. relatively shallow earthquakes that occur on the interface between the Juan de Fuca and North American plates (i.e., Subduction Zone earthquakes) and 2. deep earthquakes that occur along faults within the subducting Juan de Fuca plate (i.e., intraplate earthquakes). These two types of earthquakes are discussed in the following sections.

4.1.2.1 Subduction Zone Earthquakes

Large subduction zone (megathrust) earthquakes occur within the upper approximate 30 kilometers of the contact between the two plates (Pacific Northwest Seismic Network (PNSN), 2020). As the Juan de Fuca Plate subducts beneath the North American Plate through this zone, the plates are locked together by friction (PNSN, 2020). Stress slowly builds as the plates converge until the frictional resistance is exceeded, and the plates rapidly slip past each other resulting in a megathrust earthquake. The USGS estimates megathrust earthquakes on the CSZ may have magnitudes up to M9.2. Geologic evidence indicates a recurrence interval for major subduction zone earthquakes of 250 to 650 years, with the last major event occurring in 1700 (Atwater, B.F., 1992).

4.1.2.2 Intraplate Earthquakes

Below depths of approximately 30 kilometers, the plate interface does not appear to be locked by friction, and the plates slowly slide past each other. The curvature of the subducted plate increases as the advancing edge moves east, creating extensional forces within the plate. Normal faulting occurs in response to these extensional forces. This region of maximum curvature and faulting of the subducting plate is where large intraplate earthquakes are expected and is located at approximate depths ranging from 30 to 60 kilometers (Geomatrix Consultants, 1993, 1995 and Kirby, S.H. et al., 2002). Intraplate earthquakes within the Juan de Fuca plate generally have magnitudes less than M7.5 (Cascadia Region Earthquake Workshop, 2008). The 2001 M6.8 Nisqually earthquake near Olympia, Washington, occurred within this seismogenic zone at a depth of 52 kilometers.

4.2 Site Classification

We assigned a seismic site class for the Project site following code-based procedures in ASCE/SEI 7-16, Chapter 20 (2016). Site class is used to categorize common subsurface conditions into broad classes to which ground motion attenuation and amplification effects are assigned. Site classification is based on the weighted average of the shear wave velocity or Standard Penetration Test (SPT) blow counts (N-value) in the upper 100 feet of subsurface profile. Based on the SPT N-values from our recent geotechnical exploration, a Site Class D is appropriate for design purposes.

4.3 Seismic Design Parameters

The 2019 OSSC requires that spectral response accelerations be developed based on ASCE 7-16. We developed spectral response accelerations using the online ASCE 7 Hazard Tool, which references ground motion procedures in accordance with ASCE 7-16 and is based on the USGS 2014 National Seismic Hazard Mapping Project (NSHMP) developed for the Maximum Considered Earthquake (MCE)

(Peterson et. al., 2008). The MCE consists of ground motions (accelerations) with a 2-percent probability of exceedance in 50 years (return period of 2,475 years). The mean earthquake magnitude and the mean site-to-source distance for the zero-second period of vibration (e.g., PGA) are 8.34 and 68.75 km, respectively, for the MCE. The recommended spectral acceleration parameters for use in structural design are provided in Table 4-2.

Table 4-2. 2019 OSSC MCE Spectral Acceleration Parameters for Site Class D

Parameter	0.2-Second Period	1-Second Period
Mapped MCE_R (Rock site)	S_S = 0.821g	S_1 = 0.415g
Site Coefficients	F_a = 1.172	F_v = 1.885
Site-Adjusted MCE_R	S_{MS} = 0.962g	S_{M1} = 0.782g
Design MCE_R	S_{DS} = 0.641g	S_{D1} = 0.523g
Mapped MCE_G PGA (Rock Site)	0.382g	
Site Coefficient F_{PGA}	1.218	
Site-adjusted MCE_G PGA	0.466g	
Site-Adjusted Peak Ground Velocity (PGV) (cm/sec)	74	

It is important to note that Section 11.4.8 of ASCE 7-16 requires a site-specific ground motion hazard analysis be performed on structures on Site Class D sites with a 1-second spectral response acceleration parameter (S_1) greater than 0.2g. However, exception No. 2 in Section 11.4.8 states that a site-specific ground motion hazard analysis is not required if the structure's fundamental period of vibration T is less than $1.5T_s$. When this condition is met, the seismic response coefficient C_s shall be calculated using equation 12.8-2 in ASCE 7-16. The following provides a summary of these parameters:

- The pumphouse and WTF buildings will be single-story, reinforced concrete structures; therefore, we anticipate T will be less than or equal to 0.5 second;
- T_s equals the design 0.2-second spectral response parameter S_{DS} divided by the design 1-second spectral response parameter S_{D1}. Using this equation and the S_{DS} and S_{D1} values in Table 4-2, T_s equals 1.226 and $1.5T_s$ equals 1.839; and
- T is less than $1.5T_s$ and therefore, a site-specific ground motion hazard analysis is not required, and the seismic response coefficient C_S shall be calculated using equation 12.8-2 in ASCE 7-16.

4.4 Seismic Sources and Hazard Deaggregation

The probabilistic seismic hazard assessment (PSHA) produces a mean source event (e.g., the MCE) that generates the spectral accelerations reported in Table 4-2. The deaggregation data identify the earthquake sources, magnitudes, and site-to-source distances that contribute to the mean source event. Table 4-3 summarizes the results of the mean source event hazard deaggregation for the zero-second period of vibration (e.g., PGA).

Table 4-3: Deaggregation Results for 2,475-year Mean Source Event (MCE), PGA Period

Source	Moment Magnitude, M_W[1]	Site-to-Source Distance[2] (km)	% Contribution to Hazard
CSZ Interface	8.95	78.61	73.37
CSZ Intraslab	6.98	59.84	16.49
Crustal Faults[3]	6.19 to 6.6	11.77 to 13.92	10.14

Notes:
1. M_W values represent the mean value from each type of earthquake source.
2. Site-to-Source distances represent the mean value from each type of earthquake source.
3. Crustal faults source includes gridded seismic sources that represent earthquakes that do not occur on known, mapped faults.

4.5 Liquefaction

Liquefaction is a phenomenon whereby saturated cohesionless soils, generally sands and silts, undergo significant loss of strength and stiffness when they are subjected to vibration or large cyclic ground motions produced by earthquakes. During earthquake shaking (e.g., undrained conditions), loads are transferred from the soil skeleton to the pore-water with consequent reduction in the soil shear strength. The shear strength of a cohesionless soil is directly proportional to the effective stress; equal to the difference between the overburden pressure and the pore water pressure. The susceptibility of sands, gravels, and sand-gravel mixtures to liquefaction is typically assessed based on in-situ penetration resistance tests (e.g., SPT, CPT, etc.).

For fine-grained soils, however, susceptibility to liquefaction can be characterized into three categories (Boulanger and Idriss, 2014):

1. **Sand-like behavior:** Fine-grained soils with a plasticity index less than 7 are considered to exhibit classical soil liquefaction behaviors like clean sand.
2. **Clay-like behavior:** Fine-grained soils with a plasticity index greater than 12 are not considered to be liquefiable; however, cyclic softening and strain accumulation from seismic shaking should be considered.
3. **Transitional behavior:** Fine-grained soils with a plasticity index between 7 and 12 should be considered as “transitional” soils, and their seismic behaviors are expected to include cyclic softening, strength loss, and post liquefaction settlement.

As discussed in Section 3.3, we encountered residual soils from the ground surface to the maximum depths explored in each of our exploratory borings, about 50 feet bgs. Based on the results of our visual-manual soil classification, the residual soils exhibited clay-like behavior. Seven Atterberg Limits tests were completed on representative soils encountered during our geotechnical exploration, with plasticity indices ranging from 24 to 51; well within the clay-like behavior category. Therefore, we conclude the risk of liquefaction is negligible for the Project. This judgement is further supported by the liquefaction susceptibility map, which indicates “No Susceptibility” hazard level for the Project site (Wang and Leonard, 1996).

4.6 Lateral Spreading

Lateral spreading is a liquefaction-related phenomenon that results in ground displacement during an earthquake and occurs in sloping ground or flat ground with free face. Surface rupture due to lateral spreading can occur on sites underlain by liquefiable soils that are located immediately adjacent to slopes steeper than about 3 degrees (20H:1V), and/or adjacent to a free face, such as a stream bank or the shore of an open body of water. During lateral spreading, the materials overlying the liquefied soils are subject to lateral movement downslope or toward the free face. Due to the lack of liquefiable soils at the Project site, we conclude that the risk of lateral spreading is negligible for the Project.

4.7 Fault Rupture

There are no known active faults that are mapped on or immediately adjacent to the Project site. The nearest fault considered to be active is the Waldo Hills fault located 4.5 km southwest of the Project site. Therefore, we conclude the risk of surface rupture due to faulting is negligible.

4.8 Slope Stability

Due to the relatively flat to gently sloping topography on and surrounding the Project site, we conclude that the risk of slope instability, for both static and seismic conditions, is negligible. This conclusion is supported by our review of available online landslide susceptibility mapping on the Oregon Department of Geology and Mineral Industries (DOGAMI) Statewide Geohazards Viewer (HazVu), which indicates a “Low – Landsliding Unlikely” hazard level at the Project site (Oregon DOGAMI, 2020).

4.9 Flood Hazard

The Oregon DOGAMI HazVu mapping utilizes FEMA Flood Insurance Rate Maps (FIRM) to assess flood hazard. HazVu mapping indicates the project alignment is outside of any mapped flood hazards (DOGAMI, 2020).

4.10 Other Hazards

Other geologic and seismic hazards, including debris flows, and tsunamis/seiches are not considered hazards to the Project.

5.0 Conclusions and Key Geotechnical Considerations

Based on the results of our field explorations and analyses, the site can be developed as described in Section 1.2 of this report, provided the recommendations presented in this report are incorporated into the design and development. The primary geotechnical considerations for the Project are as follows:

- The presence of potentially shrinking-swelling, high plasticity soils (e.g., elastic silt and fat clay) at or near the anticipated foundation depths of the WTF and pumphouse buildings; and
- The presence of fine-grained silty residual soils which are sensitive to construction disturbance and moisture changes.

These considerations are discussed in further detail in the following sections.

5.1 Shrink-Swell Potential of Subgrade Soils

Three, shrink-swell / expansion pressure tests were performed within the residual soil unit. The test methods are discussed in Section 2.4.1, with the results summarized in Table 2-1. Detailed shrink-swell / expansion pressure test results are included in Appendix B.

Boring B-1 sample S-3/S-4 and boring B-4 sample S-1/S-2 did not exhibit expansive behavior; rather, they consolidated slightly (e.g., less than ½ percent) under a 100-psf load. The boring B-1 S-1/S-2 sample indicated an expansion pressure of about 7,400 psf. However, this expansion pressure is based on inundating the soil sample from an air-dried condition and is not representative of anticipated conditions expected during the service life of the proposed structures. Incidentally, this test demonstrates that foundation and structural distress could occur if the foundation subgrade soils are allowed to dry excessively during construction. As indicated in Table 2-1, the in-situ moisture content of each soil sample was negligibly less than saturated. Soil expansion corresponds to the volume increase due to the soil's absorption of water. A saturated or nearly-saturated soil cannot absorb additional water, or only a negligible amount of water. Therefore, at their in-situ saturated or nearly-saturated moisture conditions, the foundation subgrade soils are not susceptible to appreciable expansion if subjected to additional wetting.

As indicated by the shrink-swell / expansion pressure test results, each of the soil samples exhibited shrinkage ranging from 5.1 to 26.1 percent. The shrinkage occurred with a loss in moisture content, from saturated to oven-dry conditions, and corresponds to the decrease in soil sample height, expressed as a percentage.

The primary concern related to expansive and shrinkage potential of the foundation subgrade soils is the fluctuation of moisture content. The shrink-swell / expansion pressure tests demonstrate these soils are subject to significant changes in volume with changes in moisture content; swelling with an increase in moisture content and shrinking with a decrease in moisture content. Foundations and floor slabs founded directly on these soils may be subject to cyclic shrink-swell movements that can result in differential movements and structural distress. Therefore, to mitigate the shrink-swell potential of the foundation subgrade soils, we recommend the following:

- Extend the shallow foundations to a depth at which seasonal moisture content fluctuation is minimal, about 2.5 feet below adjacent site grades;
- Concrete slabs-on-grade should be supported on a minimum 6-inch thick layer of structural fill over properly prepared subgrade;

- Pavement subgrade should be overexcavated 12 inches and replaced with imported granular structural fill;
- Exposed foundation subgrade soils should be covered immediately with either structural fill or a geotextile fabric to minimize moisture loss;
- Exposed foundation subgrade soils should be saturated prior to casting concrete;
- Install perimeter drains around footings to drain the collected water in the backfill above footing level;
- Provide positive surface drainage away from all points around the building, with hardscaping extending a minimum of 5 feet from the building perimeter;
- Prevent the root systems of nearby large trees from approaching the foundations; and
- Eliminate landscaping around the perimeter of the structures to prevent water infiltration into the foundation subgrade.

5.2 Construction Disturbance on Moisture-Sensitive Soils

The fine-grained residual soils are highly susceptible to disturbance and damage due to construction traffic during wet weather construction. If earthwork is undertaken without proper precautions, when the exposed soils are wet of optimum moisture content by more than a few percentage points, then significant damage to the subgrade could occur, making trafficability of these soils difficult. If construction occurs during wet weather, McMillen Jacobs recommends measures be implemented to protect the fine-grained subgrade in areas of heavy construction traffic, as provided in Section 7.6.

It is important to note that these conditions may be initiated regardless of the time of year; even during the dry summer months. When initially exposed, these soils can appear to be competent and at their near-optimum moisture content. However, once disturbed by construction equipment and traffic, the residual soil / decomposed rock structure can be crushed, thereby releasing water trapped inside and rendering a once-competent material to a wet and sticky mass. Once the soil is rendered to this condition, it can be extremely difficult to compact, as well as for construction equipment to travel upon. If these conditions prevail during the dry summer months, we recommend implementing measures provided in Section 7.6 to facilitate earthwork activities.

6.0 Design Recommendations

6.1 Water Treatment Facility, ASR Well Houses, & Below-Grade Structures

We understand the new WTF will be a 50-foot by 54-foot, single-story building with a finished floor elevation of 379 feet, which will likely require cuts up to about 3 feet deep. We understand the new WTF structure will be lightly loaded and founded on a perimeter footing with slab-on-grade foundation system. The new WTF will be situated in the middle of an approximately 180-foot by 110-foot area that will be regraded/flattened and paved with asphalt.

Three new ASR wellhouses for ASR well Nos. 8 through 10 will each be approximately 900 square-foot single-story structures. We anticipate the pumphouses will be lightly loaded and founded on either a conventional perimeter footing with slab-on-grade foundation system or on a mat foundation with thickened edges.

In addition to the above-referenced at-grade structures, we anticipate there will be several ancillary below-grade structures, including buried vaults/manholes and pump stations. We anticipate the below grade structures will consist of either precast or cast-in-place reinforced concrete construction.

Foundation recommendations for both the at-grade and below-grade structures are provided in the following sections.

6.1.1 Subgrade Preparation

Satisfactory subgrade support for shallow foundations (for WTF and new ASR wellhouses) can be obtained from the native, medium stiff to stiff residual soil, or new structural fill that is properly placed and compacted on these soils during construction. As discussed in Section 5.1, the foundation subgrade soils are susceptible to shrink-swell behavior. To mitigate this behavior, we recommend overexcavating to a minimum depth of 2.5 feet below the adjacent finished grade elevation.

Once the overexcavations are complete, McMillen Jacobs should be contacted to observe subgrade conditions. The exposed foundation subgrade soils should not be exposed to period of wetting or drying but should be covered with geotextile fabric as soon as possible. The geotextile fabric should be in conformance with Section 7.3.8.

Overexcavations should be brought back to grade with imported structural fill in conformance with Section 7.3.2. The maximum particle size of overexcavation backfill should be limited to 1½ inches. All granular pads for footings should be constructed a minimum of 6 inches wider on each side of the footing for every vertical foot of overexcavation. To minimize the disturbance of the fine-grained subgrade, we recommend the use of lightweight excavation equipment with smooth-edged digging buckets. In addition, we recommend only static compaction with a small roller be used for the granular footing pads.

6.1.2 Minimum Footing Width and Embedment

Minimum footing widths should be in conformance with the 2019 OSSC. We recommend individual spread footings have a minimum width of 24 inches and continuous wall footings have a minimum width of 18 inches. All footings should be founded at least 18 inches below the lowest, permanent adjacent grade to develop lateral capacity and for frost protection.

6.1.3 Footing Bearing Pressure & Settlement

Footings founded as recommended above may be proportioned for an allowable soil bearing pressure of 2,000 pounds per square foot (psf), based on a factor of safety (FOS) of 3. This bearing pressure is a net bearing pressure (i.e., footing weight and overlying backfill are neglected), applies to the total of dead and long-term live loads, and may be increased by one-third when considering seismic or wind loads.

For foundations founded as recommended above, total settlement of foundations is anticipated to be less than 1 inch. We expect that differential static settlements across the foundation elements will be on the order of one-half of the total settlement.

6.1.4 Interior Floor Slabs

6.1.4.1 Subgrade Preparation

Satisfactory subgrade support for shallow foundations can be obtained from the native, medium stiff to better residual soil, or new structural fill that is properly placed and compacted on these soils during construction. If soft, loose, or otherwise unsuitable materials are encountered, they should be overexcavated as recommended by McMillen Jacobs during construction. Interior floor slabs should be founded upon a minimum 6-inch thick layer of structural fill that should be placed in one lift and compacted to at least 90 percent of the maximum dry density as determined by ASTM D1557.

6.1.4.2 Interior Floor Slab Design Parameters

A modulus of subgrade reaction value of 100 pounds per cubic inch (pci) may be used for slab-on-grade design. This subgrade modulus value represents the anticipated value, which would be obtained in a standard in situ plate test with a 1-foot square plate. Use of this subgrade modulus for design should include appropriate modifications based on dimensions as necessary.

6.1.4.3 Subgrade Moisture Considerations

Liquid moisture and moisture vapor should be anticipated at the subgrade surface. The recommended crushed rock base is anticipated to provide protection against liquid moisture. Where moisture vapor emission through the slab must be minimized (e.g., impervious floor coverings, storage of moisture sensitive materials and or equipment directly on the slab surface, etc.), a vapor retarding membrane or vapor barrier below the slab should be considered. Factors such as cost, special considerations for construction, floor coverings, and end use suggest that the decision regarding a vapor retarding membrane or vapor barrier be made by the designer and owner.

