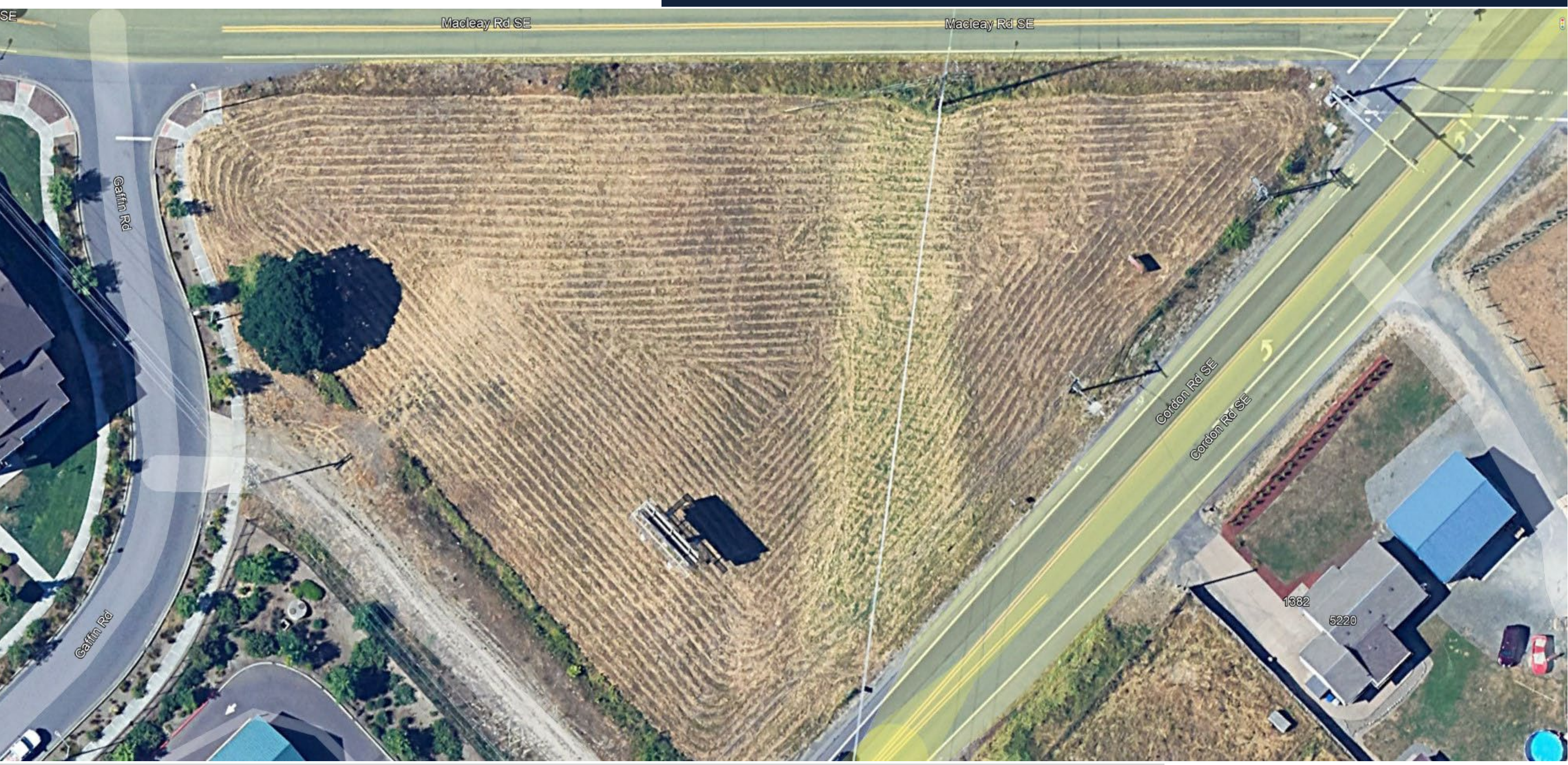
Report for Geotechnical Engineering Design Services

Macleay and Cordon Development
Macleay Road SE & Cordon Road SE
Salem, Oregon

CGS Project: Westech-19-01



Prepared For:

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Date: March 6, 2025

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March 6, 2025

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Attention: Josh Wells (jwells@westech-eng.com)

Subject: Report for Geotechnical Engineering Design Services
Macleay and Cordon Development
Macleay Road SE & Cordon Road SE
Salem, Oregon
Central Project No.: Westech-19-01

Central Geotechnical Services, LLC (Central) is pleased to submit this report of geotechnical engineering services for the proposed development located at Macleay Road SE and Cordon Road SE in Salem, Oregon. This report was prepared for conformance with our proposal dated June 18, 2024. Please feel free to call our office with questions about this report.

Respectfully,

Central Geotechnical Services, LLC

Julio Vela, Ph.D., P.E., G.E.
Principal

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1.0 INTRODUCTION

Central Geotechnical Services, LLC (Central) is pleased to submit this geotechnical engineering report for the proposed development located at Macleay Road SE and Cordon Road SE in Salem, Oregon. The location of the site is shown in the Vicinity Map, Figure 1.

We understand the proposed development will consist of a new convenience store, gas canopy with gas pumps, car wash, parking areas, and a new traffic signal. Project development will also include off-site public improvements including roadway widening and installation of a traffic signal.

Structural design loads were not provided at the time this report was prepared. However, we have assumed maximum column and wall loads will be on the order of 50 kips per column and 2 kips per lineal foot (klf) respectively, and that floor loads for slabs on grade will be 150 pounds per square foot (psf) or less. We have also assumed that maximum cuts and fills for general site grading (not including cuts for structures below grade) will be less than 4 feet.

2.0 PURPOSE AND SCOPE OF WORK

The purpose of our services was to evaluate soil and groundwater conditions as a basis for developing geotechnical engineering design criteria for roadway and site improvements. Our scope of services was provided in general accordance with our proposal dated June 18, 2024 and authorized on July 10, 2024, which included the following:

1. Reviewed information regarding subsurface soil and groundwater in the vicinity of the site, including reports in our files, selected geologic maps, and other geotechnical engineering-related information.
2. Coordinated and managed the field investigation, including public utility notification and scheduling of subcontractors and Central's field staff. Public locates were called in by our office as required by law.
3. Explored subsurface soil and groundwater conditions at the project site by advancing the following drilled borings:
   - Two borings to depths of 26.5 and 31.5 feet below ground surface (bgs) in the approximate vicinity of the planned convenience store and gas canopy.
   - Two borings advanced using hollow-stem auger drilling methods to depths of 16.5 feet bgs with infiltration testing (INF-1 and INF-2) conducted at depths of 10 feet and 5 feet bgs.
   - Three borings to depths of 6.5 feet bgs with pavement cores within the right-of-way of Macleay Road SE.
   - One boring to a depth of 31 feet bgs in the general vicinity of the proposed new traffic signal.
4. Obtained samples at representative intervals from the explorations, observed groundwater conditions, and maintained detailed logs in general accordance with ASTM International (ASTM) Standard Practices Test Method D2488.
5. Conducted infiltration testing in two areas adjacent to proposed boring locations at depths of approximately 5 and 10 feet bgs.
6. Performed laboratory tests on selected soil samples obtained from the explorations to evaluate pertinent engineering characteristics. Laboratory testing included moisture content determination, Atterberg limits tests, and percent fines tests.

7. Provided this geotechnical engineering evaluation of the site and design recommendations in a geotechnical report that addresses the following geotechnical components:
   - A general description of site topography, geology, and subsurface conditions.
   - An opinion as to the adequacy of the proposed development from a geotechnical engineering standpoint.
   - Recommendations for site preparation measures, including disposition of undocumented fill and unsuitable native soils, recommendations for temporary cut slopes, and constraints for wet-weather construction.
   - Recommendations for temporary excavation and temporary excavation protection, such as excavation sheeting and bracing.
   - Recommendations for earthwork construction, including use of on-site and imported structural fill and fill placement and compaction requirements.
   - Recommendations for site retaining walls for walls up to 8 feet tall, including static and seismic active earth pressures, and drainage and backfill recommendations.
   - Recommendations for foundations to support the proposed structures, including minimum width and embedment, design soil bearing pressures, settlement estimates (total and differential), coefficient of friction, and passive earth pressures for sliding resistance. We have assumed that shallow foundations can be used to adequately support the structures.
   - Recommendations for supporting on-grade slabs, including base rock, capillary break, and modulus of subgrade reaction.
   - Recommendations for pavement design sections including base rock and asphaltic concrete (AC) thickness, product requirements, and construction considerations.
   - Seismic design parameters, including soil site class evaluation in accordance with the 2018 International Building Code (IBC) and the 2019 Oregon State Structural Code.

3.0 SITE CONDITIONS

3.1 GEOLOGY

The site is located along the drainage valley of Mill Creek (Gannett and Caldwell, 1998). The valley separates the Salem Hills to the west and the Waldo Hills to the east, which are structural highs located within the larger Willamette Valley physiographic province (Orr and Orr, 1999). Turner Gap is a valley cut along the ancestral course of the North Santiam River, which emptied into the Salem Basin to the north during the late Pliocene to Pleistocene (approximately 2 million to 100,000 years before present) but is now occupied by Mill Creek.

The Waldo Hills flanking the valley are highlands underlain by basalt flows belonging to the Miocene Age (approximately 17 million to 6 million years old) Columbia River Basalt Group (CRBG), comprising a series of thick basalt flows that filled lowland areas throughout much of the northern Willamette Valley (Tolan and Beeson, 2000). In-place spheroidal weathering of the basalt is common in the shallow subsurface, producing large cobbles and boulders (or "corestones") of less weathered basalt in a strongly decomposed matrix of residual soil. Geologic mapping by Tolan and Beeson (2000) indicates the CRBG and younger materials are underlain by a thick assemblage of Eocene to early Miocene Age (approximately 40 million to 16 million years old) marine sedimentary rocks consisting of interbedded sandstone, siltstone, shale, and claystone. The marine sedimentary bedrock is mapped near the surface in the north-central portion of the site where tectonic uplift and erosion has removed the overlying units (Tolan and Beeson, 2000). The sedimentary bedrock is the oldest unit mapped in the region.

3.2 SURFACE CONDITIONS

The site is currently a vacant lot vegetated with grass and occasional shrubbery and trees. A billboard is present on the south end of the site. The project site is bounded by Macleay Road SE to the north, Cordon Road SE to the east and south, and Gaffin Road to the west.

3.3 SUBSURFACE CONDITIONS

3.3.1 Surficial Materials

3.3.1.1 Macleay Road SE

Our explorations within Macleay Road SE encountered 5.5 to 6 inches of AC over approximately 6.5 to 10 inches of aggregate base. Pavement fabric was observed between pavement layers at approximately 1.5 inches into the AC cores in explorations B-2 and B-3. We did not observe pavement fabric between layers of AC in exploration B-1.

3.3.1.2 On-site

Explorations B-4 through B-6, and INF-1 and INF-2 encountered an approximate 4-inch-thick root zone and approximately 12 inches of topsoil (tilled/disturbed soil).

3.3.2 Undocumented Fill

Undocumented fill was observed in explorations B-1, B-2, and B-3 to depths of 2 to 6 feet bgs. Undocumented fill generally consists of soft to medium-stiff silt with varying amounts of sand, gravel, and organics. An organic odor was noted. Laboratory testing on selected samples of the undocumented fill indicated mosture contents of approximately 35 percent at the time of our explorations.

3.3.3 Silt

Silt was observed underlying the surficial materials and the undocumented fill in our explorations to depths of approximately 25 feet bgs. Laboratory testing on selected samples of the silt indicates moisture contents ranging from approximately 31 to 40 percent at the time of our explorations. Fines content analyses indicates the silt has a fines content ranging from 90 to 99 percent, and Atterberg Limits testing indicates the silt has moderate plasticity.

3.3.4 Gravel

Gravel was encountered in explorations B-4, B-5, and B-6 at depths between 24.5 feet and 25.5 feet bgs underlying the silt. Gravel is generally very dense with varying amounts of sand and silt. Laboratory testing on selected gravel samples indicates a moisture content ranging from 13 to 21 percent at the time of our explorations, with a fines content of approximately 9 percent.

3.3.5 Groundwater Conditions

Groundwater was not observed within our explorations due to the drilling method used. Based on readily available water well logs from Oregon Water Resources Department, groundwater in the area is at an approximate depth of 30 to 33 feet bgs. The depth to groundwater may fluctuate in response to seasonal changes, prolonged rainfall, changes in surface topography, and other factors not observed in this study.

4.0 INFILTRATION TESTING

We conducted two on-site field infiltration tests at locations and depths of 5 feet and 10 feet bgs as indicated by Westech. Approximate locations are shown in Figure 2. On-site testing was conducted in general accordance with the encased falling-head procedure in general accordance with Division 004 in Appendix C of the City of Salem Public Works Design Standards.

Our general procedure included excavating a 6-inch-diameter hole using hollow-stem auger drilling techniques and placing approximately 2 inches of clean gravel in the bottom of the auger. The auger was then filled with clean water to approximately 1 foot above the gravel at the bottom of the drilled hole. The initial fill of water did not drain into the soil within 10 minutes, so the water level was maintained, and the soil was allowed to saturate for 4 hours. Following the presoak period, water levels were measured, and the augers were refilled to approximately 12 inches above the soil at the end of each round of testing. The drop in water level was measured over a 1- to 3-hour period at each location. Field test results are summarized in Table 1.

Table 1. Field Measured Infiltration Results

Infiltration Test No.	Depth (feet)	USCS[1] Material Type	Percent Fines Content	Field Measured Infiltration Rate[2] (in/hr)[3]
INF-1	10	ML	95%	0.1
INF-2	5	ML	90%	0.2

Notes:

1. USCS = Unified Soil Classification System
2. Appropriate factors should be applied to the field-measured infiltration rate, based on the design methodology and specific system used. Measured rates may not reflect long-term sustainable rates area wide.
3. In/hr = inches per hour

As shown in Table 1, field-measured infiltration rates were relatively low over the period of testing, as expected in fine-grained soils in the area. Field measurements are limited to the accuracy of equipment employed to conduct the test and should be considered accurate to the nearest 10^{th} of an inch per hour.

In general, field-measured rates represent a relatively short-term infiltration rate, and factors of safety have not been applied for the type of infiltration system being considered, or for variability that may be present in the on-site soil. Even with higher tested values, appropriate correction factors should be applied by the project civil engineer to account for long-term infiltration parameters. From a geotechnical perspective, we recommend a factor of safety (correction factor) of at least 2 be applied to this type of field infiltration testing result to account for potential soil variability with depth and location within the area tested. In addition, the stormwater system design engineer should determine and apply appropriate remaining correction factor values, or factors of safety, to account for repeated wetting and drying that occurs in this area, degree of in-system filtration, frequency and type of system maintenance, vegetation, potential for siltation and bio-fouling, etc., as well as system design correction factors for overflow or redundancy, and base and facility size.

The actual depths, lateral extent, and estimated infiltration rates can vary from the values presented above. Field testing/confirmation during construction is often required in large or long systems or other situations where soil conditions may vary within the area where the system is constructed. The results of this field testing might necessitate that the infiltration locations be modified to achieve the design infiltration rate.

The infiltration flow rate of a focused stormwater system typically diminishes over time as suspended solids and precipitates in the stormwater slowly clog the void spaces between the soil particles or cake on the infiltration surface. The serviceable life of a stormwater system can be extended by pre-filtering or with ongoing accessible maintenance. Eventually, most systems will fail and will need to be replaced or have media regenerated or replaced.

At a minimum, with low infiltration rates anticipated for on-site soils, we recommend that infiltration systems include an overflow that is connected to a suitable discharge point or other means of managing stormwater in low-infiltration areas. Also, infiltration systems can cause localized high groundwater levels and should not be located near basement walls, retaining walls, or other embedded structures unless these are specifically designed to account for the resulting hydrostatic pressure. Infiltration locations should not be located on sloping ground, unless approved by a geotechnical engineer, and should not be at a location that allows for lateral flow toward a slope face, such as a mounded water condition, or too close to a slope face.