If a vapor retarder or vapor barrier is placed below the slab, its location should be based on current American Concrete Institute (ACI) guidelines, ACI 302 Guide for Concrete Floor and Slab Construction. In some cases, this indicates placement of concrete directly on the vapor retarder or barrier. It should be noted that the placement of concrete directly on impervious membranes increases the risk of plastic shrinkage cracking and slab curling in the concrete. Construction practices to reduce or eliminate such risk, as described in ACI 302, should be employed during concrete placement.

6.1.5 Lateral Earth Pressures

Backfill material placed behind the below-grade structures should consist of free-draining crushed aggregate, as described in Section 7.3.3. The following table summarizes our recommended lateral earth pressure values, expressed as the equivalent fluid pressures.

Table 6-1. Recommended Lateral Earth Pressures

Design Condition	Groundwater Condition[1]	Static At-rest Pressure (psf)	Static & Live Load Surcharge Pressure (psf)	Additional Seismic Pressure (psf)	Hydrostatic Pressure (psf)
At-Rest Earth Pressure	Above Groundwater	$50(H-H_W)$	0.40q	34H	--
	Below Groundwater	$50(H-H_W)+27H_W$	0.40q	34H	$62.4H_W$
Active Earth Pressure	Above Groundwater	$32(H-H_W)$	0.26q	17H	--
	Below Groundwater	$32(H-H_W)+18H_W$	0.26q	17H	$62.4H_W$

Notes:

1. We recommend a groundwater level of 5 feet bgs for the calculation of hydrostatic pressure.

H is the total height of the buried wall and H_W is the submerged portion of the buried wall (i.e., from the bottom of the buried wall up to the groundwater level). The above recommendations are valid only for imported, free-draining crushed aggregate and finished backfill slopes of flatter than 4H:1V (horizontal:vertical). The above earth pressures can be assumed to act horizontally on the embedded walls. The equivalent fluid earth pressures and seismic earth pressures increase with depth in a hydrostatic, triangular pressure distribution with the resultant force acting at approximately 0.3H above the base of the wall. The pressure distribution of the surcharge loads is a constant value of lateral pressure resulting from the vertical, surface surcharge loads (q) with the resultant lateral surcharge force acting approximately at a height above the base of the wall equal to one-half the total wall height. The distribution and resultant of the backfill, groundwater, and seismic earth pressure are shown in Figure 4.

6.1.6 Lateral Resistance

Lateral resistance for the below- and at-grade structure foundations can be provided by frictional resistance between the subgrade and the bottom of the foundations and by passive resistance around the footings. For the base frictional resistance, we recommend using an ultimate friction coefficient of 0.55 for cast-in-place concrete on prepared subgrade or structural fill. A coefficient of 0.45 may be used for pre-cast concrete foundations (i.e., vaults and manholes). Typically, a FOS of 1.5 is used to convert to allowable friction coefficients.

Lateral resistance can also be provided by passive resistance of the foundations. We recommend using an ultimate equivalent fluid pressure of 375 pounds per cubic foot (pcf) in the design of foundations. This resistance should be applied across the face of the foundation element. To develop full passive resistance, slight movement may first need to occur. Because of this, we recommend (1) neglecting using passive resistance in the upper 12 inches of the foundation element and (2) applying a FOS of 3 to the ultimate value (e.g., use a recommended passive earth pressure of 125 pcf).

6.1.7 Buoyancy

Buoyancy concerns are generally during periods of seasonal high groundwater levels and when below-grade structures are partially or entirely empty. We evaluated buoyancy potential of a precast concrete vault with 6-foot wide by 8-foot long by 7-foot deep outside dimensions with a 6-inch wall, base, and lid thickness and a groundwater level of 2 feet below the existing ground surface. Results of our buoyancy analysis showed a FOS exceeding 1.5.

Below-grade structures should be designed to resist buoyant uplift and lateral hydrostatic forces. Buoyant uplift is resisted by the dead weight of the structure, the weight of the backfill projected vertically above the outside edge of foundations extending beyond the vertical walls, and by frictional resistance between the structure and the backfill. The friction coefficients from Section 6.1.6 can be used for vertical frictional resistance along the earth/structure interface. If other below-grade structures are proposed, or the dimensions vary from the assumptions stated herein, McMillen Jacobs should be contacted to re-evaluate buoyancy potential.

6.1.8 Drainage Considerations

Because of the shrink-swell potential of the foundation subgrade soils, site drainage and stormwater management are of critical importance for this Project. We recommend the following implementations be maintained during the operation of the ASR Facilities:

- A minimum 5-foot wide hardscaping/concrete apron should extend around the perimeter of the buildings and should be sloped to drain away from the foundations in all directions;
- Roof downspouts and drains should discharge well beyond the limits of the building footprints;
- Any subsurface drains should be connected to the nearest storm drain or other suitable discharge point;
- Paved surfaces near the buildings should be slopes to drain away from the buildings; and
- Eliminate landscaping around the perimeter of the structures to prevent water infiltration into the foundation subgrade.

6.2 Pipelines

As discussed in Section 1.2, piping for the Project will include 10- to 24-inch DIP and 4- to 8-inch PVC. The 24-inch DIP will have a typical 48-inch depth of cover, while the depth-of-cover ranges from approximately 15 to 48 inches for the 10-inch DIP. The 50% Design Submittal drawings do not indicate a depth-of-cover for the PVC piping. The recommendations provided herein are for flexible (e.g., DIP and PVC) with a minimum depth of cover of 2 feet.

6.2.1 Pipeline Subgrade Support

We anticipate pipeline subgrade soils will consist of medium stiff to stiff, medium to high plasticity residual soils (e.g., silts and clays). Scattered zones of very soft soil will likely be encountered along the alignment which may require subgrade stabilization. Details of subgrade stabilization are provided in Section 7.3.6.

The new pipeline construction will not result in a net increase in pressure at the base of the pipeline, and therefore pipe settlement under static conditions is expected to be negligible.

6.2.2 Soil Design Parameters

Flexible pipes derive their load-carrying capacity from their interaction with the pipe zone backfill as the pipe deflects under load and pushes laterally against the soil. Load-carrying capacity depends on the depth of the pipe, the surrounding soil conditions, the type and density of the backfill, and the thickness of compacted pipe zone backfill between the pipe and the native soil in the trench wall. Based on the anticipated subsurface soil types and relative densities, the following geotechnical design parameters are recommended for pipeline design. These are provided in Table 6-2.

Table 6-2. Pipeline Geotechnical Design Parameters

Property	Depth of Cover (feet)[3]	Residual Soil	Granular Backfill	CLSM[1]
Moist Unit Weight, γ_m (pcf)	--	115	130	125
Saturated Unit Weight, γ_{sat} (pcf)	--	120	135	125
Friction Angle, ϕ (degrees)	--	28	36	34
Modulus of Soil Reaction, E' (psi)[2]	2≤D≤5	500	1,500	2,000
Soil/Steel Pipe Friction Coefficient, μ	--	0.2	0.4	0.4

Notes:
1. CLSM: Controlled Low Strength Material, Unit weight of CLSM may be specified by the designer; 125 pcf is typical value.
2. Modulus of soil reaction values are unfactored.
3. D: Depth of cover above top of pipe.

The design parameters presented in Table 6-2 are appropriate for use in the Iowa deflection formula (Spangler, 1941) and are consistent with American Water Works Association Manual M11 (2004). Note that the Modulus of soil reaction, E', is approximately equivalent to the constrained soil modulus, M_S.

The pipe should be designed considering traffic loads. These loads will vary depending on the final depth of the pipeline. Traffic loads are generally insignificant at depths of 10 feet or greater.

6.2.3 Pipeline Buoyancy and Flotation

When pipes are installed under the groundwater table, they can be susceptible to buoyancy if the upward buoyant forces on the pipe exceed the downward gravitational forces from the soil cover and the weight of the pipe. The following summarizes the results of our buoyancy and flotation analysis for the following pipes:

- 24-inch DIP with 48-inch depth of cover and groundwater at the ground surface. FOS against flotation exceeds 2.0;
- 10-inch DIP with 15-inch depth of cover and groundwater at the ground surface. FOS against flotation exceeds 2.0;

The results of our evaluation indicated factors of safety that either meet or exceed the typical minimum factor of safety of 1.5. It is important to note that we assumed the conditions shown in the 50% Design Submittal drawings in our buoyancy and flotation analyses. If the Project plans are revised, we should be provided with those revisions to revise our buoyancy and flotation analyses, if warranted.

6.3 Infiltration Facilities

We understand that the Project will incorporate green stormwater infrastructure (GSI) into the site design of the WTF. Accordingly, a stormwater infiltration facility is planned within the northwest of the paved area encompassing the WTF. Based on our review of the *Stormwater Design Handbook for Developers and Large Projects* (City of Salem, May 2014), we understand that if an infiltration rate is less than ½ inch per hour, partial infiltration facilities are allowed. Medium to high plasticity, elastic silt (MH) soils were encountered in the upper 5 feet in B-4, drilled at the proposed WTF site. Based on our experience with these types of soils, we anticipate that infiltration rates will be sufficiently less than ½ inch per hour (e.g., likely on the order of zero to 0.1 inch per hour). Therefore, we do not think that infiltration testing for the proposed stormwater infiltration facility is warranted. To be conservative, we recommend a design infiltration rate of zero inch per second be used.

6.4 Pavements

6.4.1 Subgrade preparation

Pavement subgrade surfaces should be crowned (or sloped) for proper drainage in accordance with specifications provided by the project civil engineer. It should be noted that any undocumented fill encountered within proposed pavement areas should be completely removed and replaced with structural fill in conformance with Section 7.3.2 of this report. The subgrade soils should be kept moist, at near-optimum moisture content, and not allowed to dry out. If surface cracking appears in the subgrade, the affected area should be overexcavated and replaced with structural fill.

After site preparation, but prior to placement of structural fill and/or aggregate base, McMillen Jacobs should be contacted to observe exposed subgrade conditions. If areas of soft soil or excessive yielding are identified, the affected material should be over-excavated to firm, stable subgrade, and replaced with structural fill in conformance with Section 7.3.2 of this report.

6.4.2 Flexible Pavement Design

6.4.2.1 Input Parameters

Design of the hot mixed asphaltic concrete (HMAC) pavement sections presented below was based on the parameters presented in Table 6-3, as well as parameters recommended in the following resources: City of Salem Department of Public Works Administrative Rules Design Standards (City of Salem, 2016); the American Association of State Highway and Transportation Officials (AASHTO) 1993 "Design of Pavement Structures" Manual (AASHTO, 1993), and the Asphalt Pavement Association of Oregon (APAO, 2003) Asphalt Paving Design Guide, and the ODOT Pavement Design (ODOT, 2011). If any of the items listed need revision, please contact us and we will reassess the provided design sections.

Table 6-3. Asphalt Pavement Design Parameters

Input Parameter	Design Value[1]
Pavement Design Life[2]	25 years
Annual Percent Growth	0 percent
Serviceability[2]	4.2 initial 2.5 terminal
Reliability[2]	90 percent
Standard Deviation[3]	0.49
Drainage Coefficient[2]	1.0 (Asphalt) 0.8 (Aggregate)

Input Parameter		Design Value[1]
Resilient Modulus	Subgrade (Native Elastic Silt)[4]	5,000 psi
	Crushed Aggregate Base[5]	22,500 psi
Structural Coefficient[2]	Crushed Aggregate Base	0.10
	Asphalt	0.41
Vehicle Traffic[6] (in ESALs)	Local (cul-de-sac)	10,000
	Local	100,000

Notes:

1. If any of the above parameters are incorrect, please contact us for revision, if warranted.
2. Per Section 6.24(e) of City of Salem Department of Public Works Administrative Rules Design Standards.
3. Per Section 5.3.3 of ODOT Pavement Design Guide.
4. Subgrade resilient modulus based on results of DCP testing. Results of DCP testing are presented in Appendix C.
5. Value based on our experience with similar compacted crushed rock fill materials.
6. ESAL = Total 18-Kip equivalent single axle load. Design values based on Table 6-23 in City of Salem Department of Public Works Administrative Rules Design Standards and based on anticipated street classification(s) for Project pavements.

6.4.2.1 Recommended Minimum Asphalt Pavement Sections

Table 6-4 presents the minimum asphalt pavement sections for the anticipated street classifications for the Project, based on the results of our pavement design calculations.

Table 6-4. Recommended Asphalt Pavement Sections

Material	City of Salem Street Classification	
	Local (Cul-de-Sac)	Local
Asphalt Pavement Thickness[1,2] (inches)	4	4
Crushed Aggregate Base (inches)	11	14
Subgrade Soils	Prepared in conformance with Section 6.4.1 of this report.	

Notes:

1. Pavement section thicknesses shown assume dry weather construction. Additional granular material or geo-grid reinforcement may also be recommended based on site conditions at the time of construction. See Section 7.6 for wet weather earthwork recommendations.
2. Pavement sections exceed minimum pavement sections for various street classifications in Table 6-24 in City of Salem Department of Public Works Administrative Rules Design Standards.

6.4.2.1 Asphalt Pavement Materials

Pavement aggregate base should consist of dense-graded aggregate in conformance with ODOT Section 02630.10, with the following additional considerations. We recommend the material consist of crushed rock or gravel, have a maximum particle size of 1½ inches, and have less than 5 percent material passing the U.S. Standard No. 200 Sieve. Aggregate base should be compacted to at least 95 percent of the material’s maximum dry density as determined in general accordance with ASTM D1557.

Asphalt pavement should consist of Level 2, ½-inch, dense-graded HMAC in conformance with ODOT Section 00745. Asphalt pavement should be compacted to at least 91 percent of the material’s theoretical maximum density as determined in general accordance with ASTM D2041 (Rice Specific Gravity), or as specified by the local jurisdiction.

7.0 Construction Recommendations

Construction recommendations for the Project are presented in the following sections. All material specifications referenced in this section referred to 2018 Oregon Standard Specifications for Construction (ODOT, 2018). A number of project details are still undefined at this level of design. McMillen Jacobs should review the design as it is advanced to confirm recommendations are applicable in the event facility layout and details change.

7.1 Site Preparation

7.1.1 Demolition

If applicable, demolition of any existing structures should include complete removal of all structural elements, including asphalt parking areas, foundations, and concrete slabs. Abandoned buried utilities should similarly be removed or grouted full. Concrete and asphalt debris resulting from demolition may be re-used as structural fill, provided it is processed in accordance with the recommendations presented in Section 7.3.1.2. Alternatively, demolition debris should be hauled off site for disposal.

7.1.2 Site Stripping

Vegetation, topsoil, and any undocumented fill encountered should be removed from the proposed building and pavement areas, and for a 5-foot-margin around such locations. Based on the results of our field explorations, stripping depths at the site are anticipated to extend up to approximately 2.5 feet bgs for the four proposed structures at the Project site (e.g., overexcavation depth for the footings). We anticipate stripping depths in pavement areas will generally be on the order of 1 foot, although this may increase in localized zones due to tree removal. These materials may be deeper or shallower at locations away from our explorations. The geotechnical engineer or his representative should provide recommendations for actual stripping depths based on observations during site stripping. Stripped topsoil and rooted soils should be transported off-site for disposal or stockpiled for later use in landscaped areas. Concrete and asphalt debris resulting from demolition may be stockpiled for later re-use as structural fill, as discussed in Section 7.3.1.2.

7.1.3 Existing Utilities & Below-Grade Structures

All existing utilities should be identified prior to excavation. Abandoned utility lines beneath the new buildings, pavements, and hardscaping features should be completely removed or grouted full. Soft, loose, or otherwise unsuitable soils encountered in utility trench excavations should be removed and replaced with structural fill in conformance with Section 7.3.2 this report. Buried structures (e.g., footings, foundation walls, slabs-on-grade, tanks, etc.), if encountered during site development, should be completely removed and replaced with structural fill in conformance with Section 7.3.2.

7.1.4 Erosion Control

Erosion and sedimentation control measures should be employed in accordance with applicable City, County, and State regulations.

7.1.5 Subgrade Preparation

Subgrade preparation for shallow foundations, interior floor slabs, and pavements are provided in Sections 6.1.1, 6.1.4.1, and 6.4.1, respectively. To minimize the disturbance of the fine-grained subgrade, we recommend the use of lightweight excavation equipment with smooth-edged digging buckets. McMillen Jacobs should observe the final subgrade surface, inspect the condition of the subgrade, and identify additional overexcavation as necessary. Because of the sensitive nature of the subgrade soils and the potential for disturbance, even in dry weather, we do not recommend evaluating subgrade conditions by proof rolling. Instead, we recommend subgrade probing be performed to evaluate subgrade conditions. This subgrade observation should occur prior to the subgrade being covered with crushed rock or formwork.

7.2 Pipeline Trench Excavation

We anticipate the maximum pipeline trench depths will be on the order of 5 feet. The final trench excavation should be performed with a straight-edged excavator bucket to minimize disturbance to the base of the trench. Following excavation, the trench base should be thoroughly cleaned of loosened or disturbed soils, by hand if necessary.

For pipe sizes up to 24 inches in diameter, the trench width should extend a minimum of 12 inches beyond each side of the pipe (i.e., OD + 24 inches + trench protection). Where trench shielding or shoring is used, the 12 inches should be measured between the pipe and inside face of the shielding or shoring. This will allow for the use of mechanical compaction equipment on the sides of the pipe.

7.3 Fill Materials and Compaction Criteria

7.3.1 On-Site Materials

7.3.1.1 Residual Soil

We do *not* recommend the re-use of the native residual soil as structural fill, due to its fine-grained, high-plasticity properties. In addition, these soils are extremely difficult to properly moisture condition and compact during construction; especially during the wet winter months.

7.3.1.2 Asphalt and Concrete Debris

Any asphalt and concrete debris resulting from the demolition of existing pavements and other features may be re-used as structural fill provided it can be processed into material that is reasonably well-graded. In general, the following recommendations should be considered when using the onsite asphalt and concrete debris as structural fill materials on the Project:

- All processed fill should be free of objectionable debris (clay clumps, organic and/or deleterious material, etc.) and within moisture contents suitable for compaction;
- All processed fill should have less than 15 percent material passing the U.S. Standard No. 200 Sieve as determined by ASTM D1140 (i.e., fines content);

- If encountered, cobbles, boulders, or other oversized particles greater than 4 inches should be removed from on-site materials considered for use as fill;
- Prior to filling operations, representative samples of each proposed fill type should be collected. Gradation tests (particle-size analysis) should be performed on the samples to evaluate their suitability for use as fill materials; and

If used as structural fill, the on-site materials should be prepared in conformance with the following paragraph.

7.3.2 Structural Fill

Structural fill materials should be placed after subgrade preparation and approval. Structural fill below new structure footprints should consist of either 1½-inch or ¾-inch minus Dense-Graded Aggregates conforming to ODOT Section 02630.10. Unless otherwise noted, structural fill should be compacted to 90 percent of ASTM D1557. The structural fill should be placed in maximum lifts of 8 inches of loose material. Each lift of compacted engineered fill should be tested by a qualified testing agency prior to placement of subsequent lifts. This fill condition should extend laterally beyond the exterior perimeter of the building foundation a distance equal to the thickness of the fill or 3 feet; whichever is less.