4.1 SUITABILITY OF INFILTRATION SYSTEMS

Successful design and implementation of stormwater infiltration systems and whether a system is suitable for development depends on several site-specific factors. Stormwater infiltration systems are generally best suited for sites having sandy or gravelly soil with saturated hydraulic conductivities greater than 2 in/hr. Sites with silty or clayey soil, such as found on this site, are generally not well- suited for long-term stormwater infiltration or as a sole method of stormwater infiltration. Soils that have fine-grained matrices are susceptible to volumetric change and softening during wetting and drying cycles. Fine-grained soils also have large variations in the magnitude of infiltration rates because of bedding and stratification that occurs during alluvial deposition and often have thin layers of less permeable or impermeable soil within a larger layer.

Local groundwater conditions also significantly affect the capacity to infiltrate from a stormwater system. Sites with shallow groundwater can result in groundwater mounding. A hydraulic gradient that reaches the level of water in the soil immediately drops to zero and local groundwater will rise and mound and the infiltration rate slows dramatically, resulting in overflows or system flooding (failure). Groundwater mounding can also negatively impact structures, slopes, or other areas adjacent to the stormwater infiltration facility. Typically, we do not recommend using infiltration systems where groundwater is less than 10 feet below the bottom of the proposed system unless the host soil is very permeable and consistently graded and will not cause mounding. Some jurisdictions require a minimum of 5 or 10 feet between high groundwater conditions and the bottom of proposed facilities. Depending on the size of the project, adjacent features such as streams that can source water to a system instead of allowing it to drain, and on-site soil infiltration capacities, there may be conditions where even a 10-foot separation between the level of groundwater and the base of the infiltration system may not be sufficient.

Considering relatively low measured field infiltration rates, on-site infiltration will likely be minimal during wet times of the year as a function of wet fine-grained host soils. In addition, infiltration may cause mounding of groundwater if areas of perched water are present in the area. We do not recommend stormwater infiltration be used as the exclusive method of stormwater management and recommend an overflow be a part of system design.

5.0 CONCLUSIONS

Based on our explorations, testing, and analyses, it is our opinion that the site is suitable for the proposed project from a geotechnical standpoint, provided the recommendations in this report are incorporated into the project design and implemented during construction. We offer the following conclusions regarding geotechnical engineering design and construction at the site.

- Near-surface soil is sensitive to disturbance when at a moisture content that is above optimum. As discussed in the report, the subgrade should be protected from disturbance and damage by construction traffic. Proper subgrade protection during earthwork will be essential. Cement amendment of the subgrade should be considered if construction takes place during wet-weather months.

- Proposed buildings can be supported on conventional shallow footings bearing on 1-foot-thick gravel pads placed over firm, undisturbed native soil, or on structural fill over firm native soils.
- Low infiltration rates were observed on the site. Based on the soil conditions encountered, we do not recommend on-site stormwater infiltration as a sole method of stormwater disposal.
- Within the asphalt section of Macleay Road, a pavement fabric was observed below the upper 1.5-inch lift of AC within the total asphalt section. For the roadway widening of Macleay Road, the existing asphalt section should be sawcut through the full existing asphalt section to ensure that the fabric that may be present (it was not observed in all of the exploration locations) is cut through prior to demolition/removal if required as part of preparation for placing the widened section.
- New signal poles and signs will require drilled shaft foundations.

6.0 DESIGN RECOMMENDATIONS

6.1 SHALLOW FOUNDATION SUPPORT RECOMMENDATIONS

Based on the results of our explorations, and assumed building loads, the proposed buildings can be supported on conventional shallow footings bearing on 1-foot-thick gravel pads placed over firm, undisturbed native soil, or on structural fill over firm native soils. Continuous wall and isolated spread footings should be at least 16 and 20 inches wide, respectively. The bottom of exterior column or continuous footings should be at least 18 inches below the adjacent exterior grade. The bottom of the interior footings should be established at least 12 inches below the base of the slab. Foundations should not be established on soft soil. Grading plans were not available at the time of this report.

Gravel pads should extend 6 inches beyond the margins of the foundations for every foot excavated below the foundations' base grade, and should consist of imported granular material as described in the Imported Granular Material section. The imported granular material should be compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D1557, or until well keyed, as determined by one of our geotechnical staff. We recommend that a member of our geotechnical staff observe the prepared footing subgrade before placement of gravel pads as well.

6.1.1 Shallow Foundation Subgrade Preparation

Subgrades beneath proposed structural elements should be prepared as described below and in the Earthwork Recommendations section of this report. We recommend loose or disturbed soils resulting from prior site uses or from foundation excavation be removed before placement of gravel pads. Foundation bearing surfaces should not be exposed to standing water. If water infiltrates and pools in the excavation, the water, along with any disturbed soil, should be removed before placement of reinforcing steel and concrete.

We recommend Central observe all foundation subgrades before placement of concrete forms and reinforcing steel to determine that bearing surfaces have been adequately prepared and the soil conditions are consistent with those observed during our explorations.

6.1.2 Spread Footings

We recommend conventional footings be proportioned using a maximum allowable bearing pressure of 2,500 psf if supported on firm native soils or on structural fill placed over firm native soils. This bearing pressure applies to the total of dead and long-term live loads and may be increased by one-third when considering earthquake or wind loads. This is a net bearing pressure. The weight of the footing and overlying backfill can be ignored in calculating footing sizes.

6.1.3 Foundation Settlement

Foundations designed and constructed as recommended are expected to experience settlements of less than 1 inch. Differential settlements of up to one-half of the total settlement magnitude can be expected between adjacent footings supporting comparable loads.

6.1.4 Shallow Foundation Lateral Resistance

Lateral loads can be resisted by a combination of friction between the footing and the supporting soil, and by the passive lateral resistance of the soil surrounding the embedded portions of the footings. A coefficient of friction between the concrete and gravel pad of 0.45 and a passive lateral resistance corresponding to an equivalent fluid density of 250 pounds per cubic foot (pcf) may be used for design. These values are appropriate for foundation elements that are poured directly against the native soils or surrounded by compacted structural fill.

The passive earth pressure and friction components may be combined, provided the passive component does not exceed two-thirds of the total.

The passive earth pressure value is based on the assumptions that the adjacent grade is level and static groundwater remains below the base of the footing throughout the year. The top 1 foot of soil should be neglected when calculating passive lateral earth pressures unless the adjacent area is covered with pavement. The lateral resistance values do not include safety factors.

6.2 FLOOR SLABS

Satisfactory subgrade support for floor slabs on grade supporting the planned 150-psf floor loads can be obtained provided the floor slab subgrade is as described in the Earthwork Recommendations section of this report, including re-working and compacting upper site soils disturbed as part of prior site use. Slabs should be reinforced according to their proposed use and per the structural engineer's recommendations. Subgrade support for concrete slabs can be obtained from the firm native soils underlying the topsoil or on structural fill placed over firm native soils.

We recommend that on-grade slabs be underlain by a minimum 6-inch-thickness of aggregate base in order to provide the structural design support for subgrade reaction as described below and to act as a capillary break material to reduce the potential for moisture migration into the slab. The aggregate base section should be placed as recommended in the Fill Placement and Compaction section of this report.

If dry on-grade slabs are required, for example at interior spaces where adhesives are used to anchor carpet or tile to the slab, a waterproof liner may be placed as a vapor barrier below the slab. The vapor barrier should be selected by the structural engineer and should be accounted for in the design floor section and mix design selection for the concrete, to accommodate the effect of the vapor barrier on concrete slab curing.

Load-bearing concrete slabs prepared as recommended should be designed assuming a modulus of subgrade reaction (k) of 150 pounds per square inch (psi) per inch. This value is for a 1-foot-by-1-foot square plate. The coefficient of subgrade reaction for a foundation varies based on its minimum width according to the following equation:

$$k_s = k_{s1}\left[\frac{B+1}{2B}\right]^2$$

Where k_s is the coefficient of subgrade reaction, k_{s1} is the coefficient of subgrade reaction for a 1-foot-by-1-foot plate, and B is the minimum width or lateral dimension of the mat.

We estimate that concrete slabs constructed as recommended will settle less than ½ inch. Floor slab subgrades should be evaluated according to the Site Subgrade Preparation and Evaluation section of this report.

6.3 DRILLED-SHAFT FOUNDATION RECOMMENDATIONS

The proposed new traffic signal requires a drilled-shaft foundation consistent with the requirements of City of Salem Department of Public Works, Standard Plan, Traffic Signal Support Foundation Details (Sheet No. 758), and Traffic Signal Support Design Specifications (Sheet No. 759). Drilled-shaft support has the advantage of large moment transfer capacity as well as extending to depths where firm support is available.

Foundation depths should be based on selected signal pole type, including anticipated loading, as well as soil engineering properties derived from field-correlated values and laboratory testing since the upper 24.5 feet are considered “poor soil conditions” relative to City Sheet No 759. Soft upper soils may require embedment to the depth of underlying gravel (24.5 feet bgs). We recommend an assumed groundwater depth of 15 feet bgs during design to account for layers of potentially perched groundwater. Total depth determination should also include consideration or torsional resistance and foundation depths calculated using the American Association of State Highway and Transportation Officials (AASHTO) Brom’s Design Method for cohesive soils.

6.3.1 Drilled-Shaft Lateral Foundation Capacity

Determination of embedment depth for shaft foundations requires analysis of lateral resistance capacity of the surrounding soil. Lateral resistance design should be based on allowable deflection at the top of the shaft as determined by the design engineer. A computer program, such as Ensoft’s LPILE, can be used to calculate the lateral capacity of drilled-shaft foundations. LPILE uses lateral soil reaction (p) and lateral deflection (y) curves generalized from field load tests and soil input properties to approximate lateral pile deflections and moments. Recommended soil parameters for use by the LPILE program are presented in Table 2.

Table 2. Recommended LPILE Soil Parameters

Depth Below Existing Grade (ft)	Effective Unit Weight (pcf)(1)	Cohesion, C (psf) (2)	Soil Modulus K (pci) (3)	Strain Value, ε_{50}	Soil Model
0 – 30	53	150	100	0.007	Silt (Cemented C-phi)

1) pcf = Pounds per cubic foot
2) psf = Pounds per square foot
3) pci = Pounds per cubic inch

6.3.2 Quality Control

Quality of shaft construction will be critical to provide acceptable settlement behavior. At a minimum, we recommend the following quality control measures:

- The hole bottom should be thoroughly cleaned as required by OSSC 00512.43 (Drilled Shaft Excavation).
- A qualified technician should be present during construction to verify subsurface conditions are as interpreted and the general design intentions are met during construction.

6.4 SEISMIC DESIGN

Parameters provided on Table 3 are based on the conditions encountered during our subsurface exploration program and the procedures and requirements outlined in the 2021 IBC. Per American Society of Civil Engineers (ASCE) 7-22 Section 11.4.8, a site-specific response analysis is required for Site Class F sites, and a ground motion hazard analysis or site-specific response analysis is required to determine the design ground motions for structures on Site Class D and E sites with S_1 greater than or equal to 0.2g.

For this project, the site is classified as Site Class D; therefore, the provisions of 11.4.8 are applicable. Alternatively, the parameters listed in Table 5-1 may be used to determine the design ground motions if the exceptions provided in ASCE 7-16 Supplement 3 are met. The applicable exceptions are provided below.

From ASCE 7-16 Supplement 3
Exception: A ground motion hazard analysis not required:

1. Where the values of the parameter S_{M1} determined by Eq. (11.4-2) is increased by 50% for all applications of S_{M1} in the standard. And:
2. The resulting value of the parameter S_{D1} determined by Eq. (11.4-4) shall be used for all applications of S_{D1} in the standard.

Table 3. Mapped ASCE 7-16 Seismic design parameters

Parameter	Recommended Value[1,2]
Site Class	D
Mapped Spectral Response Acceleration at Short Period (S_S)	0.792 g
Mapped Spectral Response Acceleration at 1 Second Period (S_1)	0.397 g
Site Modified Peak Ground Acceleration (PGA_M)	0.452 g
Site Amplification Factor at 0.2 second period (F_a)	1.183
Site Amplification Factor at 1.0 second period (F_v)	1.903
Design Spectral Acceleration at 0.2 second period (S_{DS})	0.625 g
Design Spectral Acceleration at 1.0 second period (S_{D1})	0.504 g

Note:
[1] Parameters developed based on Latitude 44.913505° and Longitude -122.954657°.
[2] These values are only valid if the structural engineer utilizes ASCE 7-16 Supplement 3 Exception 1.

Seismic design parameters are applicable to site structures using the exception noted above (Supplement 3). The exception typically does not significantly impact small structures or structures with low fundamental structural periods. Studies noted in Section 11.4.8 of ASCE-7-22 as described above can provide significant reductions in design-level requirement compared to designs based on the exception of Supplement 3 for longer-period and large-footprint structures.

6.5 LIQUEFACTION POTENTIAL

Liquefaction is a phenomenon caused by a rapid increase in pore water pressure that reduces the effective stress between soil particles to near zero. The excessive buildup of pore water pressure results in the sudden loss of shear strength in a soil. Granular soil, which relies on interparticle friction for strength, is susceptible to liquefaction until the excess pore pressures can dissipate. Sand boils and flows observed at the ground surface after an earthquake are the result of excess pore pressures dissipating upwards, carrying soil particles with the draining water. In general, loose, saturated sand soil with low silt and clay contents is the most susceptible to liquefaction. Low plasticity, silty sand may be moderately susceptible to liquefaction under relatively higher levels of ground shaking.

Based on the subsurface conditions encountered in the explorations and the laboratory testing results, it is our opinion that there is a low risk of seismic-induced liquefaction at the site during the design-level earthquake.

6.6 RETAINING WALLS

6.6.1 Assumptions

Our retaining wall design recommendations are based on the following assumptions: (1) walls consist of conventional, cantilevered retaining walls, (2) walls are less than 8 feet in height, (3) the backfill is drained and consists of imported granular materials, and (4) the backfill has a finish slope flatter than 4H:1V. Re-evaluation of our recommendations will be required if the retaining wall design criteria for the project varies from these assumptions.

6.6.2 Subgrade Preparation

Wall footings should bear on a minimum 12-inch-thick layer of imported granular material underlain by firm, undisturbed native soil and be prepared as listed in the Site Subgrade Preparation and Evaluation section of this report. The imported granular material should be fairly well graded between coarse and fine material, have less than 6 percent by dry weight passing the U.S. Standard 200 sieve, should have at least two mechanically fractured faces, and should be compacted to 95 percent of maximum dry density as obtained from ASTM D1557.

6.6.3 Wall Design Parameters

Wall footings prepared as recommended should be sized based on an allowable bearing pressure of 2,500 psf. The weight of the footing and overlying backfill can be ignored in calculating footing sizes.

For unrestrained retaining walls, an active equivalent fluid pressure of 35 pcf should be used for design. Where retaining walls are restrained from rotation (such as basement walls), an at-rest equivalent fluid pressure of 55 pcf should be used for design. A superimposed seismic lateral force should be calculated based on a dynamic force of 7H2 pounds per lineal foot of wall, where H is the height of the wall in feet, and applied as a distributed load with the centroid located at a distance of 0.6H from the base of the wall.

If surcharges (e.g., structure foundations, concrete slabs, vehicles, steep slopes, terraced walls, etc.) are located within a horizontal distance from the back of a wall equal to the height of the wall, additional pressures will need to be accounted for in the wall design. Our office should be contacted for appropriate wall surcharges based on the actual magnitude and configuration of the applied loads. The base of the wall footing should extend a minimum of 12 inches below the lowest adjacent finish grade.

Lateral loads on the proposed structures can be resisted by a combination of sliding resistance on the base of footings and passive earth pressure on the sides of footings. We recommend a coefficient of friction of 0.35 for footings bearing on undisturbed, native silt, and 0.45 for footings bearing on granular engineered structural fill.