7.3.3 Embedded Walls and Below-Grade Structure Backfill

Backfill for embedded walls and below-grade structures should consist of imported structural fill as described in Section 7.3.2 and contain less than 5 percent fines (e.g., passing the U.S. Standard No. 200 Sieve). This material should be compacted to a minimum of 90 percent of the maximum dry density, as determined by ASTM D1557. Backfill placed within 3 feet of the below-grade structures should be compacted in lifts less than 6-inches thick using hand-operated tamping equipment (e.g., jumping jack or vibratory plate compactors). If flat work (e.g., concrete slabs or pavements) will be placed atop the wall backfill, we recommend that the upper 2 feet of material be compacted to 92 percent of the maximum dry density, as determined by ASTM D1557.

7.3.4 Pipe Bedding and Pipe Zone Backfill

We recommend that pipe bedding and pipe zone in the trench be constructed with imported, well-graded crushed rock material, such as ¾-inch minus crushed aggregate base material. The material must be suitable for compaction and able to be worked under the curvature of the pipe. We recommend a minimum bedding thickness of 6 inches below the bottom of the pipe, or as determined by the pipeline designer. In areas where weak subgrade is encountered, a foundation stabilization layer should be placed below the bedding. Foundation stabilization is discussed in Section 7.3.6.

Above pipe bedding, imported crushed rock aggregate should be used for backfill within the pipe zone, which typically extends at least 12 inches above the top of the pipe, or as determined by the pipeline designer.

Pipe bedding and pipe zone backfill should be compacted to 90 percent of the maximum dry density, as determined by ASTM D1557, except the portion directly below the pipe that should be leveled without compaction. This will allow for uniform pressure distribution under the pipe invert.

7.3.5 CLSM Backfill

Controlled Low Strength Material (CLSM) is commonly used as an alternative to granular fill bedding in portions of pipelines. CLSM fill mixtures are typically composed of a combination of cement, water, fine aggregate, and fly ash. The material is flowable and self-leveling, which greatly simplifies placement around pipelines. The material typically is specified to have unconfined compressive strength of 50 to 200 psi.

7.3.6 Foundation Stabilization

Based on the subsurface explorations across the alignment, we anticipate competent subgrade conditions at the bottom of the trench. However, the subgrade soils can be disturbed if left exposed to water or general construction activities. If the subgrade becomes weakened or if soft/wet subgrade is encountered in localized areas due to perched groundwater, a foundation stabilization layer may be required.

To construct the foundation stabilization layer, the trench should be overexcavated a minimum 12 inches below the bottom of the bedding and replaced with the foundation stabilization layer. The foundation stabilization layer should consist of compacted, free-draining aggregate consisting of 1-½ to ¾- inch conforming with the requirements of ODOT Section 00430.11. Vibratory compaction equipment is not recommended due to risk of additional disturbance to the subgrade. A geotextile should be used below the aggregate as described in Section 7.3.8. The foundation stabilization backfill may also be used as the drainage layer for in-trench dewatering, as described in Section 7.3.6.

7.3.7 Pavement Materials

Asphalt pavement section materials are discussed in Section 6.4.2.1.

7.3.8 Geotextiles

7.3.8.1 Separation Geotextiles

In general, the widespread use of separation geotextiles is not anticipated for the Project. However, they may be required in localized areas of trench seepage or for protection of subgrade, or in other areas identified during construction. They are not required for typical trench construction, however if used, separation geotextiles should consist of a “needle-punched”, non-woven separation fabric meeting the requirements for Type 1, nonwoven drainage geotextiles, as shown in Table 02320-1 in ODOT Section 02320.

7.3.8.2 Reinforcement Geotextiles

A reinforcement geotextile system should be installed beneath foundation stabilization backfill. We recommend a single-layer system consisting of a strong geotextile, such as Mirafi RS380i, that provides both separation/filtration and reinforcement. The reinforcement/separation geotextile should be installed on the base of the trench and extend up to the top of the foundation stabilization zone (below bedding) at a minimum. Reinforcement geotextiles should meet the requirements for Type 2, woven riprap geotextiles, as shown in Table 02320-2 in ODOT Section 02320.

7.3.9 Trench Backfill

Trench backfill refers to the fill placed above the pipe zone. Where the pipeline is located below paved areas or structures, trench backfill should consist of pipe zone material, per Section 7.3.4. Trench backfill beneath paved areas or structures should be placed in 12-inch maximum loose lifts and compacted to at least 92 percent of the maximum dry density, as determined by ASTM D1557. The trench backfill should be placed up to the design top-of-subgrade elevation associated with the final pavement section.

For areas outside of the roadways, trench backfill may consist re-processed asphalt and concrete debris, if used for this Project, or imported structural fill. Trench backfill in these areas should be compacted to 90 percent of maximum dry density, as determined by ASTM D1557. The upper 18 inches of the trench should be backfilled with topsoil to allow for vegetation regrowth.

7.4 Temporary Excavations

All excavations should be in accordance with applicable OSHA and state regulations. It is the contractor's responsibility to select the excavation methods, to monitor site excavations for safety, and to provide any shoring required to protect personnel and nearby, existing structures. A competent person, as defined by Oregon OSHA, is an individual that can identify existing and predictable excavation-related hazards and has the authority to take prompt corrective measures to eliminate such hazards. McMillen Jacobs' Project role does not include review or oversight of excavation safety.

We anticipate the pipeline trenches will be on the order of 5 feet deep and excavated using a vertical shoring system. No information has been provided to date regarding the depth of other ancillary structures, such as vaults/manholes and pump stations. For the purposes of this report, we assume a maximum excavation depth of 10 feet. In the case that cut slopes are utilized for any excavations, the maximum slope inclinations must be in accordance with OSHA regulations. For use in the planning and construction of temporary excavations up to 10 feet in depth, an OSHA soil type of “B” can be used for the predominately fine-grained, near-surface residual soils encountered.

Temporary slope recommendations do not consider site constraints such as groundwater, surcharge, or nearby structures. Temporary slopes should be evaluated on a case-by-case basis and incorporate groundwater conditions, soil classification, and site constraints. Slopes should be inspected and maintained as required by OSHA.

With time and the presence of seepage and precipitation, the stability of temporary unsupported cut slopes can be significantly reduced. Therefore, temporary slopes kept open for construction activities should be protected from erosion by installing a surface water diversion ditch or berm at the top of the slope and covering the cut face with well-anchored plastic sheets. In addition, the contractor should monitor the stability of the temporary cut slopes and adjust the construction schedule and slope inclination accordingly. Maintenance of safe working conditions, including temporary excavation stability, is the responsibility of the contractor and all excavations must comply with current federal, state, and local requirements.

7.5 Groundwater Control

Based on our groundwater measurements, we anticipate groundwater could be as shallow as 5 feet bgs. Based on their low hydraulic conductivity, the medium to high plasticity, fine-grained soils are not anticipated to produce large volumes of groundwater. Therefore, we anticipate that groundwater inflow can be controlled with a well-constructed, sump pumping dewatering system. Sump pumps should be installed with close spacing to maintain water levels below the subgrade surface. In case that large volumes of water seepage are encountered, perforated drainpipes installed in drainage layers (i.e., crushed rock) may be necessary to convey water to the sump pump systems.

7.6 Wet Weather Earthwork

The soils encountered within the project area are highly moisture sensitive and will degrade after being traversed by construction equipment during periods of wet weather or wet conditions. Therefore, during or after wet weather, it will likely be necessary to import granular materials for structural fill or to protect exposed subgrade materials. Delays in site earthwork activities should be anticipated during periods of heavy rainfall. If earthwork is performed during extended periods of wet weather or in wet conditions, we recommend the following:

- Cover the base of trenches within soil with trench stabilization material.
- Haul roads subjected to repeated, heavy, tire-mounted construction traffic (e.g., dump trucks, concrete trucks, etc.) will require a minimum of 18 inches of imported granular material to facilitate traffic. Additional granular material or geo-grid reinforcement may also be recommended based on site conditions at the time of construction.
- Excavations should be protected from surface water runoff by placing sandbags or by other means to direct runoff of precipitation away from work areas and to prevent ponding of water in excavations.
- Plastic covers, sloping, ditching, sumps, dewatering, and other measures should be employed in work areas as necessary to permit timely completion of work. Bales of straw and/or geotextile silt fences should be used to control surface soil movement and erosion.
- Excavations (specifically trench excavations) should be completed in small sections and backfilled at the end of each day to reduce exposure to wet conditions.
- Excavation or the removal of unsuitable soil should be followed promptly by placement and compaction of trench or foundation stabilization fill.
- The size and type of construction equipment used may have to be limited to minimize soil disturbance.

8.0 Closure

This report has been prepared for the exclusive use of the City of Salem and Murraysmith, in connection with the City of Salem – Improvements to the Aquifer Storage and Recovery Facility project. The data presented in this report is based on the subsurface conditions encountered during our site explorations and previous geotechnical exploration conducted nearby. The data presented herein is intended to support the design of the proposed improvements. McMillen Jacobs Associates is not responsible for the interpretation of the data contained in this report by anyone; as such interpretations are dependent on each person's subjectivity.

In the performance of geotechnical work, specific information is obtained at specific locations at specific times, and geologic conditions can change over time. It should be acknowledged that variations in soil conditions may exist between exploration and exposed locations and this report does not necessarily reflect variations between different explorations. The nature and extent of variation may not become evident until construction. If, during construction, conditions observed or encountered differ from those disclosed by this report, McMillen Jacobs Associates should be advised at once so we can observe and review these conditions and reconsider our recommendations where necessary.

The geotechnical engineering evaluations and interpretations included in this report are completed within the limitations of McMillen Jacobs Associates approved scope of work, schedule and budget. The services rendered by McMillen Jacobs Associates have been performed in a manner consistent with the level of care and skill ordinarily exercised by members of the profession currently practicing under similar conditions in the same area. The construction recommendations are considered preliminary and provided for planning purposes only. McMillen Jacobs Associates is not responsible for the use of this report in connection with anything other than the project at the location described above.

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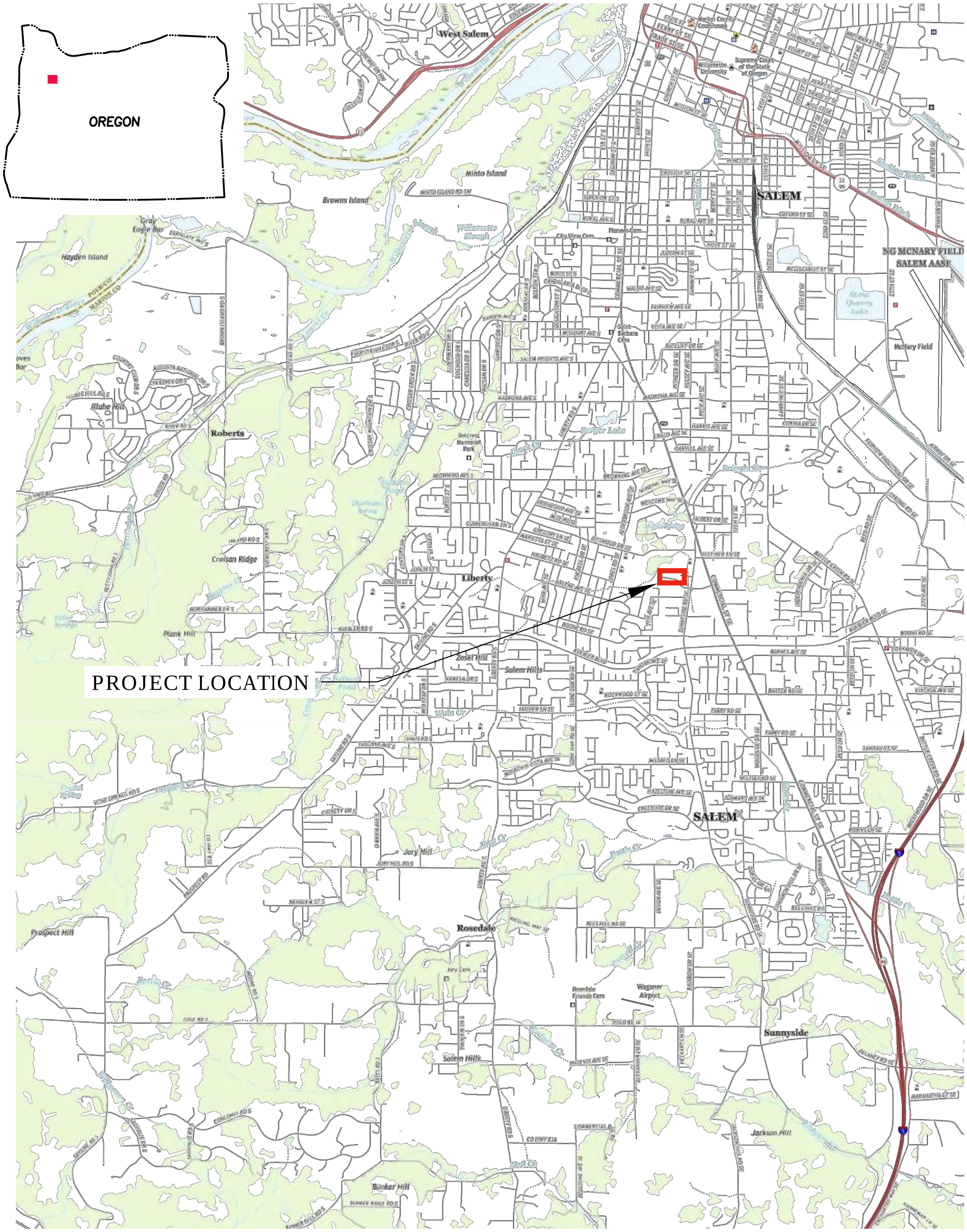
Figures