Passive earth pressures on the sides of buried spread footings may be calculated using an equivalent fluid pressure of 300 pcf per foot of embedment. For this value, backfill against the footing should be compacted to at least 92 percent of the maximum dry density as obtained from ASTM D1557. The upper foot of embedment should be neglected unless protected by pavement or concrete slabs on grade.

6.6.4 Wall Drainage and Backfill

The above design parameters have been provided assuming back-of-wall drains will be installed to prevent buildup of hydrostatic pressures behind all walls. If a drainage system is not installed, Central should be contacted for revised design forces.

Backfill material placed behind retaining walls and extending a minimum horizontal distance of H, where H is the height of the retaining wall, should consist of select granular wall backfill meeting the requirements described in the Structural Fill and Backfill section. All wall backfill should be compacted to at least 92 percent of the maximum dry density as obtained from ASTM D1557. Wall backfill in the top 2 feet should be compacted to at least 95 percent of the maximum dry density when under structural areas such as footings, concrete slabs, or pavement.

Perforated collector pipes should be placed at the base of the granular backfill behind the walls. The pipe should be embedded in a minimum 1-foot-wide zone of angular drain rock and the drain rock should be wrapped in a drainage geotextile fabric. The collector pipes should discharge at an appropriate location away from the base of the wall. The discharge pipe should not be tied directly into stormwater drain systems, unless measures are taken to prevent backflow into the drainage system of the wall.

6.6.5 Settlement

Settlement of up to 1 percent of the wall height commonly occurs immediately adjacent to the wall as the wall rotates and develops active lateral earth pressures. Consequently, we recommend that construction of flatwork adjacent to retaining walls be postponed at least 4 weeks after backfilling of the wall, unless survey data indicates that settlement is complete prior to that time.

6.7 PAVEMENT

New pavement is proposed for on-site parking lots and widening of Macleay Road. The pavement section will be conventional (AC over Aggregate Base). Pavement sections should be installed on native subgrade or engineered fills prepared in conformance with the Earthwork Recommendations and Structural Fill sections. All thicknesses are intended to be the minimum acceptable. Design of the recommended pavement section assumes that construction will be completed during an extended period of dry weather. Wet-weather construction could require an increased thickness of aggregate base.

Construction traffic should be limited to haul roads. Construction traffic should not be allowed on new pavement. If construction traffic is to be allowed on newly constructed pavement, an allowance for this additional traffic will need to be made in the design pavement section. The pavement thicknesses do not account for construction traffic, and haul roads and staging areas should be used.

Pavement material recommendations are provided in the Structural Fill and Backfill section of this report.

6.7.1 On-Site Parking Lot

6.7.1.1 On-Site Asphalt Concrete Pavement

Pavement subgrades should be prepared in accordance with the Earthwork Recommendations section of this report. Our pavement recommendations assume that traffic at the site will consist of occasional truck traffic and passenger cars. We do not have specific information on the frequency and type of vehicles that will use the area; however, we have based our design analysis on traffic loading consistent with heavy trucks to account for delivery- and service-type vehicles and passenger car traffic for the heavy-duty pavement sections, and passenger car traffic only for the light-duty pavement sections and the assumed equivalent single-axle loads (ESALS) noted in the recommended sections. Additionally, our analyses are based on the following:

- The pavement subgrades, fill subgrades, and site earthwork used to establish road grades below the Aggregate Subbase and Aggregate Base materials have been prepared as described in the Earthwork Recommendations section of this report.
- A resilient modulus of 20,000 psi for compacted Aggregate Subbase and Aggregate Base materials.
- A resilient modulus of 6,000 psi for improved tilled zone site fine-grained soils.

- Initial and terminal serviceability indices of 4.2 and 2.0, respectively.
- Reliability and standard deviations of 75 percent and 0.45, respectively.
- Structural coefficients of 0.42 and 0.10 for the asphalt and base rock, respectively.
- A 20-year design life with no growth.
- Traffic loading as follows:
  - Parking lot stalls (passenger vehicle parking only).
  - Light duty (access roads for parking lots with up to five trucks per day).

If any of the noted assumptions vary from project design use, our office should be contacted with the appropriate information so that the pavement designs can be revised or confirmed adequate. Additionally, we have provided the recommend the following AC pavement section options for on-site Local roads and the AC-paved shared access road:

On-site Parking Stalls and Light-Duty AC Pavement Section without Cement Amendment

- Design life 20-years
- 3.0 inches Level 2, dense ACP
- 9.0 inches of aggregate base
- Subgrade stabilization (if required)
- Subgrade geotextile
- Assumed ESAL of 10,000

On-site Parking Stalls and Light-Duty AC Pavement Section with Cement Amendment

- Design life 20-years
- 3.0 inches Level 2, dense ACP
- 4.0 inches of aggregate base
- 12.0 inches of cement treated base
- Assumed ESAL of 10,000
- Assumed ESAL of 20,000

6.7.2 Macleay Road Widening

Pavement subgrades should be prepared in accordance with the Earthwork Recommendations section of this report. We have provided pavement section recommendations based on the subsurface conditions observed at our explorations and City of Salem Design Standard. Macleay Road is classified as a minor arterial. Design 18- kip ESALs given in the City of Salem Design Standard for minor arterials are 4,000,000. Our recommended section differs from the existing pavement section of Macleay Road; variable pavement sections observed at our explorations are most likely a result of original design and construction, and subsequent maintenance and construction in the area. Our analyses are based on the following:

- The pavement subgrades, fill subgrades, and site earthwork used to establish road grades below the aggregate base materials have been prepared as described in the Earthwork Recommendations section of this report.

- A resilient modulus of 20,000 psi for compacted aggregate base materials.
- A resilient modulus of 4,500 psi for soil observed in our explorations for Macleay Road.
- Initial and terminal serviceability indices of 4.2 and 2.5, respectively.
- Reliability and standard deviations of 90 percent and 0.45, respectively.
- Structural coefficients of 0.41 and 0.10 for the asphalt and base rock, respectively.
- A 20-year design life with 2.0-percent growth.

If any of the noted assumptions vary from project design use, our office should be contacted with the appropriate information so that the pavement designs can be revised or confirmed adequate.

Macleay Road Widening AC Pavement Section

- Design life 20-years
- 9.0 inches Level 2, dense ACP
- 12 inches of aggregate base
- Subgrade stabilization (if required)
- Subgrade geotextile (optional)
- Design ESAL of 4,000,000

Design roadway sections provided above are greater than existing roadway sections. This is likely a result of existing sections designed to a lesser ESAL load (likely on the order of 500,000 ESAL) or being designed to another lesser existing roadway standard at the time of construction.

Within the asphalt section of Macleay Road, a pavement fabric was observed beneath the upper 1.5-inch upper lift of asphalt section. For the roadway widening of Macleay Road, the existing asphalt section should be sawcut through the full existing asphalt section during demolition to reduce the risk of pulling the pavement fabric from the existing asphalt and causing damage to the asphalt section that is planned to remain in place.

6.7.3 Cold Weather Paving Considerations

In general, AC paving is not recommended during cold weather (surface temperature less than 40 degrees Fahrenheit). Compacting under these conditions can result in low compaction and premature pavement distress.

Each AC mix design has a recommended compaction temperature range that is specific for the particular AC binder used. In colder temperatures, it is more difficult to maintain the temperature of the AC mix as it can lose heat while stored in the delivery truck, as it is placed, and in the time between placement and compaction. In Oregon, the surface temperature during paving should be at least 40 degrees Fahrenheit for lift thicknesses greater than 2.5 inches and at least 50 degrees Fahrenheit for lift thicknesses between 2 and 2.5 inches.

If paving activities must take place during cold-weather construction as defined above, the project team should be consulted and a site meeting should be held to discuss ways to lessen low-compaction risks.

7.0 EARTHWORK RECOMMENDATIONS

7.1 SITE PREPARATIONS

7.1.1 Demolition

In general, initial site preparation and primary earthwork operations will include demolition and removal of hardscaped surfaces, stripping and grubbing of upper organics, removal of undocumented fill, grading to create level working surfaces, excavating and filling for roads, pavements, excavation for foundations and utilities, recompacting (dry weather) or replacing (wet weather) unsuitable soil in areas of the site that are to receive fill, and fine-grading to establish final grades for structural improvements.