PROJECT VICINITY MAP

SCALE: 1"=1MILE 0 1 MILE 2 MILES

N

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CITY OF SALEM

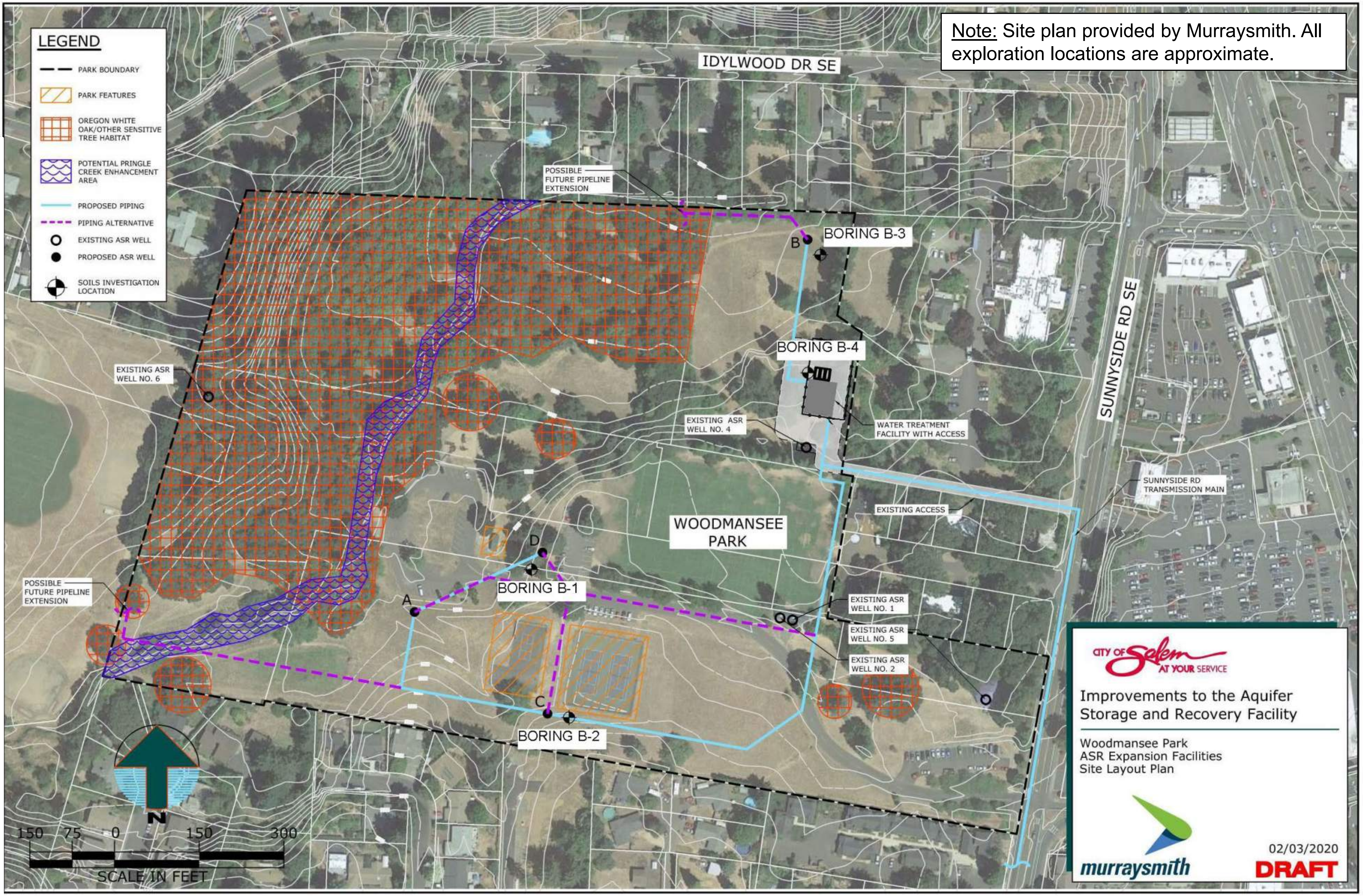
IMPROVEMENTS TO THE ASR FACILITY

GEOTECHNICAL ENGINEERING REPORT
PROJECT VICINITY MAP

FIG.1

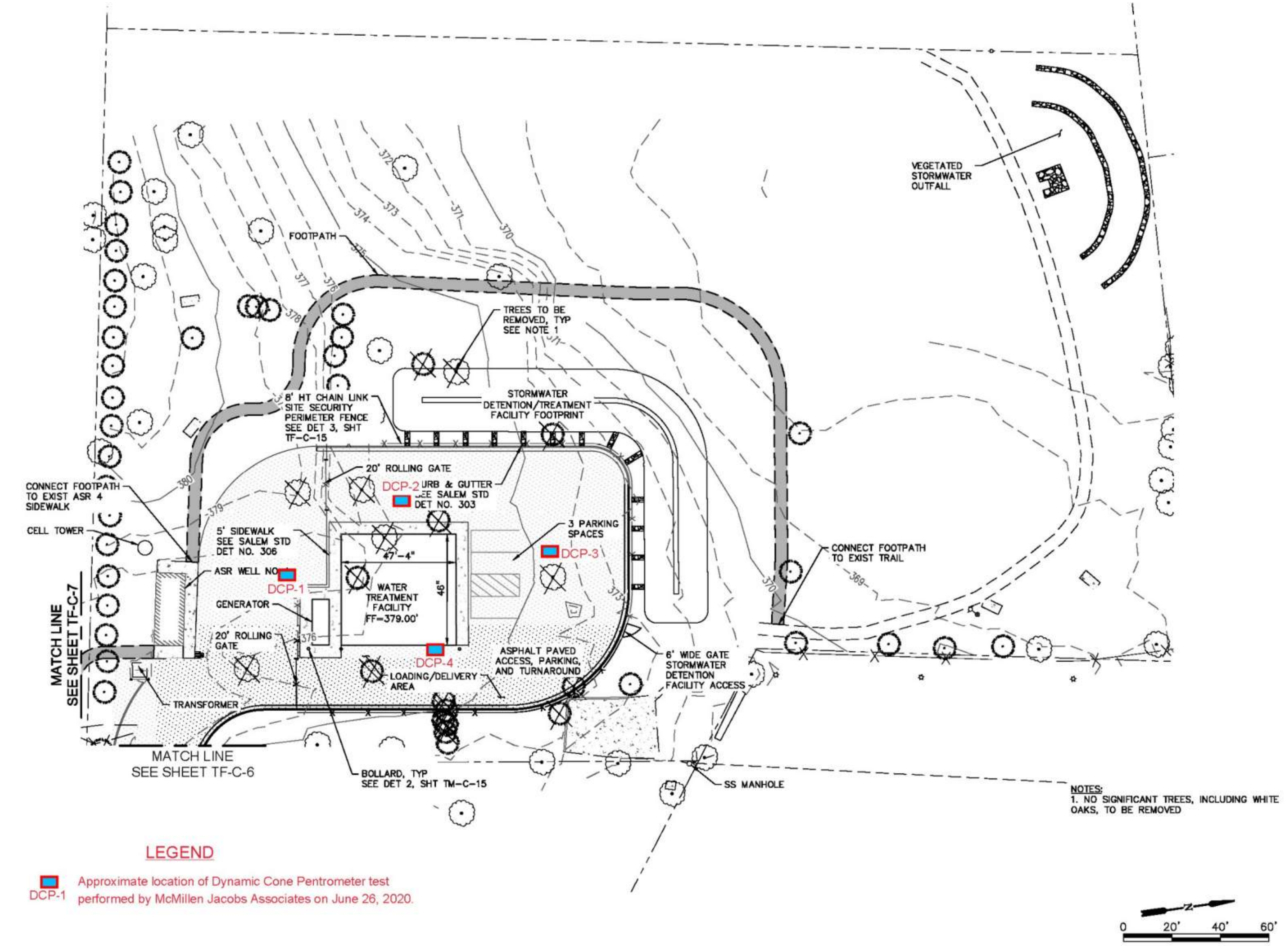
JULY 2020

Path: C:\Users\cburke\Desktop\Vicinity Map\Fig.1_Salem.dwg Plot date: May 28, 2020, CAD User: cburke



Note: Site plan provided by Murraysmith. All exploration locations are approximate.

McMILLEN JACOBS ASSOCIATES	CITY OF SALEM – IMPROVEMENTS TO THE ASR FACILITY SALEM, OREGON	FIGURE 2
	EXPLORATION PLAN	JULY 2020



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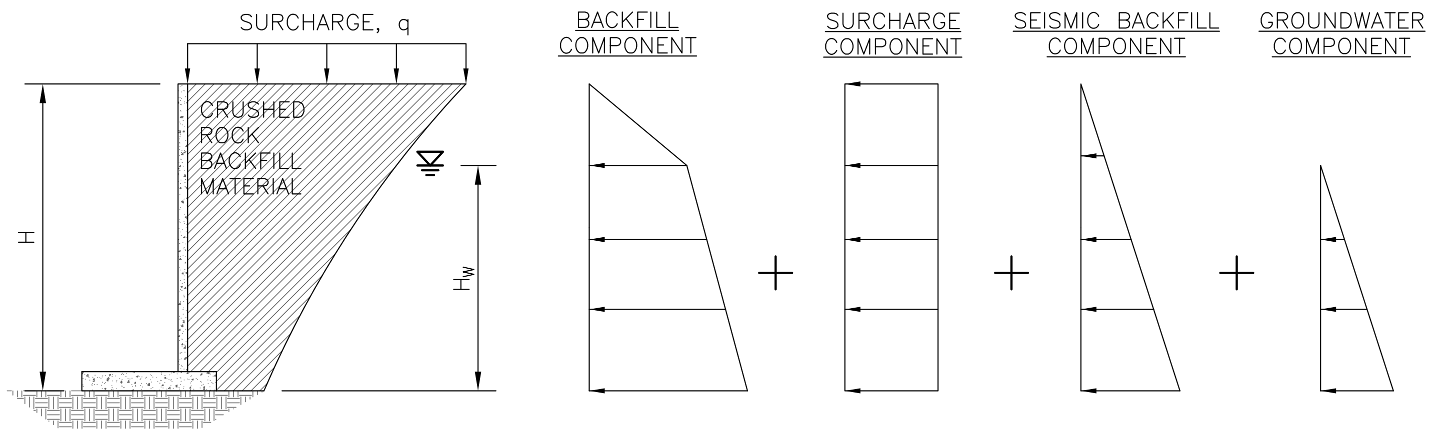
CITY OF SALEM – IMPROVEMENTS TO THE ASR FACILITY
SALEM, OREGON

FIGURE
3

DYNAMIC CONE PENETROMETER TEST EXPLORATION PLAN

JULY
2020

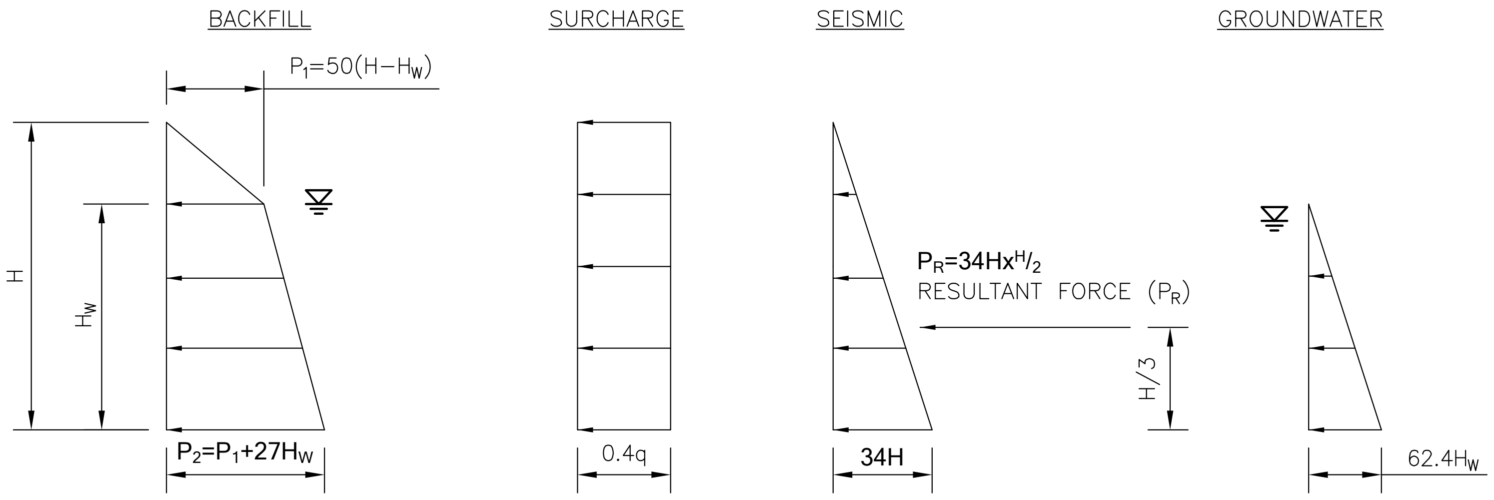
LATERAL EARTH PRESSURES ON EMBEDDED WALLS & STRUCTURES



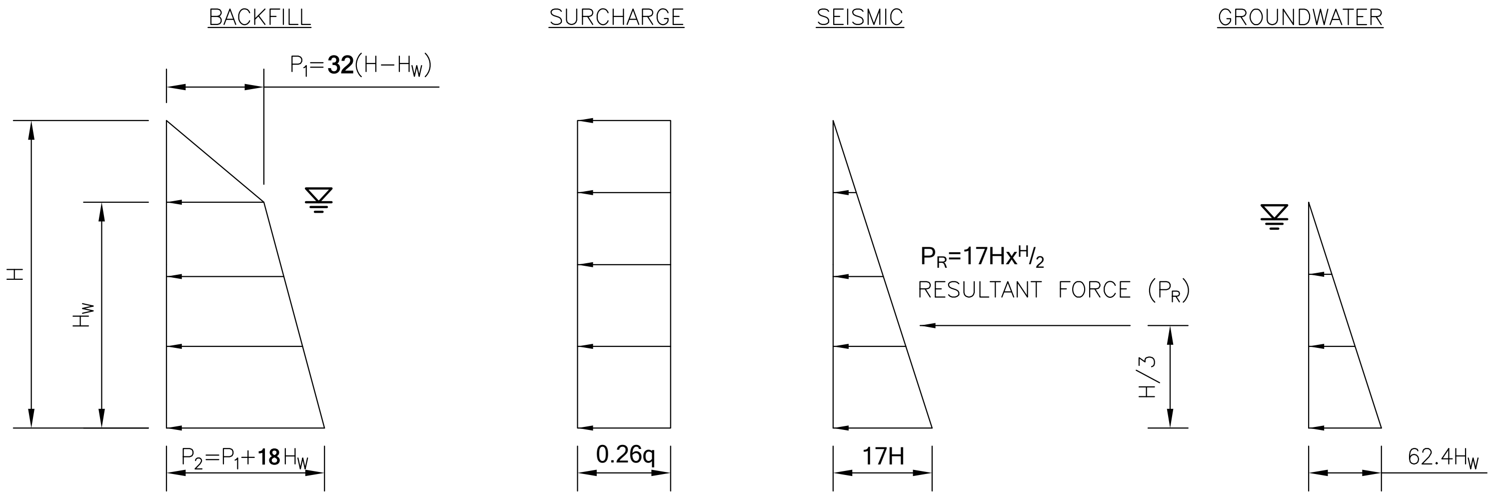
NOTES:

1. UNITS ARE POUNDS PER SQUARE FOOT (PSF).
2. BACKFILL PRESSURES BASED ON IMPORTED CRUSHED ROCK.

RESTRAINED (NON–YIELDING) EMBEDDED WALLS & STRUCTURES



NON-RESTRAINED (YIELDING) EMBEDDED WALLS & STRUCTURES



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CITY OF SALEM
IMPROVEMENTS TO THE ASR FACILITY
GEOTECHNICAL ENGINEERING REPORT
LATERAL EARTH PRESSURES FOR EMBEDDED WALLS

FIG.4

JULY 2020

Appendix A

Boring Logs

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Key to Boring Logs - Soil

UNIFIED SOIL CLASSIFICATION SYSTEM (USCS based on ASTM D2488)

MAJOR DIVISIONS			GROUP/SYMBOL		TYPICAL DESCRIPTION
COARSE-GRAINED SOILS (50% or more retained on No. 200 sieve)	GRAVELS (more than 50% retained on No. 4 sieve)	CLEAN GRAVELS (less than 5% fines)	GW	▦	WELL-GRADED GRAVEL
			GP	▦	POORLY GRADED GRAVEL
		GRAVELS (with 5 to 12% fines)	GW-GM	▦	WELL-GRADED GRAVEL WITH SILT
			GW-GC	▦	WELL-GRADED GRAVEL WITH CLAY
			GP-GM	▦	POORLY GRADED GRAVEL WITH SILT
			GP-GC	▦	POORLY GRADED GRAVEL WITH CLAY
		GRAVELS WITH FINES (more than 12% fines)	GM	▦	SILTY GRAVEL
			GC	▦	CLAYEY GRAVEL
	SANDS (less than 50% retained on No. 4 sieve)	CLEAN SANDS (less than 5% fines)	SW	░	WELL-GRADED SAND
			SP	░	POORLY GRADED SAND
		SANDS (with 5 to 12% fines)	SW-SM	░	WELL-GRADED SAND WITH SILT
			SW-SC	░	WELL-GRADED SAND WITH CLAY
			SP-SM	░	POORLY GRADED SAND WITH SILT
			SP-SC	░	POORLY GRADED SAND WITH CLAY
		SANDS WITH FINES (more than 12% fines)	SM	▥	SILTY SAND
			SC	▨	CLAYEY SAND
FINE-GRAINED SOILS (50% or more passes No. 200 sieve)	SILTS & CLAYS (liquid limit less than 50)	INORGANIC	ML	▥	SILT
			CL	▨	LEAN CLAY
		ORGANIC	OL	▨	LOW PLASTICITY ORGANIC CLAY
	SILTS AND CLAYS (liquid limit greater than 50)	INORGANIC	MH	▥	ELASTIC SILT
			CH	▨	FAT CLAY
		ORGANIC	OH	▨	HIGH PLASTICITY ORGANIC CLAY
	SILT/CLAY (liquid limit 12 -25; PI 4-7)	INORGANIC	CL-ML	▨	CLAYEY SILT/SILTY CLAY
HIGHLY ORGANIC SOILS	PRIMARILY ORGANIC MATTER		PT	⁂	PEAT

Note:

Dual symbols (symbols separated by a hyphen, e.g. SP-SM) are used for soils between 5% and 12% fines or when liquid limit and plasticity index values plot in the CL-ML area of the plasticity chart.

Coare-Grained Soils

Relative Density	N, SPT Blows/Foot
Very Loose	0 - 4
Loose	5 - 10
Medium Dense	11 - 30
Dense	31 - 50
Very Dense	> 50

Fine-Grained Soils

Relative Consistency	N, SPT Blows/Foot
Very Soft	< 2
Soft	2 - 4
Medium Stiff	5 - 8
Stiff	9 - 15
Very Stiff	16 - 30
Hard	> 30

Abbreviations

AL	Atterberg Limit
MC	Moisture Content
SA	Sieve Analysis
LL	Liquid Limit
PL	Plastic Limit

Sample Symbols

⧗	SPT Sample 2" OD
■	Shelby Tube Sample
✋	Grab Sample
N	Blows/ft

Well and Backfill Symbols

▦	Bentonite Chips
▩	Concrete
░	Sand
■	Asphalt
⁘	Gravel
▧	Grout
◫	Observation Well - Solid Interval
▤	Observation Well - Screened Interval
▮	Vibrating Wire Piezometer
▼	Measured groundwater level

AL/MC Symbols

☐	Blows/Ft
○	Moisture Content
⊢⊣	Liquid Limit/Plastic Limit

Modifiers & Percentages

Trace	Component is present at less than 5% of the less than 3-inch portion.
Few (Sand or Gravel)	Coarse particles present at levels estimated at 6-10%.
With (Sand or Gravel)	Coarse particles present at levels estimated at 15-30%.
Sandy or Gravelly	Coarse particles present at levels estimated at 30-50%.

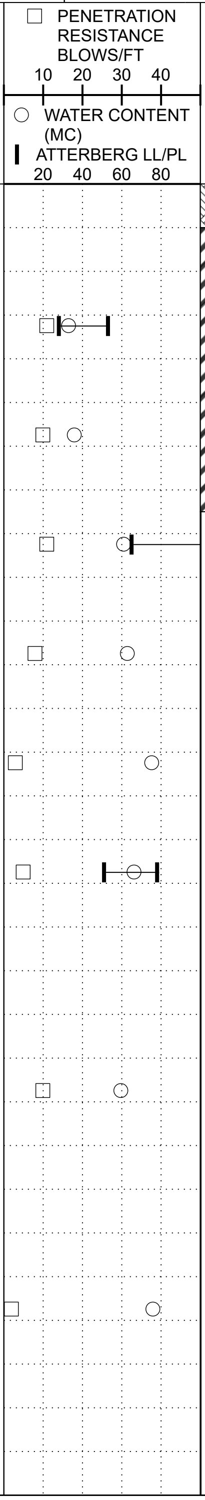
Moisture Content

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

Project: City of Salem ASR Facility Improvements
Project Location: Woodmansee Park
Project Number: 6129.0

Log of Boring B-1

Date(s) Drilled	**04/06/2020**	Geotechnical Consultant	**McMillen Jacobs Associates**	Logged By	**A. Judy**	Checked By	**J. Quinn**
Drilling Method/ Rig Type	**8.25" Hollow Stem Auger/CME 55 Track Mounted**	Drilling Contractor	**Western States Soil Conservation, Inc.**	Total Depth of Borehole	**51.5 ft**		
Hole Diameter	**8.25 in**	Hammer Weight/Drop (lb/in.)/Type	**140 lb / 30 in / Automatic**	Ground Surface Elevation/Datum	**380.0 ft**		
Location	**West of center parking lot.**	Coordinates	**7544037.26E,455771.40N**	Elevation Source	**Google Earth**		



ELEV. (FT)	DEPTH (FT)	RECOVERY (%)	BLOW COUNTS	SAMPLE NUMBER	USCS	MATERIAL DESCRIPTION	REMARKS AND TESTS
						Moist, dark brown, SILT (ML).	Topsoil thickness estimated from auger cuttings.
		93	2-4-7 (N=11)	S-1	CH	Stiff, moist, red-brown, FAT CLAY (CH); medium plasticity, trace black and yellow mottles, with occasional fine-gravel-sized clasts of decomposed rock, relict rock fabric present. **Residual Soil of CRB**	
375	5	80	3-5-5 (N=10)	S-2			
		100	3-5-6 (N=11)	S-3	MH	Stiff, moist, red with light brown and yellow mottling, ELASTIC SILT (MH); high plasticity, with frequent fine to coarse gravel-sized highly weathered to decomposed rock, relict rock fabric present. **Residual Soil of CRB**	
370	10	100	3-4-4 (N=8)	S-4		*At 10 feet, grades to medium stiff, with black mottling.*	
		100	1-1-2 (N=3)	S-5		*At 12.5 feet, grades to soft, medium plasticity, becomes wet.*	
365	15	100	1-2-3 (N=5)	S-6		*At 15 feet, grades to medium stiff, color becomes red with light brown and black mottles.*	
360	20	100	2-4-6 (N=10)	S-7		*At 20 feet, grades to stiff, becomes red with trace black and yellow mottles, trace fine, moderately weathered gravel.*	
355	25	100	1-0-2 (N=2)	S-8		*At 25 feet, grades to soft, color becomes light brown-gray with red mottles; interfingered with SILT to Sandy SILT (ML).*	S-8 25 - 26.5 ft. 74% Fines by ASTM D1140.

McMILLEN JACOBS ASSOCIATES

Boring B-1

Project: City of Salem ASR Facility Improvements
Project Location: Woodmansee Park
Project Number: 6129.0

Log of Boring B-1

Date(s) Drilled	**04/06/2020**	Geotechnical Consultant	**McMillen Jacobs Associates**	Logged By	**A. Judy**	Checked By	**J. Quinn**
Drilling Method/ Rig Type	**8.25" Hollow Stem Auger/CME 55 Track Mounted**	Drilling Contractor	**Western States Soil Conservation, Inc.**	Total Depth of Borehole	**51.5 ft**		
Hole Diameter	**8.25 in**	Hammer Weight/Drop (lb/in.)/Type	**140 lb / 30 in / Automatic**	Ground Surface Elevation/Datum	**380.0 ft**		
Location	**West of center parking lot.**	Coordinates	**7544037.26E,455771.40N**	Elevation Source	**Google Earth**		

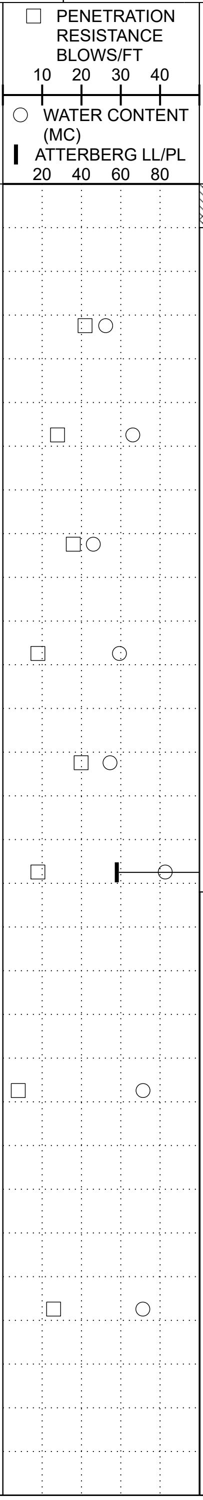
ELEV. (FT)	WATER LEVEL	DEPTH (FT)	SAMPLE TYPE	RECOVERY (%)	BLOW COUNTS	SAMPLE NUMBER	☐ PENETRATION RESISTANCE BLOWS/FT 10 20 30 40 ○ WATER CONTENT (MC) ❙ ATTERBERG LL/PL 20 40 60 80	USCS GRAPHIC	USCS	MATERIAL DESCRIPTION	REMARKS AND TESTS	BACKFILL/INSTALL.
			⧗	100	8-10-12 (N=22)	S-9	☐○			Very stiff, moist, light brown with red-brown mottles, Gravelly SILT with sand (ML); non-plastic to low plasticity, coarse gravel-sized, subangular, highly weathered to decomposed rock, trace black mottles, relict rock fabric present. **Residual Soil of CRB**		
345		35	⧗	87	9-7-9 (N=16)	S-10	☐ ○			*At 35 feet, becomes moist to wet; color becomes red-brown.*		
340		40	⧗	100	16-15-22 (N=37)	S-11	☐		ML	*At 40 feet, grades to hard; becomes moist.*		
335		45	⧗	100	19-27-26 (N=53)	S-12	☐					
330		50	⧗	100	10-18-19 (N=37)	S-13	☐					
											Borehole completed at 51.5 feet below ground surface (bgs).	
325		55										

Project: City of Salem ASR Facility Improvements
Project Location: Woodmansee Park
Project Number: 6129.0

Log of Boring B-2

Date(s) Drilled	04/06/2020	Geotechnical Consultant	McMillen Jacobs Associates
Logged By	A. Judy	Checked By	J. Quinn
Drilling Method/ Rig Type	Mud Rotary/CME 55 Track Mounted	Drilling Contractor	Western States Soil Conservation, Inc.
Total Depth of Borehole	41.5 ft	Hole Diameter	3.88 in
Hammer Weight/Drop (lb/in.)/Type	140 lb / 30 in / Automatic	Ground Surface Elevation/Datum	387.0 ft
Location	South of tennis courts.	Coordinates	7544108.04E,455515.67N
Elevation Source	Google Earth		

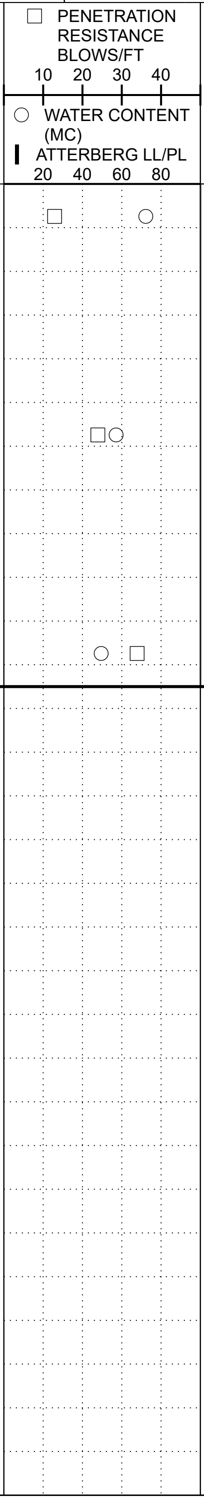
ELEV. (FT)	WATER LEVEL	DEPTH (FT)	SAMPLE TYPE	RECOVERY (%)	BLOW COUNTS	SAMPLE NUMBER	USCS	MATERIAL DESCRIPTION	REMARKS AND TESTS	BACKFILL/INSTALL.
								Soft, moist, dark brown, SILT (ML).	Topsoil thickness estimated from auger cuttings.	▨
							ML	Very stiff, moist, red-brown with black mottles, SILT with sand (ML); non-plastic, trace fine to coarse-gravel-sized slightly weathered to fresh, subangular rock, fine sand, relict rock fabric present. **Residual Soil of CRB**		
			⧗	100	7-10-11 (N=21)	S-1				
382		5	⧗	100	8-4-10 (N=14)	S-2		*At 5 feet, grades to stiff, becomes orange and wet.*		
			⧗	100	8-8-10 (N=18)	S-3		*At 7.5 feet, grades to Gravelly SILT with sand (ML); low plasticity, fine to coarse-gravel-sized, angular, highly weathered to decomposed rock, fine to coarse sand. becomes orange brown with black mottling.*		
377		10	⧗	100	3-3-6 (N=9)	S-4		*At 10 feet, grades to stiff, becomes moist to wet, trace black mottles present.*		
			⧗	100	6-9-11 (N=20)	S-5		*At 12.5 feet, grades to very stiff, becomes red-brown with gray mottling.*		
372		15	⧗	100	5-4-5 (N=9)	S-6		*At 15 feet, grades to stiff.*		
							MH	Stiff, moist, red-brown, ELASTIC SILT (MH); high plasticity, frequent decomposed rock fragments, relict rock fabric present. **Residual Soil of CRB**		
367		20	⧗	100	2-1-3 (N=4)	S-7		*At 20 feet, grades to soft.*		
362		25	⧗	100	3-5-8 (N=13)	S-8		*At 25 feet, grades to stiff, medium plasticity; interfingered with non-plastic, light brown, SILT (ML).*		



Project: City of Salem ASR Facility Improvements
Project Location: Woodmansee Park
Project Number: 6129.0

Log of Boring B-2

Date(s) Drilled	**04/06/2020**	Geotechnical Consultant	**McMillen Jacobs Associates**	Logged By	**A. Judy**	Checked By	**J. Quinn**
Drilling Method/ Rig Type	**Mud Rotary/CME 55 Track Mounted**	Drilling Contractor	**Western States Soil Conservation, Inc.**	Total Depth of Borehole	**41.5 ft**		
Hole Diameter	**3.88 in**	Hammer Weight/Drop (lb/in.)/Type	**140 lb / 30 in / Automatic**	Ground Surface Elevation/Datum	**387.0 ft**		
Location	**South of tennis courts.**	Coordinates	**7544108.04E,455515.67N**	Elevation Source	**Google Earth**		



ELEV. (FT)	WATER LEVEL	DEPTH (FT)	SAMPLE TYPE	RECOVERY (%)	BLOW COUNTS	SAMPLE NUMBER	USCS GRAPHIC	USCS	MATERIAL DESCRIPTION	REMARKS AND TESTS	BACKFILL/INSTALL.
				100	3-1-12 (N=13)	S-9			Stiff, wet, red-brown, Gravelly SILT with sand (ML); non-plastic, fine to coarse gravel-sized, angular to subangular, highly weathered to decomposed rock, fine to coarse sand, trace black and yellow mottling, relict rock fabric present. **Residual Soil of CRB**		
352		35		100	6-9-15 (N=24)	S-10		ML	*At 35 feet, grades to very stiff, gray-brown with trace black and orange mottling, Sandy SILT (ML); low plasticity, fine to coarse sand.*	S-10 35 - 36.5 ft. 57.3% Fines by ASTM D1140.	
347		40		100	10-13-21 (N=34)	S-11			*At 40 feet, grades to hard, becomes gray with light brown and black mottles, trace yellow mottles.*		
										Borehole completed at 41.5 feet below ground surface (bgs).	
342		45									
337		50									
332		55									

McMILLEN JACOBS ASSOCIATES

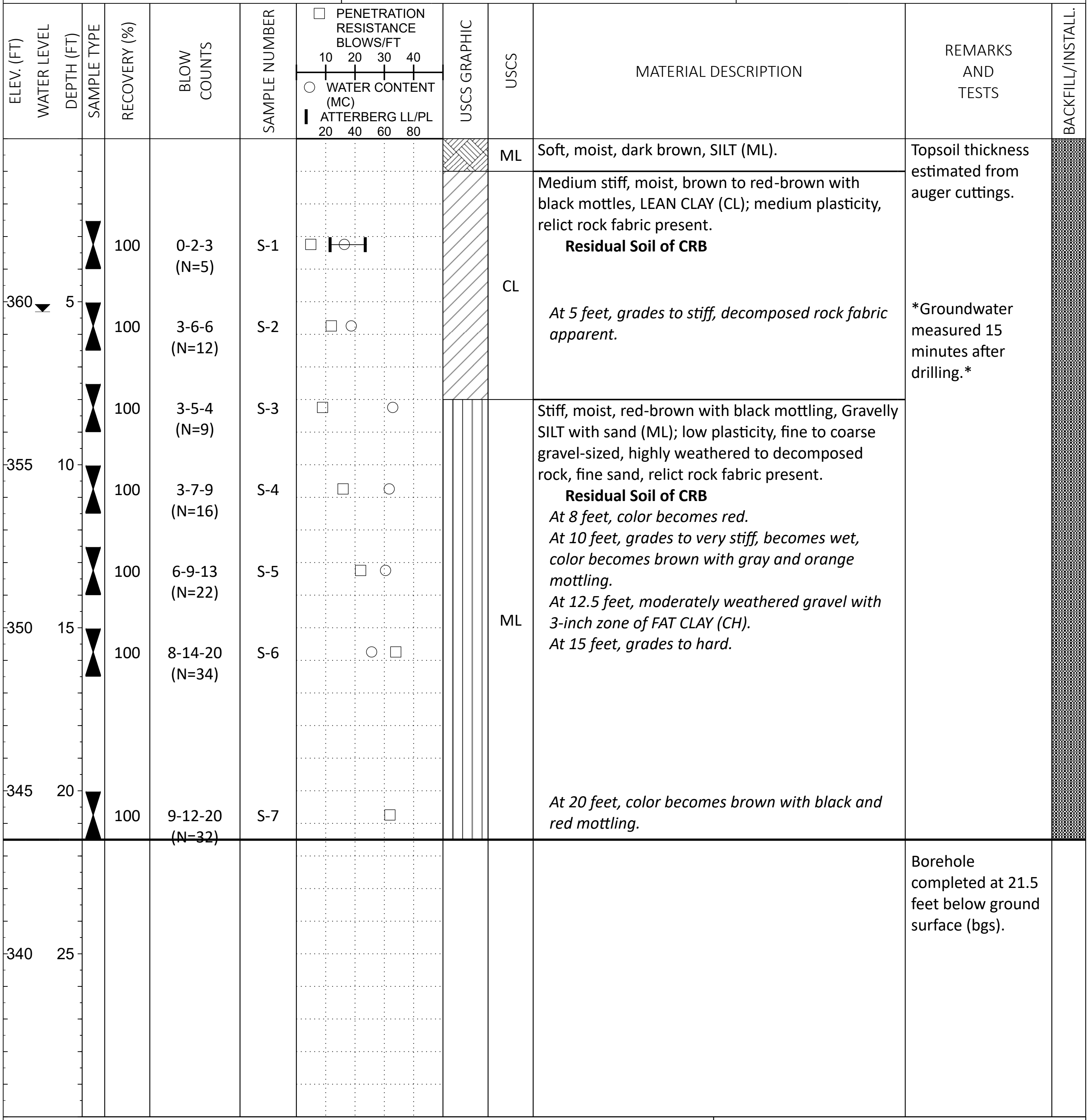
Boring B-2

Sheet 2

Project: City of Salem ASR Facility Improvements
Project Location: Woodmansee Park
Project Number: 6129.0

Log of Boring B-3

Date(s) Drilled	**04/07/2020**	Geotechnical Consultant	**McMillen Jacobs Associates**	Logged By	**A. Judy**	Checked By	**J. Quinn**
Drilling Method/ Rig Type	**Mud Rotary/CME 55 Track Mounted**	Drilling Contractor	**Western States Soil Conservation, Inc.**	Total Depth of Borehole	**21.5 ft**		
Hole Diameter	**3.88 in**	Hammer Weight/Drop (lb/in.)/Type	**140 lb / 30 in / Automatic**	Ground Surface Elevation/Datum	**365.0 ft**		
Location	**North edge of park.**	Coordinates	**7544549.01E,456338.17N**	Elevation Source	**Google Earth**		



RECOVERY (%)	BLOW COUNTS	SAMPLE NUMBER
100	0-2-3 (N=5)	S-1
100	3-6-6 (N=12)	S-2
100	3-5-4 (N=9)	S-3
100	3-7-9 (N=16)	S-4
100	6-9-13 (N=22)	S-5
100	8-14-20 (N=34)	S-6
100	9-12-20 (N=32)	S-7

USCS	MATERIAL DESCRIPTION	REMARKS AND TESTS
ML	Soft, moist, dark brown, SILT (ML).	Topsoil thickness estimated from auger cuttings.
CL	Medium stiff, moist, brown to red-brown with black mottles, LEAN CLAY (CL); medium plasticity, relict rock fabric present. **Residual Soil of CRB** *At 5 feet, grades to stiff, decomposed rock fabric apparent.*	*Groundwater measured 15 minutes after drilling.*
ML	Stiff, moist, red-brown with black mottling, Gravelly SILT with sand (ML); low plasticity, fine to coarse gravel-sized, highly weathered to decomposed rock, fine sand, relict rock fabric present. **Residual Soil of CRB** *At 8 feet, color becomes red.* *At 10 feet, grades to very stiff, becomes wet, color becomes brown with gray and orange mottling.* *At 12.5 feet, moderately weathered gravel with 3-inch zone of FAT CLAY (CH).* *At 15 feet, grades to hard.* *At 20 feet, color becomes brown with black and red mottling.*	
		Borehole completed at 21.5 feet below ground surface (bgs).

McMILLEN JACOBS ASSOCIATES

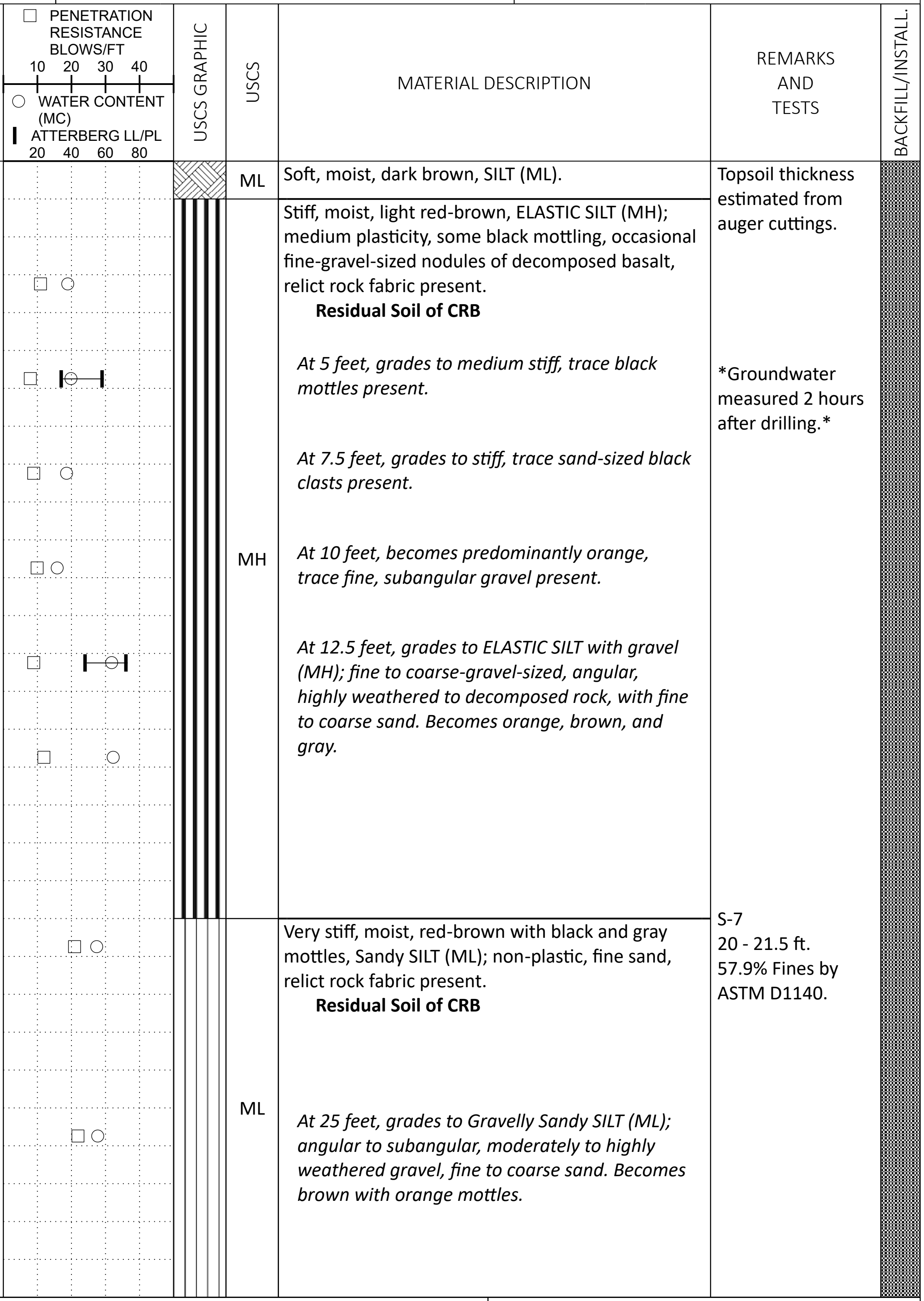
Boring B-3

Sheet 1

Project: City of Salem ASR Facility Improvements
Project Location: Woodmansee Park
Project Number: 6129.0