Within the asphalt section of Macleay Road, a pavement fabric was observed approximately 1.5-inches within the asphalt section. For the roadway widening of Macleay Road, the existing asphalt section should be sawcut through the full existing asphalt section during demolition to reduce the risk of pulling the pavement fabric from the existing asphalt and causing damage to the asphalt section that is planned to remain in place.

All existing utilities in proposed earthwork construction areas should be identified prior to excavation. Decommissioned utility lines beneath proposed structures and pavement should be completely removed or filled with grout to eliminate void space and reduce potential settlement of new structures. Voids resulting from removal of improvements or loose soil in utility lines should be backfilled with compacted structural fill, as discussed in the Structural Fill and Backfill and Fill Placement and Compaction sections. Soft or loose soil encountered in utility line excavations should be removed and replaced with structural fill when it is located within structural areas.

Existing voids and new depressions created during site preparation and resulting from the removal of existing utilities or other subsurface elements should be cleaned of loose soil or debris down to firm soil before fill placement and the sides of the excavation sloped at a minimum of 1H:1V to allow for more uniform compaction at the edges of the excavation. Disturbance to a greater depth should be expected if site preparation and earthwork are conducted during periods of wet weather.

Debris materials generated during demolition of existing improvements or relocation of utilities should be transported off site for disposal or stockpiled in areas designated by the owner.

7.1.2 Grubbing and Stripping

The existing root zone in landscaped areas should be stripped and removed from all fill areas. Based on our explorations, stripping of upper organics will generally extend to depths of 4 inches below existing grades. Deeper stripping depths may be required in lower-lying wet areas or areas of thicker vegetation growth or brush- or tree-covered areas. The actual stripping depth should be based on field observations at the time of construction. Stripped material should be transported off site for disposal or used in landscaped areas.

Trees and shrubs, if present within areas to be developed, should be removed from fill areas. In addition, root balls should be grubbed out to the depth of the roots, which could exceed 3 feet bgs. Depending on the methods used to remove root balls, considerable disturbance and loosening of the subgrade could occur during site grubbing. We recommend that soil disturbed during grubbing operations be removed to expose firm, undisturbed subgrade. The resulting excavations should be backfilled with structural fill.

7.1.3 Undocumented Fill

Undocumented fill was observed in explorations B-1, B-2, and B-3 to depths of between 2 and 6 feet bgs within Macleay Road, and on the site, the upper 12 inches of soil are generally disturbed from prior use (classified as undocumented fill in its disturbed state). Due to the variable thickness and composition of the fill in the roadway and the unknown methods of placement and compaction, and the previously tilled or disturbed upper soils on the site, reliable strength properties for undocumented fill are extremely difficult to predict and there is a risk of differential settlement of foundations, floor slabs, and pavement.

There is a small risk of poor performance of pavement established directly over the existing undocumented fill. The risk of poor performance can be reduced if soft fill is improved through methods such as moisture condition and compaction, cement amendment of the undocumented fill, or removal and replacement of the undocumented fill using imported granular material.

Where encountered, undocumented fill should be removed from under new building foundations to a minimum depth of 12 inches and replaced with granular pads consisting of imported granular material or should be compacted in place prior to placement of additional structural fill if conditions allow. Central should observe all prepared subgrade and can assist in identifying loose, soft, and/or organic undocumented fill and recommending removal depths. The project budget should include a contingency for additional removal in isolated areas as necessary.

7.2 SITE SUBGRADE PREPARATION AND EVALUATION

Upon completion of site preparation activities, exposed subgrades should be proof-rolled with a fully loaded dump truck or similar heavy rubber-tired construction equipment where space allows to identify soft, loose, or unsuitable areas. Probing may be used for evaluating smaller areas or where proof-rolling is not practical. Proof-rolling and probing should be conducted prior to placing fill and should be performed by a representative of Central, who will evaluate the suitability of the subgrade and identify areas of yielding that are indicative of soft or loose soil. We anticipate that there will be areas where soft or otherwise unsuitable soil is identified during subgrade evaluation. Unsuitable soil should be replaced by imported granular material or should be improved by scarifying and compacting the material in accordance with the Structural Fill and Backfill section.

As discussed in the Subgrade Protection and Wet-Weather Considerations section of this report, the fine-grained soil at the surface can be sensitive to small changes in moisture content and will be difficult, or not possible, to compact adequately during wet weather. While tilling and compacting the subgrade is the economical method for subgrade improvement, it will likely only be possible during extended dry periods and following moisture-conditioning of the soil.

During wet weather, or when the exposed subgrade is wet or unsuitable for proof-rolling, the prepared subgrade should be evaluated by observing excavation activity and probing with a steel foundation probe. Observations, probing, and compaction testing should be performed by a member of our staff. Wet soil that has been disturbed due to site preparation activities or soft or loose zones identified during probing should be removed and replaced with compacted structural fill.

7.3 SUBGRADE PROTECTION AND WET-WEATHER CONSIDERATIONS

Upper site soils are highly susceptible to moisture. If wet-weather construction practices are necessary based on conditions observed at the time of construction, it may be necessary to use track-mounted equipment, load removed material into trucks supported on gravel haul roads, use gravel working pads, and employ other methods to reduce ground disturbance. The contractor should be responsible for protecting the subgrade during construction.

Earthwork planning should include considerations for minimizing subgrade disturbance. We provide the following recommendations if wet-weather construction is considered:

- The ground surface in and around the work area should be sloped so that surface water is directed to a sump or discharge location. The ground surface should be graded such that areas of ponded water do not develop. Measures should be taken by the contractor to prevent surface water from collecting in excavations and trenches. Measures should be implemented to remove surface water from the work areas.
- Earthwork activities should not take place during periods of heavy precipitation.
- Slopes with exposed soils should be covered with plastic sheeting or similar means.
- Site soils should not be left in a disturbed or uncompacted state and exposed to moisture. Sealing surficial soils by rolling with a smooth-drum roller prior to periods of precipitation may reduce the extent to which these soils become wet or unstable.
- Construction activities should be scheduled so that the length of time that soil is left exposed to moisture is reduced to the extent practicable.
- Construction traffic should be restricted to specific areas of the site, preferably areas that are not susceptible to wet-weather disturbance such as haul roads and areas that are adequately surfaced with working pad materials.
- When on-site, moisture-sensitive soils are wet of optimum, they are easily disturbed and will not provide adequate support for construction traffic nor for the proposed development. The use of granular haul roads and staging areas will be necessary to support heavy construction traffic. Generally, a 12- to 16-inch-thick mat of Imported Granular Material (see reference in the Structural Fill and Backfill section of this report) should be sufficient for light staging areas for the building pad and light staging activities but is not expected to be adequate to support repeated heavy equipment or truck traffic. The thickness of the Imported Granular Material for haul roads and areas with repeated heavy construction traffic should be increased to between 18 and 24 inches. The actual thickness of haul roads and staging areas should be determined at the time of construction and based on the contractor's approach to site development and the amount and type of construction traffic.
- The Aggregate Base thicknesses described in the Pavement Recommendations section of this report are intended to support post-construction design traffic loads. The design aggregate base thicknesses will likely not support repeated heavy construction traffic during site construction or during pavement construction. A thicker aggregate base section as described above for haul roads will likely be required to support construction traffic.
- During periods of wet weather, concrete should be placed as soon as practical after preparing foundation excavations. Foundation bearing surfaces should not be exposed to standing water. Should water infiltrate and pool in the excavation, the water should be removed, and the foundation subgrade should be re-evaluated before placement of reinforcing steel or concrete. Foundation subgrade protection, such as a 3- to 4-inch thickness of aggregate base or lean concrete, may be necessary if footing excavations are exposed to extended wet-weather conditions.

During wet weather, or when the exposed subgrade is wet or unsuitable for proof-rolling, the prepared subgrade should be evaluated by observing excavation activity and probing with a steel foundation probe. Observations and probing should be performed by a member of our staff. Wet soil that has been disturbed due to site preparation activities, or soft or loose zones identified during probing, should be removed and replaced with imported granular material.