Log of Boring B-4

Date(s) Drilled	04/07/2020	Geotechnical Consultant	McMillen Jacobs Associates	Logged By	A. Judy
Checked By	J. Quinn				
Drilling Method/ Rig Type	Mud Rotary/CME 55 Track Mounted	Drilling Contractor	Western States Soil Conservation, Inc.	Total Depth of Borehole	41.5 ft
Hole Diameter	3.88 in	Hammer Weight/Drop (lb/in.)/Type	140 lb / 30 in / Automatic	Ground Surface Elevation/Datum	415.0 ft
Location	Northwest of gate.	Coordinates	7544530.12E,456124.66N	Elevation Source	Google Earth



ELEV. (FT)	WATER LEVEL	DEPTH (FT)	SAMPLE TYPE	RECOVERY (%)	BLOW COUNTS	SAMPLE NUMBER	USCS	MATERIAL DESCRIPTION	REMARKS AND TESTS
							ML	Soft, moist, dark brown, SILT (ML).	Topsoil thickness estimated from auger cuttings.
								Stiff, moist, light red-brown, ELASTIC SILT (MH); medium plasticity, some black mottling, occasional fine-gravel-sized nodules of decomposed basalt, relict rock fabric present. **Residual Soil of CRB**	
			⧗	100	4-4-7 (N=11)	S-1			
410	▼	5	⧗	100	3-4-4 (N=8)	S-2		*At 5 feet, grades to medium stiff, trace black mottles present.*	*Groundwater measured 2 hours after drilling.*
			⧗	100	3-4-5 (N=9)	S-3		*At 7.5 feet, grades to stiff, trace sand-sized black clasts present.*	
405		10	⧗	100	3-4-6 (N=10)	S-4	MH	*At 10 feet, becomes predominantly orange, trace fine, subangular gravel present.*	
			⧗	100	6-5-4 (N=9)	S-5		*At 12.5 feet, grades to ELASTIC SILT with gravel (MH); fine to coarse-gravel-sized, angular, highly weathered to decomposed rock, with fine to coarse sand. Becomes orange, brown, and gray.*	
400		15	⧗	100	3-5-7 (N=12)	S-6			
395		20	⧗	100	6-7-14 (N=21)	S-7		Very stiff, moist, red-brown with black and gray mottles, Sandy SILT (ML); non-plastic, fine sand, relict rock fabric present. **Residual Soil of CRB**	S-7 20 - 21.5 ft. 57.9% Fines by ASTM D1140.
390		25	⧗	100	3-10-12 (N=22)	S-8	ML	*At 25 feet, grades to Gravelly Sandy SILT (ML); angular to subangular, moderately to highly weathered gravel, fine to coarse sand. Becomes brown with orange mottles.*	

McMILLEN JACOBS ASSOCIATES

Project: City of Salem ASR Facility Improvements
Project Location: Woodmansee Park
Project Number: 6129.0

Log of Boring B-4

Date(s) Drilled	**04/07/2020**	Geotechnical Consultant	**McMillen Jacobs Associates**	Logged By	**A. Judy**	Checked By	**J. Quinn**
Drilling Method/ Rig Type	**Mud Rotary/CME 55 Track Mounted**	Drilling Contractor	**Western States Soil Conservation, Inc.**	Total Depth of Borehole	**41.5 ft**		
Hole Diameter	**3.88 in**	Hammer Weight/Drop (lb/in.)/Type	**140 lb / 30 in / Automatic**	Ground Surface Elevation/Datum	**415.0 ft**		
Location	**Northwest of gate.**	Coordinates	**7544530.12E,456124.66N**	Elevation Source	**Google Earth**		

ELEV. (FT)	WATER LEVEL	DEPTH (FT)	SAMPLE TYPE	RECOVERY (%)	BLOW COUNTS	SAMPLE NUMBER	☐ PENETRATION RESISTANCE BLOWS/FT 10 20 30 40 ○ WATER CONTENT (MC) ▎ATTERBERG LL/PL 20 40 60 80	USCS GRAPHIC	USCS	MATERIAL DESCRIPTION	REMARKS AND TESTS	BACKFILL/INSTALL.
			⧗	100	7-8-13 (N=21)	S-9	☐ ○			Very stiff, moist, red-brown with black and gray mottles, Sandy SILT (ML); non-plastic, fine sand, relict rock fabric present. **Residual Soil of CRB** *At 30 feet, grades to Sandy SILT (ML); fine sand. Becomes red-brown with black and gray mottles.*		▒
380		35	⧗	100	7-11-16 (N=27)	S-10	☐		ML	*Below 35 feet, grades to Gravelly Sandy SILT (ML); fine to coarse, angular, highly weathered to decomposed gravel.*		▒
375		40	⧗	100	13-20-19 (N=39)	S-11	☐			*At 40 feet, grades to hard, becomes orange with black mottling.*		▒
											Borehole completed at 41.5 feet below ground surface (bgs).	
370		45										
365		50										
360		55										

McMILLEN JACOBS ASSOCIATES

Appendix B

Laboratory Test Results

Breccia Geotechnical Testing, LLC.

Natural Moisture Content (ASTM D2216)

Client: McMillen Jacobs Associates
By: JF
Project Name: Improvements to the ASR Facility
Date: 4/29/2020
Project Number: 6129.0

Exploration ID	B-1	B-1	B-1	B-1	B-1	B-1
Samples ID	S-1	S-2	S-3	S-4	S-5	S-6
Samples Depth (ft.)	2.5-4	5-6.5	7.5-9	10-11.5	12.5-14	15-16.5
Moisture Content (%)	33.1	36.0	61.1	63.0	75.4	66.4

Exploration ID	B-1	B-1	B-1	B-2	B-2	B-2
Samples ID	S-7	S-9	S-10	S-1	S-2	S-3
Samples Depth (ft.)	20-21.5	30-31.5	35-36.5	2.5-4	5-6.5	7.5-9
Moisture Content (%)	59.7	51.8	54.4	52.5	66.3	46.2

Exploration ID	B-2	B-2	B-2	B-2	B-2	B-2
Samples ID	S-4	S-5	S-6b	S-7	S-8	S-9
Samples Depth (ft.)	10-11.5	12.5-14	15.5-16	20-21.5	25-26.5	30-31.5
Moisture Content (%)	59.5	54.7	82.8	71.5	71.4	72.4

Exploration ID	B-2	B-3	B-3	B-3	B-3	B-3
Samples ID	S-11	S-1	S-2	S-3b	S-4	S-5
Samples Depth (ft.)	40-41.5	2.5-4	5-6.5	8-9	10-11.5	12.5-14
Moisture Content (%)	49.7	32.9	37.6	65.9	63.5	61.0

Exploration ID	B-3	B-4	B-4	B-4	B-4	B-4
Samples ID	S-6	S-1	S-2	S-3	S-4	S-5
Samples Depth (ft.)	15-16.5	2.5-4	5-6.5	7.5-9	10-11.5	12.5-14
Moisture Content (%)	51.5	38.0	39.9	37.3	31.8	63.8

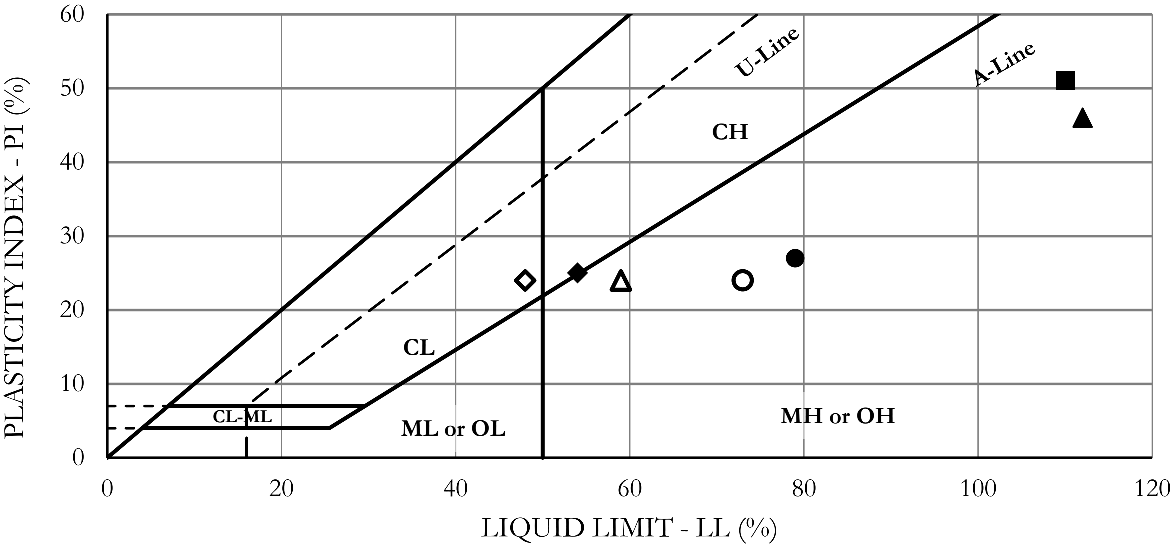
Exploration ID	B-4	B-4	B-4			
Samples ID	S-6	S-8	S-9			
Samples Depth (ft.)	15-16.5	25-26.5	30-31.5			
Moisture Content (%)	64.7	55.6	53.9			

Breccia Geotechnical Testing, LLC.	**Percent Fines (ASTM D1140)**

Client: McMillen Jacobs Associates
By: JF
Project Name: Improvements to the ASR Facility
Date: 4/29/2020
Project Number: 6129.0

Exploration ID	B-1	B-2	B-4			
Samples ID	S-8	S-10	S-7			
Samples Depth (ft.)	25-26.5	35-36.5	20-21.5			
Moisture Content (%)	76.0	57.3	55.0			
Percent Fines (%)	74.0	50.4	57.9			

ATTERBERG LIMITS TEST RESULTS (ASTM D4318)



	Boring	Sample ID	Depth (feet)	Moisture Content (%)	Atterberg Limits			%Pass #200	USCS
					LL	PL	PI		
◆	B-1	S-1	2.5-4	33.1	54	29	25	--	CH
▲	B-1	S-3	7.5-9	61.1	112	66	46	--	MH
●	B-1	S-6	15-16.5	66.4	79	52	27	--	MH
■	B-2	S-6b	15.5-16	82.8	110	59	51	--	MH
◇	B-3	S-1	2.5-4	32.9	48	24	24	--	CL
△	B-4	S-2	5-6.5	39.9	59	35	24	--	MH
○	B-4	S-5	12.5-14	63.8	73	49	24	--	MH

Remarks

Project: Improvements to the ASR Facility
Project No.: 6129.0
Location: Salem, OR

Breccia Geotechnical Testing, LLC.
Brecciageolab@gmail.com
Tel: 971-246-1324

BENCHMARK GEOLABS

Shrink-Swell / Expansion Pressure
ASTM D 3877m

Job No.: 024-039
LL
Date: 5/19/2020
Client: McMillen Jacobs
PL
By: PJ
Project: City of Salem ASR Improvements
PI
Checked By: PJ
Assumed Determined
Boring: B-1 **Sample:** S-1 & S-2 **Depth,ft:** 2.5-6.5 **Specific Gravity:** 2.85
Soil Desc. Reddish Brown CLAY

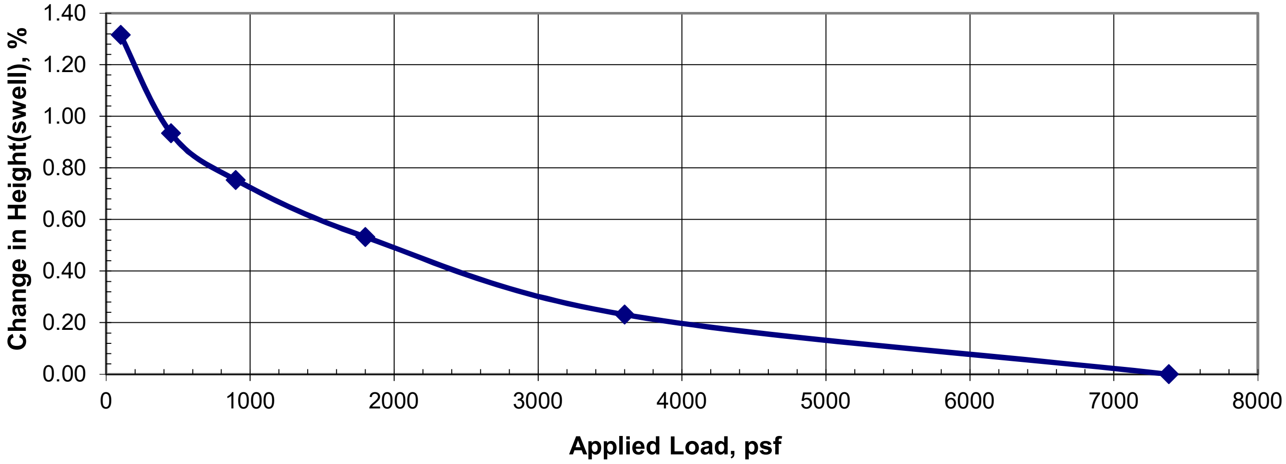
Load, psf:	7381	3600	1800	900	450	100
Exp., %	0.00	0.23	0.53	0.75	0.93	1.32

	Field	Inundated	Air-Dry	Oven-Dry
Moisture %:	6.5	32.7	7.3	0.0
Dry Density, pcf	92.7	91.5	95.0	96.4
Saturation, %	20.1	98.6	23.8	0.0
Void Ratio	0.920	0.946	0.875	0.848
Volume Change, %	0.0	1.3	-2.4	-3.8

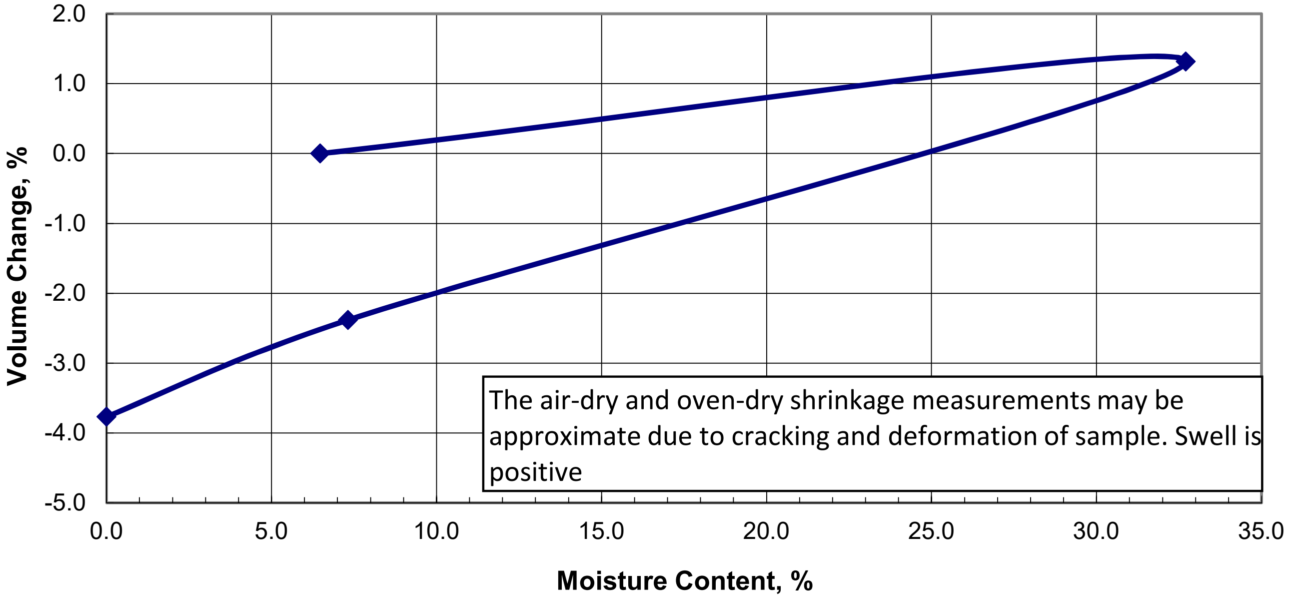
Remarks:

This sample was remolded in an air-dried moisture content to approximate a field density.

Load - Expansion Curve (loaded then expanded)



Volume Change



The air-dry and oven-dry shrinkage measurements may be approximate due to cracking and deformation of sample. Swell is positive

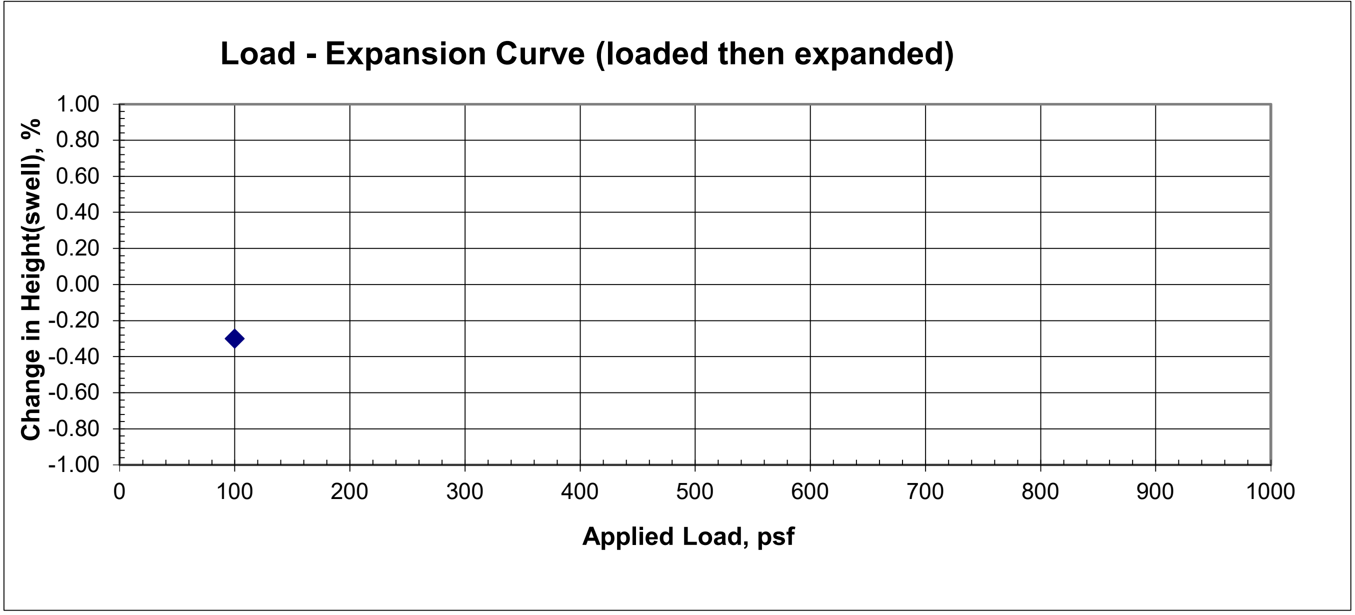
BENCHMARK GEOLABS

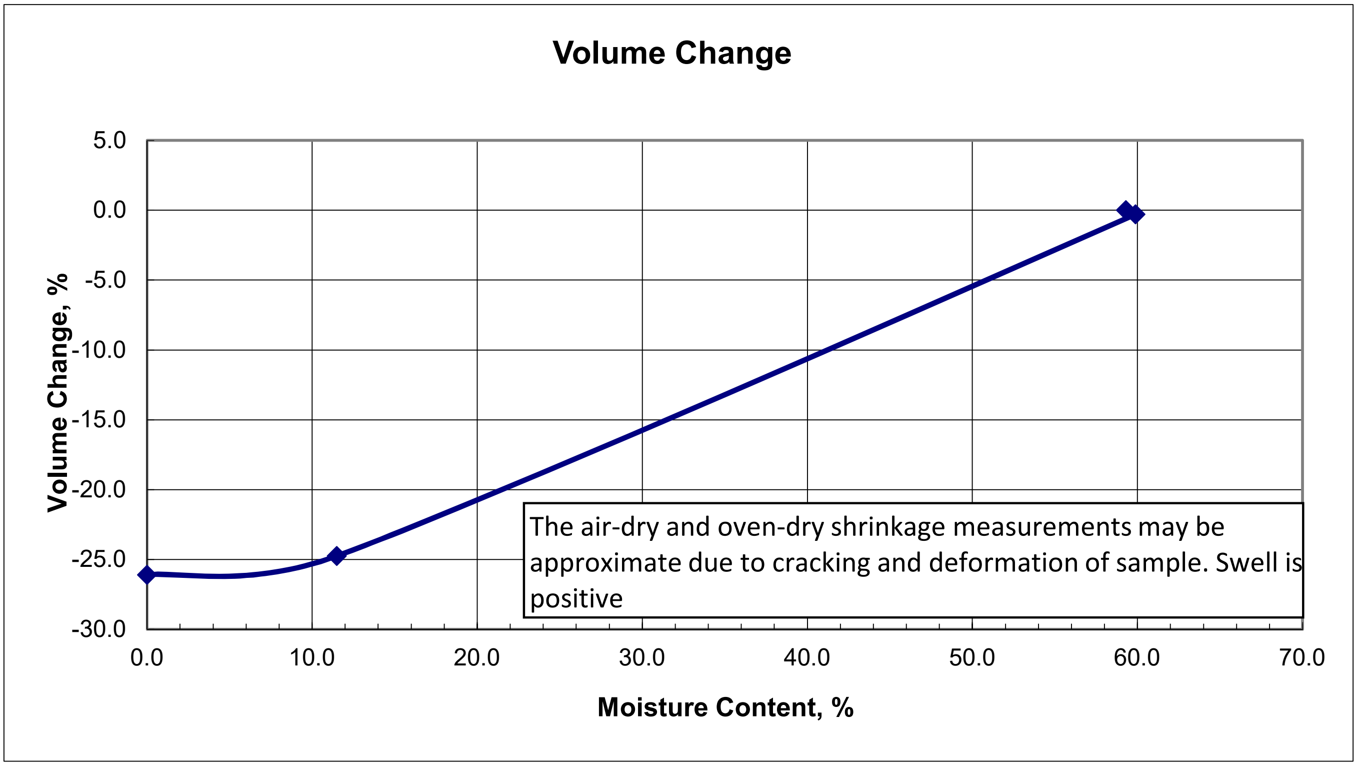
Shrink-Swell / Expansion Pressure
ASTM D 3877m

Job No.: 024-039
LL
Date: 5/18/2020
Client: McMillen Jacobs
PL
By: PJ
Project: City of Salem ASR Improvements
PI
Checked By: PJ
Assumed Determined
Boring: B-1 **Sample:** S-3 & S-4 **Depth,ft:** 7.5-11.5 **Specific Gravity:** 2.8
Soil Desc. Red SILT
Load, psf: 100
Exp., % -0.30

	Field	Inundated	Air-Dry	Oven-Dry
Moisture %:	59.3	59.9	11.5	0.0
Dry Density, pcf	65.0	65.2	86.4	88.0
Saturation, %	98.3	99.7	31.4	0.0
Void Ratio	1.689	1.681	1.024	0.987
Volume Change, %	0.0	-0.3	-24.7	-26.1

Remarks:
This sample was remolded at the as-received moisture content to approximate a field density. This sample did not swell. It consolidated slightly at 100 psf.





The air-dry and oven-dry shrinkage measurements may be approximate due to cracking and deformation of sample. Swell is positive

BENCHMARK GEOLABS

Shrink-Swell / Expansion Pressure
ASTM D 3877m

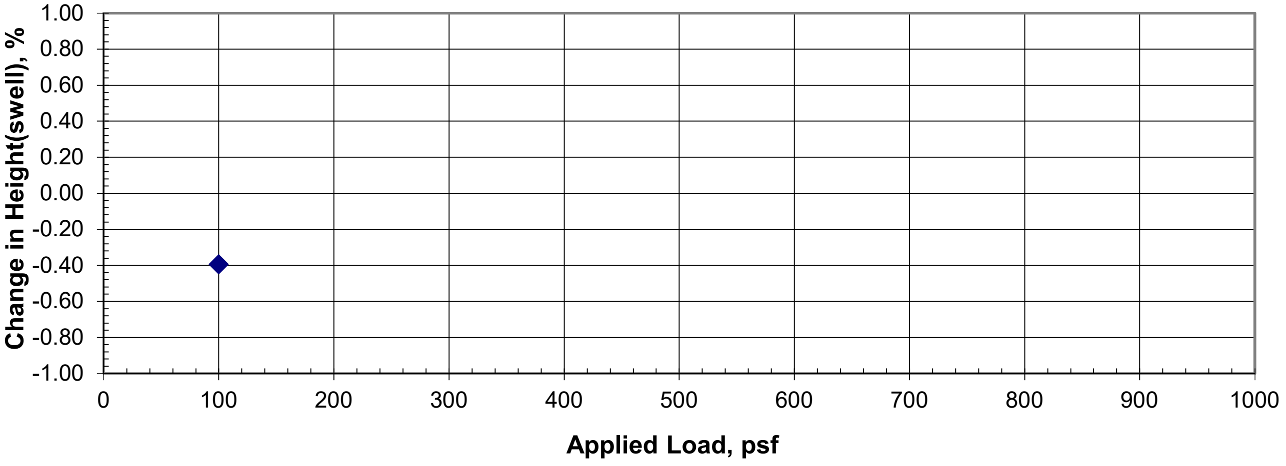
Job No.: 024-039 **LL** **Date:** 5/18/2020
Client: McMillen Jacobs **PL** **By:** PJ
Project: City of Salem ASR Improvements **PI** **Checked By:** PJ Assumed Determined
Boring: B-4 **Sample:** S-1 & S-2 **Depth,ft:** 2.5-6.35 **Specific Gravity:** 2.8
Soil Desc. Strong Brown SILT

Load, psf: 100
Exp., % -0.39

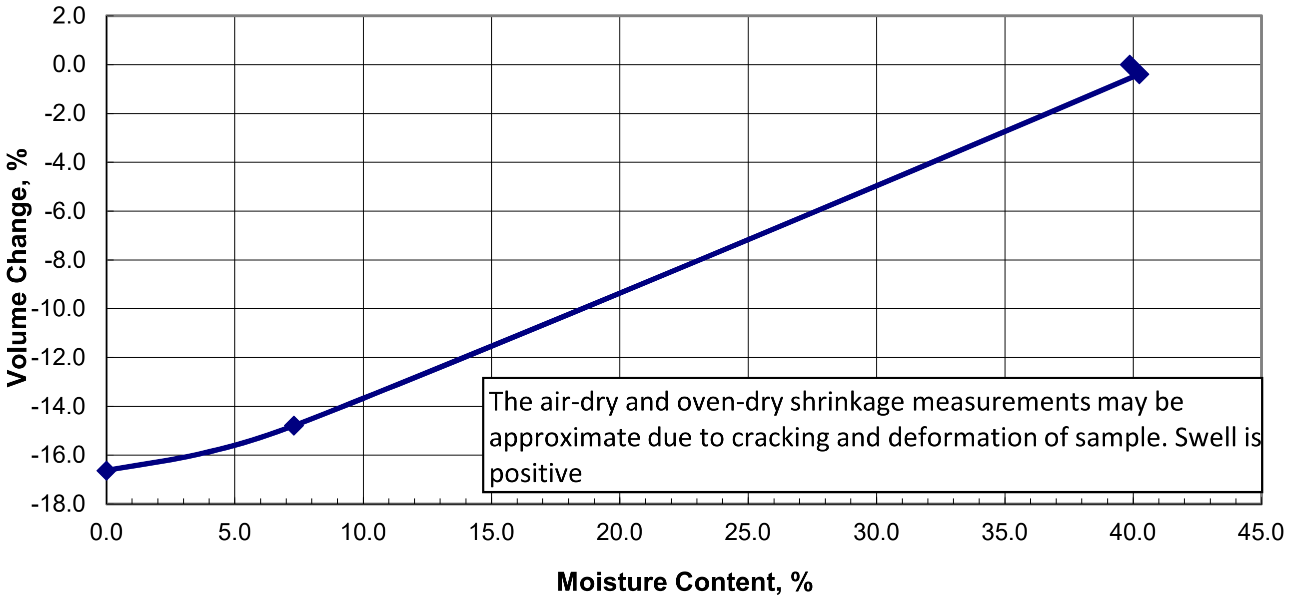
	Field	Inundated	Air-Dry	Oven-Dry
Moisture %:	39.9	40.2	7.3	0.0
Dry Density, pcf	81.6	81.9	95.8	97.9
Saturation, %	97.6	99.2	24.7	0.0
Void Ratio	1.144	1.135	0.827	0.787
Volume Change, %	0.0	-0.4	-14.8	-16.6

Remarks:
This sample was remolded at the as-received moisture content to approximate a field density. This sample did not swell. It consolidated slightly at 100 psf.

Load - Expansion Curve (loaded then expanded)



Volume Change



The air-dry and oven-dry shrinkage measurements may be approximate due to cracking and deformation of sample. Swell is positive

Appendix C

Results of Dynamic Cone Penetrometer Testing

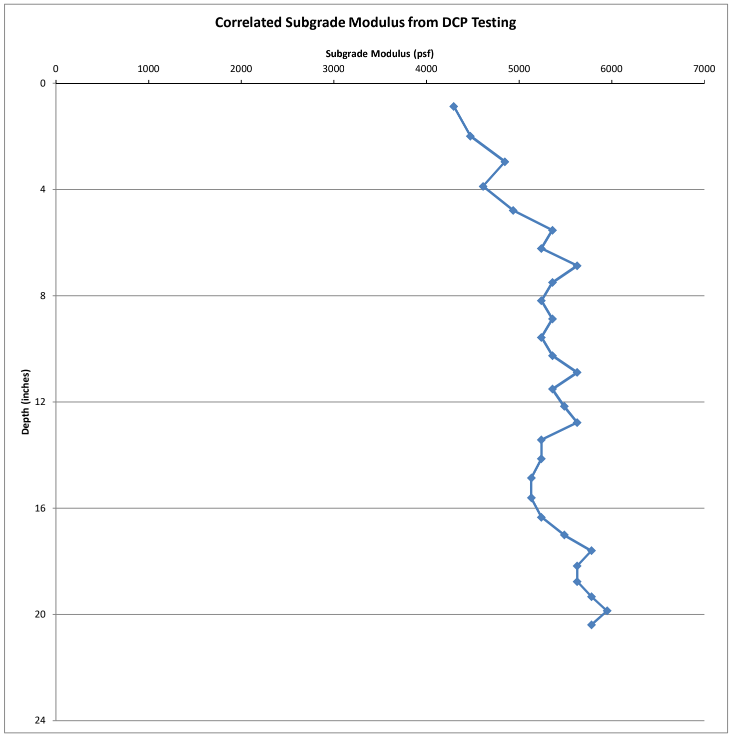
Project:	City of Salem ASR Project
Project Number:	6129
Date:	7/1/2020
Exploration Name:	DCP-1

Type of Pavement:	N	C/AC/N (C = Portland Cement Concrete, AC = Asphaltic Concrete, N = None)
Thickness of Pavement:	0	inches
Thickness of Base Rock:	0	inches
Seating Depth:	0.275590551	(inches from ground surface to bottom of excavation)
Initial DCP reading:	380	mm

Table 2 - C_f for DCP and FWD to Convert M_R to an Equivalent Saturated Laboratory M_R (ODOT Pavement Design Guide)

Layer Type & Location	C_f
Subgrade Below AC & Aggregate Base	0.35
Aggregate Base or Subbase Below AC	0.62
Subgrade Below PCC or CTB	0.25
Aggregate Base or Subbase Below PCC	0.62
None (no pavement)	0.33

Reading No.	No. of Blows	Depth Reading (mm)	Type of Hammer A=17.6 lb hammer B=10.1 lb hammer (only need to note change in hammer)	Hammer Blow Index	Accumulative Penetration (mm)	Middle of interval (mm)	Middle of interval (inches)	Material Type	Material Type Coefficient C_f	DCP Index mm/blow	CBR (correlation from user manual) %	Subgrade Modulus (Pg. 21 ODOT Pavement Design Guide) psf
1	1	410	A	1	30	22	0.9	Subgrade	0.33	30.00	6	4294
2	1	437		1	57	51	2.0	Subgrade	0.33	27.00	7	4474
3	1	459		1	79	75	3.0	Subgrade	0.33	22.00	9	4846
4	1	484		1	104	99	3.9	Subgrade	0.33	25.00	8	4610
5	1	505		1	125	122	4.8	Subgrade	0.33	21.00	10	4935
6	1	522		1	142	141	5.5	Subgrade	0.33	17.00	12	5358
7	1	540		1	160	158	6.2	Subgrade	0.33	18.00	11	5240
8	1	555		1	175	175	6.9	Subgrade	0.33	15.00	14	5626
9	1	572		1	192	191	7.5	Subgrade	0.33	17.00	12	5358
10	1	590		1	210	208	8.2	Subgrade	0.33	18.00	11	5240
11	1	607		1	227	226	8.9	Subgrade	0.33	17.00	12	5358
12	1	625		1	245	243	9.6	Subgrade	0.33	18.00	11	5240
13	1	642		1	262	261	10.3	Subgrade	0.33	17.00	12	5358
14	1	657		1	277	277	10.9	Subgrade	0.33	15.00	14	5626
15	1	674		1	294	293	11.5	Subgrade	0.33	17.00	12	5358
16	1	690		1	310	309	12.2	Subgrade	0.33	16.00	13	5487
17	1	705		1	325	325	12.8	Subgrade	0.33	15.00	14	5626
18	1	723		1	343	341	13.4	Subgrade	0.33	18.00	11	5240
19	1	741		1	361	359	14.1	Subgrade	0.33	18.00	11	5240
20	1	760		1	380	378	14.9	Subgrade	0.33	19.00	11	5131
21	1	779		1	399	397	15.6	Subgrade	0.33	19.00	11	5131
22	1	797		1	417	415	16.3	Subgrade	0.33	18.00	11	5240
23	1	813		1	433	432	17.0	Subgrade	0.33	16.00	13	5487
24	1	827		1	447	447	17.6	Subgrade	0.33	14.00	15	5780
25	1	842		1	462	462	18.2	Subgrade	0.33	15.00	14	5626
26	1	857		1	477	477	18.8	Subgrade	0.33	15.00	14	5626
27	1	871		1	491	491	19.3	Subgrade	0.33	14.00	15	5780
28	1	884		1	504	505	19.9	Subgrade	0.33	13.00	17	5949
29	1	898		1	518	518	20.4	Subgrade	0.33	14.00	15	5780


Correlated Subgrade Modulus from DCP Testing

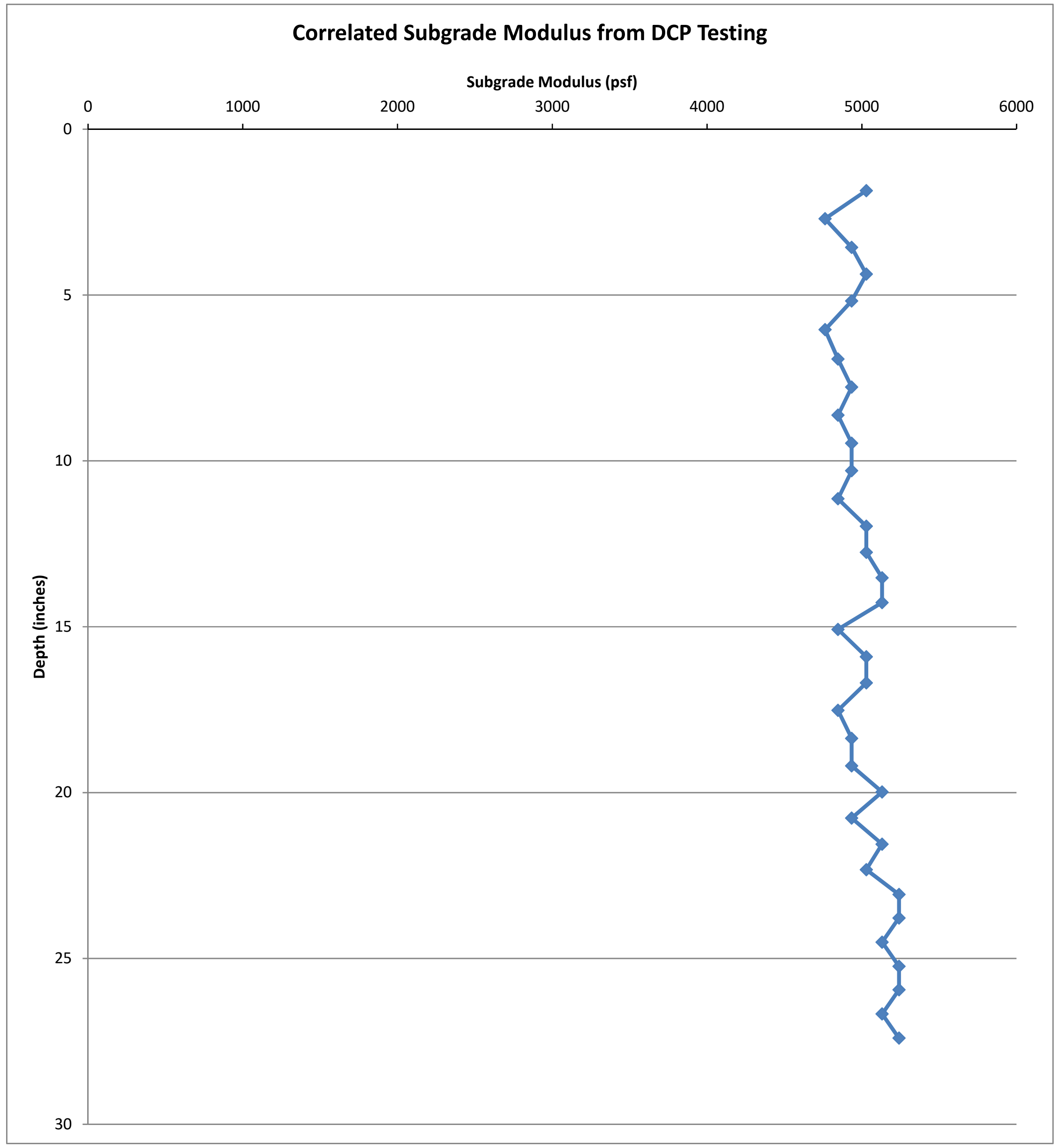
Project:	City of Salem ASR Project
Project Number:	6129
Date:	7/1/2020
Exploration Name:	DCP-2