7.4 DEWATERING

As discussed in the Groundwater section of this report, groundwater was encountered in our explorations. However, we do not expect groundwater to be a major factor during shallow excavations and earthwork, provided the contractor is aware of the potential for shallow water. Excavations that extend into saturated/wet soils, or excavations that extend into perched groundwater, should be dewatered. Sump pumps are expected to adequately address groundwater encountered in shallow excavations. In addition to groundwater seepage, surface water inflow to the excavations during the wet season can be problematic. Provisions for surface water control during earthwork and excavations should be included in the project plans and should be installed prior to commencing earthwork.

7.5 EXCAVATION

Excavations will be required for the installation of new foundations, utilities, and other earthwork activities. Conventional earthmoving equipment in proper working condition should generally be capable of making the necessary excavations. Excavations deeper than 4 feet bgs will likely require shoring or should be sloped.

Open excavation techniques may be used to excavate trenches to depths of 4 feet bgs, provided the walls of the excavation are cut at a slope of 1H:1V and groundwater seepage is not present. At this inclination, the slopes may ravel and require some ongoing repair. Excavations should be flattened if excessive sloughing or raveling occurs. In lieu of large and open cuts, approved temporary shoring may be used for excavation support. A wide variety of shoring and dewatering systems are available. Consequently, we recommend the contractor be responsible for selecting the appropriate shoring and dewatering systems.

If box shoring is used, it should be understood that box shoring is a safety feature used to protect workers and does not prevent caving. If excavations are left open for extended periods of time, caving of sidewalls will likely occur. The presence of caved material will limit the ability to properly backfill and compact the trenches. The contractor should be prepared to fill voids between the box shoring and the sidewalls of the trenches with sand or gravel before caving occurs.

If shoring is used, we recommend that the type and design of the shoring system be the responsibility of the contractor, who is in the best position to choose a system that fits the overall plan of operation.

7.5.1 Drilled Shafts

Groundwater will likely be present in deeper shaft foundation excavations. Sloughing and caving may occur where excavations extend below the groundwater table or if the excavations are dry but left open for extended periods of time (more than a few hours). Depending on the foundation installation methods, dewatering and the use of temporary casing and/or drilling slurry may be required. The use of open-hole drilling methods may not be effective for shaft foundation excavations below the groundwater table. The shaft foundation system should be constructed in conformance with the specifications provided in Oregon Standard Specifications for Construction (OSSC) 00963 (Signal Support Drilled Shafts).

7.6 DRAINAGE CONSIDERATIONS

7.6.1 Temporary

During earthwork at the site, the contractor should be responsible for the temporary drainage of surface water as necessary to prevent standing water and/or erosion at the working surface. During rough and finished grading of the site, the contractor should keep all pads and subgrade free of ponding water.

7.6.2 Surface

The ground surface around the finished building pads should be sloped away from the edge of the pad at a minimum 2 percent gradient for a distance of at least 5 feet. Downspouts or roof scuppers should discharge into a storm drain system that carries collected water to an appropriate stormwater system. Trapped planter areas should not be created adjacent to pavement and structures without providing means for positive drainage (e.g., swales or catch basins).

7.7 PERMANENT SLOPES

Permanent cut and fill slopes should not exceed 2H:1V. Slopes that will be maintained by mowing should be constructed steeper than 3H:1V. Access roads and pavement should be located at least 5 feet away from the top of cut and fill slopes. The setback should be increased to 10 feet for buildings. The slopes should be planted with appropriate vegetation to provide protection against erosion as soon as possible after grading. Surface water runoff should be collected and directed away from slopes to prevent water from running down the face of the slope.

7.8 STRUCTURAL FILL AND BACKFILL

7.8.1 General

Structural areas include areas beneath foundations, floor slabs, pavements, and any other areas intended to support structures or within the influence zone of structures. Fill intended for use in structural areas should meet the criteria for structural fill presented below. All structural fill soils should be free of debris, clay balls, roots, organic matter, frozen soil, man-made contaminants, particles with greatest dimension exceeding 4 inches (3-inch-maximum particle size in building footprints), and other deleterious materials.

Recommendations for suitable fill material are provided in the following sections.

7.8.2 On-Site Soil

The on-site material should generally be suitable for use as general structural fill, provided it is properly moisture-conditioned; free of debris, organic material, and particles over 6 inches in diameter; and meets the specifications provided in OSSC 00330.12 (Borrow Material). The suitability of soil for use as structural fill will depend on the gradation and moisture content of the soil. As the amount of fines in the soil matrix increases, the soil becomes increasingly sensitive to small changes in moisture content and achieving the required degree of compaction becomes more difficult or impossible.

When used as structural fill, native soil should be placed in lifts with a maximum uncompacted thickness of 6 to 8 inches and compacted to not less than 95 percent of the maximum dry density for granular soil, as determined by ASTM D1557.

If desired, an experienced geotechnical engineer from Central can determine the suitability of on-site soil encountered during earthwork activities for use as structural fill.

7.8.3 Imported Granular Material

Imported granular material used as structural fill should be pit- or quarry-run rock, crushed rock, or crushed gravel and sand. The imported granular material should also be durable, angular, and fairly well graded between coarse and fine material; should have less than 5 percent passing the U.S. No. 200 sieve (3 percent for retaining walls) by dry weight; and should have at least two mechanically fractured faces. In addition, aggregate base shall have a minimum of 75 percent fractured particles according to AASHTO T-355 and a sand equivalent of not less than 30 percent based on AASHTO T-176.

7.8.4 Trench Backfill

Backfill for pipe bedding and in the pipe zone should consist of well-graded granular material with a maximum particle size of ¾ inch and less than 5 percent passing the U.S. No. 200 sieve. The material should be free of organic matter and other deleterious materials. Further, the backfill should meet the pipe manufacturer's recommendations. Above the pipe zone backfill, imported granular material may be used as described above.

7.8.5 Aggregate Base

Imported granular material used as aggregate base for pavement and as referenced within this report should consist of ¾- or 1 ½-inch-minus material and meet the general specifications listed in OSSC 00640 (Aggregate Base and Shoulders) In addition, the aggregate should have less than 5 percent fines by dry weight and have at least two mechanically fractured faces. The aggregate base should be compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D1557.

7.8.6 Drain Rock Material

Drain rock should consist of angular, granular material with a maximum particle size of 2 inches. The material should be free of roots, organic material, and other unsuitable material; should have less than 2 percent by dry weight passing the U.S. Standard No. 200 sieve (washed analysis); and should have at least two mechanically fractured faces. Drain rock should be compacted to a well-keyed, firm condition.

7.8.7 Fill Placement and Compaction

Structural fill should be compacted at moisture contents that are within 3 percent of the optimum moisture content as determined by D 1557 (Modified Proctor). The optimum moisture content varies with gradation and should be evaluated during construction. Fill material that is not near the optimum moisture content should be moisture-conditioned prior to compaction.

Fill and backfill material should be placed in uniform, horizontal lifts and compacted with appropriate equipment. The appropriate lift thickness will vary depending on the material and compaction equipment used. Fill material should be compacted in accordance with Table 4. It is the contractor's responsibility to select appropriate compaction equipment and place the material in lifts that are thin enough to meet these criteria. However, in no case should the loose lift thickness exceed 18 inches. Initial lift thickness over pipe may need to be thicker than 18 inches to prevent damage to the pipe during the application of compactive effort.

A representative from Central should evaluate the compaction of every 2 vertical feet (or less) and 500 cubic yards of fill material placed. Compaction should be evaluated by compaction testing unless other methods are proposed for oversized materials and are approved by Central during construction. These other methods typically involve procedural placement and compaction specifications together with verification requirements such as proof-rolling.