Type of Pavement:	N	C/AC/N (C = Portland Cement Concrete, AC = Asphaltic Concrete, N = None)
Thickness of Pavement:	0	inches
Thickness of Base Rock:	0	inches
Seating Depth:	1.456692913	(inches from ground surface to bottom of excavation)
Initial DCP reading:	360	mm

Table 2 - C_f for DCP and FWD to Convert M_R to an Equivalent Saturated Laboratory M_R (ODOT Pavement Design Guide)

Layer Type & Location	C_f
Subgrade Below AC & Aggregate Base	0.35
Aggregate Base or Subbase Below AC	0.62
Subgrade Below PCC or CTB	0.25
Aggregate Base or Subbase Below PCC	0.62
None (no pavement)	0.33

Reading No.	No. of Blows	Depth Reading (mm)	Type of Hammer A=17.6 lb hammer B=10.1 lb hammer (only need to note change in hammer)	Hammer Blow Index	Accumulative Penetration (mm)	Middle of interval (mm)	Middle of interval (inches)	Material Type	Material Type Coefficient C_f	DCP Index mm/blow	CBR (correlation from user manual) %	Subgrade Modulus (Pg. 21 ODOT Pavement Design Guide) psf
1	1	380	A	1	20	47	1.9	Subgrade	0.33	20.00	10	5029
2	1	403		1	43	69	2.7	Subgrade	0.33	23.00	9	4763
3	1	424		1	64	91	3.6	Subgrade	0.33	21.00	10	4935
4	1	444		1	84	111	4.4	Subgrade	0.33	20.00	10	5029
5	1	465		1	105	132	5.2	Subgrade	0.33	21.00	10	4935
6	1	488		1	128	154	6.0	Subgrade	0.33	23.00	9	4763
7	1	510		1	150	176	6.9	Subgrade	0.33	22.00	9	4846
8	1	531		1	171	198	7.8	Subgrade	0.33	21.00	10	4935
9	1	553		1	193	219	8.6	Subgrade	0.33	22.00	9	4846
10	1	574		1	214	241	9.5	Subgrade	0.33	21.00	10	4935
11	1	595		1	235	262	10.3	Subgrade	0.33	21.00	10	4935
12	1	617		1	257	283	11.1	Subgrade	0.33	22.00	9	4846
13	1	637		1	277	304	12.0	Subgrade	0.33	20.00	10	5029
14	1	657		1	297	324	12.8	Subgrade	0.33	20.00	10	5029
15	1	676		1	316	344	13.5	Subgrade	0.33	19.00	11	5131
16	1	695		1	335	363	14.3	Subgrade	0.33	19.00	11	5131
17	1	717		1	357	383	15.1	Subgrade	0.33	22.00	9	4846
18	1	737		1	377	404	15.9	Subgrade	0.33	20.00	10	5029
19	1	757		1	397	424	16.7	Subgrade	0.33	20.00	10	5029
20	1	779		1	419	445	17.5	Subgrade	0.33	22.00	9	4846
21	1	800		1	440	467	18.4	Subgrade	0.33	21.00	10	4935
22	1	821		1	461	488	19.2	Subgrade	0.33	21.00	10	4935
23	1	840		1	480	508	20.0	Subgrade	0.33	19.00	11	5131
24	1	861		1	501	528	20.8	Subgrade	0.33	21.00	10	4935
25	1	880		1	520	548	21.6	Subgrade	0.33	19.00	11	5131
26	1	900		1	540	567	22.3	Subgrade	0.33	20.00	10	5029
27	1	918		1	558	586	23.1	Subgrade	0.33	18.00	11	5240
28	1	936		1	576	604	23.8	Subgrade	0.33	18.00	11	5240
29	1	955		1	595	623	24.5	Subgrade	0.33	19.00	11	5131
30	1	973		1	613	641	25.2	Subgrade	0.33	18.00	11	5240
31	1	991		1	631	659	25.9	Subgrade	0.33	18.00	11	5240
32	1	1010		1	650	678	26.7	Subgrade	0.33	19.00	11	5131
33	1	1028		1	668	696	27.4	Subgrade	0.33	18.00	11	5240



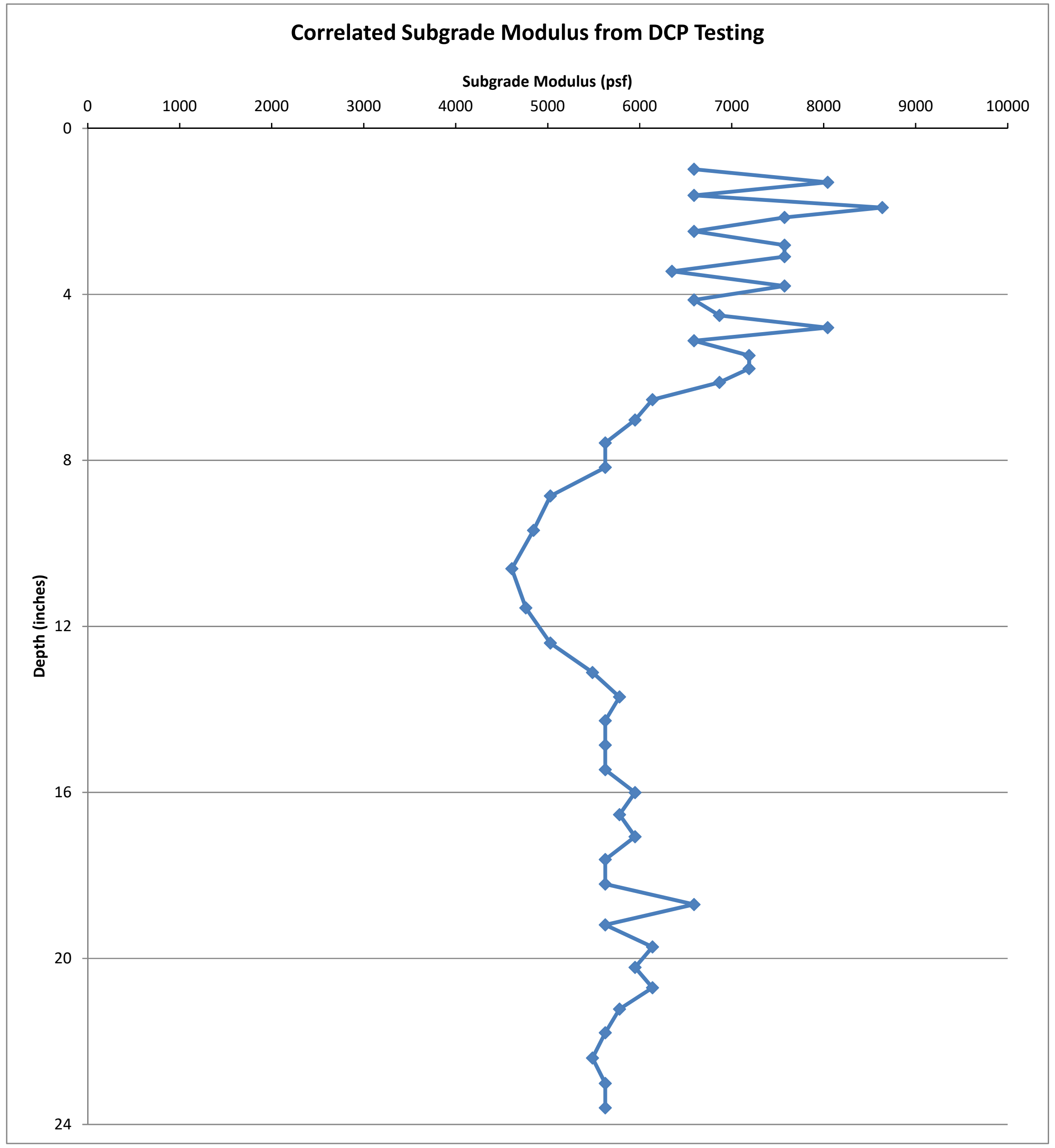
Project:	City of Salem ASR Project
Project Number:	6129
Date:	7/1/2020
Exploration Name:	DCP-3

Type of Pavement:	N	C/AC/N (C = Portland Cement Concrete, AC = Asphaltic Concrete, N = None)
Thickness of Pavement:	0	inches
Thickness of Base Rock:	0	inches
Seating Depth:	0.787401575	(inches from ground surface to bottom of excavation)
Initial DCP reading:	355	mm

Table 2 - C_f for DCP and FWD to Convert M_R to an Equivalent Saturated Laboratory M_R (ODOT Pavement Design Guide)

Layer Type & Location	C_f
Subgrade Below AC & Aggregate Base	0.35
Aggregate Base or Subbase Below AC	0.62
Subgrade Below PCC or CTB	0.25
Aggregate Base or Subbase Below PCC	0.62
None (no pavement)	0.33

Reading No.	No. of Blows	Depth Reading (mm)	Type of Hammer A=17.6 lb hammer B=10.1 lb hammer (only need to note change in hammer)	Hammer Blow Index	Accumulative Penetration (mm)	Middle of interval (mm)	Middle of interval (inches)	Material Type	Material Type Coefficient C_f	DCP Index mm/blow	CBR (correlation from user manual) %	Subgrade Modulus (Pg. 21 ODOT Pavement Design Guide) psf
1	1	365	A	1	10	25	1.0	Subgrade	0.33	10.00	22	6590
2	1	371		1	16	33	1.3	Subgrade	0.33	6.00	39	8043
3	1	381		1	26	41	1.6	Subgrade	0.33	10.00	22	6590
4	1	386		1	31	49	1.9	Subgrade	0.33	5.00	48	8636
5	1	393		1	38	55	2.1	Subgrade	0.33	7.00	33	7574
6	1	403		1	48	63	2.5	Subgrade	0.33	10.00	22	6590
7	1	410		1	55	72	2.8	Subgrade	0.33	7.00	33	7574
8	1	417		1	62	79	3.1	Subgrade	0.33	7.00	33	7574
9	1	428		1	73	88	3.4	Subgrade	0.33	11.00	20	6350
10	1	435		1	80	97	3.8	Subgrade	0.33	7.00	33	7574
11	1	445		1	90	105	4.1	Subgrade	0.33	10.00	22	6590
12	1	454		1	99	115	4.5	Subgrade	0.33	9.00	25	6867
13	1	460		1	105	122	4.8	Subgrade	0.33	6.00	39	8043
14	1	470		1	115	130	5.1	Subgrade	0.33	10.00	22	6590
15	1	478		1	123	139	5.5	Subgrade	0.33	8.00	28	7190
16	1	486		1	131	147	5.8	Subgrade	0.33	8.00	28	7190
17	1	495		1	140	156	6.1	Subgrade	0.33	9.00	25	6867
18	1	507		1	152	166	6.5	Subgrade	0.33	12.00	18	6138
19	1	520		1	165	179	7.0	Subgrade	0.33	13.00	17	5949
20	1	535		1	180	193	7.6	Subgrade	0.33	15.00	14	5626
21	1	550		1	195	208	8.2	Subgrade	0.33	15.00	14	5626
22	1	570		1	215	225	8.9	Subgrade	0.33	20.00	10	5029
23	1	592		1	237	246	9.7	Subgrade	0.33	22.00	9	4846
24	1	617		1	262	270	10.6	Subgrade	0.33	25.00	8	4610
25	1	640		1	285	294	11.6	Subgrade	0.33	23.00	9	4763
26	1	660		1	305	315	12.4	Subgrade	0.33	20.00	10	5029
27	1	676		1	321	333	13.1	Subgrade	0.33	16.00	13	5487
28	1	690		1	335	348	13.7	Subgrade	0.33	14.00	15	5780
29	1	705		1	350	363	14.3	Subgrade	0.33	15.00	14	5626
30	1	720		1	365	378	14.9	Subgrade	0.33	15.00	14	5626
31	1	735		1	380	393	15.5	Subgrade	0.33	15.00	14	5626
32	1	748		1	393	407	16.0	Subgrade	0.33	13.00	17	5949
33	1	762		1	407	420	16.5	Subgrade	0.33	14.00	15	5780
34	1	775		1	420	434	17.1	Subgrade	0.33	13.00	17	5949
35	1	790		1	435	448	17.6	Subgrade	0.33	15.00	14	5626
36	1	805		1	450	463	18.2	Subgrade	0.33	15.00	14	5626
37	1	815		1	460	475	18.7	Subgrade	0.33	10.00	22	6590
38	1	830		1	475	488	19.2	Subgrade	0.33	15.00	14	5626
39	1	842		1	487	501	19.7	Subgrade	0.33	12.00	18	6138
40	1	855		1	500	514	20.2	Subgrade	0.33	13.00	17	5949
41	1	867		1	512	526	20.7	Subgrade	0.33	12.00	18	6138
42	1	881		1	526	539	21.2	Subgrade	0.33	14.00	15	5780
43	1	896		1	541	554	21.8	Subgrade	0.33	15.00	14	5626
44	1	912		1	557	569	22.4	Subgrade	0.33	16.00	13	5487
45	1	927		1	572	585	23.0	Subgrade	0.33	15.00	14	5626
46	1	942		1	587	600	23.6	Subgrade	0.33	15.00	14	5626


Correlated Subgrade Modulus from DCP Testing

Project:	City of Salem ASR Project
Project Number:	6129
Date:	7/1/2020
Exploration Name:	DCP-4

Type of Pavement:	N	C/AC/N (C = Portland Cement Concrete, AC = Asphaltic Concrete, N = None)
Thickness of Pavement:	0	inches
Thickness of Base Rock:	0	inches
Seating Depth:	1.102362205	(inches from ground surface to bottom of excavation)
Initial DCP reading:	351	mm

Table 2 - C_f for DCP and FWD to Convert M_R to an Equivalent Saturated Laboratory M_R (ODOT Pavement Design Guide)

Layer Type & Location	C_f
Subgrade Below AC & Aggregate Base	0.35
Aggregate Base or Subbase Below AC	0.62
Subgrade Below PCC or CTB	0.25
Aggregate Base or Subbase Below PCC	0.62
None (no pavement)	0.33

Reading No.	No. of Blows	Depth Reading (mm)	Type of Hammer A=17.6 lb hammer B=10.1 lb hammer (only need to note change in hammer)	Hammer Blow Index	Accumulative Penetration (mm)	Middle of interval (mm)	Middle of interval (inches)	Material Type	Material Type Coefficient C_f	DCP Index mm/blow	CBR (correlation from user manual) %	Subgrade Modulus (Pg. 21 ODOT Pavement Design Guide) psf
1	1	372	A	1	21	39	1.5	Subgrade	0.33	21.00	10	4935
2	1	393		1	42	60	2.3	Subgrade	0.33	21.00	10	4935
3	1	415		1	64	81	3.2	Subgrade	0.33	22.00	9	4846
4	1	436		1	85	103	4.0	Subgrade	0.33	21.00	10	4935
5	1	461		1	110	126	4.9	Subgrade	0.33	25.00	8	4610
6	1	485		1	134	150	5.9	Subgrade	0.33	24.00	8	4684
7	1	509		1	158	174	6.9	Subgrade	0.33	24.00	8	4684
8	1	530		1	179	197	7.7	Subgrade	0.33	21.00	10	4935
9	1	550		1	199	217	8.5	Subgrade	0.33	20.00	10	5029
10	1	568		1	217	236	9.3	Subgrade	0.33	18.00	11	5240
11	1	586		1	235	254	10.0	Subgrade	0.33	18.00	11	5240
12	1	604		1	253	272	10.7	Subgrade	0.33	18.00	11	5240
13	1	622		1	271	290	11.4	Subgrade	0.33	18.00	11	5240
14	1	641		1	290	309	12.1	Subgrade	0.33	19.00	11	5131
15	1	660		1	309	328	12.9	Subgrade	0.33	19.00	11	5131
16	1	678		1	327	346	13.6	Subgrade	0.33	18.00	11	5240
17	1	697		1	346	365	14.4	Subgrade	0.33	19.00	11	5131
18	1	716		1	365	384	15.1	Subgrade	0.33	19.00	11	5131
19	1	731		1	380	401	15.8	Subgrade	0.33	15.00	14	5626
20	1	755		1	404	420	16.5	Subgrade	0.33	24.00	8	4684
21	1	775		1	424	442	17.4	Subgrade	0.33	20.00	10	5029
22	1	795		1	444	462	18.2	Subgrade	0.33	20.00	10	5029
23	1	815		1	464	482	19.0	Subgrade	0.33	20.00	10	5029
24	1	832		1	481	501	19.7	Subgrade	0.33	17.00	12	5358
25	1	847		1	496	517	20.3	Subgrade	0.33	15.00	14	5626
26	1	864		1	513	533	21.0	Subgrade	0.33	17.00	12	5358
27	1	882		1	531	550	21.7	Subgrade	0.33	18.00	11	5240
28	1	902		1	551	569	22.4	Subgrade	0.33	20.00	10	5029
29	1	920		1	569	588	23.1	Subgrade	0.33	18.00	11	5240
30	1	940		1	589	607	23.9	Subgrade	0.33	20.00	10	5029