Table 4. Compaction Criteria

Fill Type	Compaction Requirements		
	Percent Maximum Dry Density Determined by ASTM Test method D 1557 at ± 3% of Optimum Moisture		
	0 to 2 Feet Below Subgrade	➢ 2 Feet Below Subgrade	Pipe Zone
Fine-grained soils (non expansive)	92	92	-----
Imported Granular, maximum particle size < 1½ inch	95	92	-----
Imported Granular, maximum particle size 1½ inch to 6 inches (3-inch-maximum under building footprints)	n/a (proof-roll)	n/a (proof-roll)	-----
Retaining Wall Backfill*	95	92	-----
Nonstructural Zones	92	92	90
Trench Backfill	95	92	90

Note:

* Measures should be taken to prevent overcompaction of the backfill behind retaining walls. We recommend placing the zone of backfill located within 5 feet of the wall in lifts not exceeding about 6 inches in loose thickness and compacting this zone with hand-operated equipment such as a vibrating plate compactor or a jumping jack.

7.8.8 Soil Amendment with Cement

As an alternative to the use of imported granular material for wet-weather structural fill, an experienced contractor should be able to amend the on-site soil with portland cement to obtain suitable support properties. Successful use of soil amendment depends on the use of correct mixing techniques, soil moisture content, and amendment quantities. The amount of cement used during amendment should be based on an assumed soil dry unit weight of 100 pcf.

In addition, the new Oregon Department of Environmental Quality requirements under 1200C permits include additional requirements and testing runoff from sites where cement amendment is used.

Specific recommendations based on exposed site conditions for soil amendment can be provided if necessary. However, for preliminary design purposes, we recommend a target strength for cement-amendment subgrade of 100 psi. The amount of cement used to achieve this target generally varies with moisture content and soil type. It is difficult to predict field performance of soil cement amendment due to variability in soil response, and we recommend laboratory testing to confirm expectations. In general, 6 percent cement by weight of dry soil can be used when the soil moisture content does not exceed approximately 25 percent. If the soil moisture content is in the range of 25 to 35 percent, 7 to 8 percent by weight of dry soil is recommended. The amount of cement added to the soil may be need to be adjusted based on field observations and performance. Moreover, depending on the time of year and moisture content levels during amendment, water may need to be applied during tilling to appropriately condition the soil moisture content.

We recommend assuming a minimum cement ratio of 6 percent by dry weight if amendment is performed during the dry summer months. The cement should be increased to 7 to 8 percent if amendment occurs during any other time of the year.

We recommend cement amendment equipment be equipped with balloon tires to reduce rutting and disturbance of the fine-grained soil. A sheepsfoot or segmented pad roller with a minimum static weight of 40,000 pounds should be used for initial compaction of the fine-grained soil without the use of vibratory action. A smooth-drum roller with a minimum applied linear force of 700 pounds per inch should be used for final compaction. The amended soil should be compacted to at least 92 percent of the maximum dry density, as determined by ASTM D1557.

A minimum curing time of 4 days is required between amendment and construction traffic access. Construction traffic should not be allowed on unprotected, cement-amended subgrade. To protect the cement-amended surfaces from abrasion or damage, the finished surface should be covered with 4 to 6 inches of imported granular material.

Amendment depths for subgrade beneath buildings and pavement, haul roads, and staging areas are typically on the order of 12, 16, and 12 inches, respectively. The crushed rock typically becomes contaminated with soil during construction. Contaminated base rock should be removed and replaced with clean rock in pavement areas. The actual thickness of the amended material and imported granular material for haul roads and staging areas will depend on the anticipated traffic as well as the contractor's means and methods and should be the contractor's responsibility.

Cement amendment should not be attempted when the air temperature is below 40 degrees Fahrenheit or during moderate to heavy precipitation. Cement should not be placed when the ground surface is saturated or standing water exists.

Portland cement-amended soil is hard and has low-permeability. This soil does not drain well and is not suitable for planting. Future planted areas should not be cement amended, if practical, or accommodations should be made for drainage and planting. Moreover, cement amendment of soil within building areas must be done carefully to avoid trapping of water under floor slabs. We should be contacted if this approach is considered. Cement amendment should not be used if runoff during construction cannot be directed away from adjacent wetlands. In general, cement amendment is not recommended during cold weather or steady rainfall.

7.8.9 Asphalt Concrete Pavement (ACP)

The ACP should be Level 2, ½ inch (or finer), dense ACP according to OSSC 00744 (Asphalt Concrete Pavement) and generally compacted to 92 percent of the theoretical maximum density of the mix, as determined by AASHTO T209. Minimum and maximum lift thicknesses are generally held to be 2 and 3.5 inches, respectively, for ½-inch ACP.

Asphalt binder should be performance graded and conform to PG 64-22. The binder grade may be adjusted depending on the aggregate gradation and amount of recycled asphalt pavement and/or recycled asphalt shingled in the contractor's mix design controls.

7.8.10 Subgrade Geotextile

The subgrade geotextile should conform to OSSC 00350. The subgrade geotextile should conform to the minimum property values presented in ODOT spec Table 02320-4 – Subgrade Geotextile (Separation). A minimum initial aggregate base lift of 6 inches is required over geotextiles.

8.0 LIMITATIONS OF REPORT

We have prepared this report for the exclusive use of Westech and their authorized parties for the project specifically identified in this report only. The report should be provided in its entirety to prospective contractors for bidding and estimating purposes; however, the conclusions and interpretations presented should not be construed as a warranty of the subsurface conditions. Experience has shown that soil and groundwater conditions can vary significantly over small distances. Inconsistent conditions can occur between explorations that may not be detected by a geotechnical study. If, during future site operations, subsurface conditions are encountered which vary appreciably from those described herein, Central should be notified for review of the recommendations of this report, and revision of such if necessary.

This report is not intended for use by others, and the information contained herein is not applicable to other sites. No other party may rely on the product of our services unless we agree in advance and in writing to such reliance.

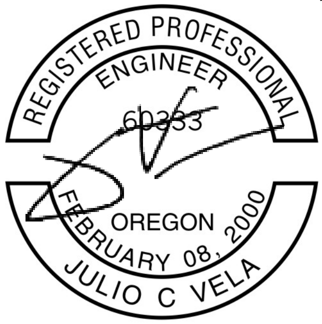
We recommend that Central be retained to review the plans and specifications and verify that our recommendations have been interpreted and implemented as intended. Sufficient geotechnical monitoring, testing and consultation should be provided during construction to confirm that the conditions encountered are consistent with those indicated by explorations. Recommendations for design changes will be provided should conditions revealed during construction differ from those anticipated.

Within the limitations of scope, schedule and budget, the analysis, conclusions, and recommendations presented in this report were prepared in accordance with generally accepted professional principles and practices in the fields of geotechnical engineering and engineering geology in this area at the time the report was prepared.

9.0 SIGNATURES

Thank you very much for the opportunity to work with you. If you feel obliged, we welcome referrals from our previous clients and would enjoy the opportunity to work with others in your professional and personal networks.

Central Geotechnical Services, LLC



EXPIRES: 6/30/25

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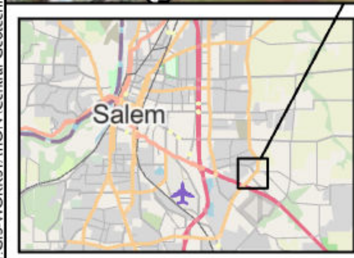

N

0 710 1,420 Feet

Esri Community Maps Contributors, City of Salem, Oregon, Oregon Metro, Oregon State Parks, State of Oregon GEO, Esri, TomTom, Garmin, SafeGraph, GeoTechnologies, Inc, METI/NASA, USGS, Bureau of Land Management, EPA, NPS, US Census Bureau, USDA, USFWS, Map data © OpenStreetMap contributors, Microsoft, Facebook, Inc. and its affiliates, Esri Community Maps contributors, Map layer by Esri, Maxar

Westech-19-01 - Macleay and Cordon Development	
Project Vicinity	
CENTRAL Geotechnical Services, LLC	Figure-1

Default Folder: C:\Users\cgsuser.GIS-WORKSTATION\Central Geotechnical Services\CGS - Projects\R-Z\Salem\Salem-13\Salem-13-01\Analysis\GIS\Salem-13-15-01

Legend

Exploration Designation and Approximate Location

- Pavement Coring
- Infiltration Testing
- Borings

N

0 130 260 Feet

Westech-19-01 - Macleay and Cordon Development	
Site Plan	
CENTRAL Geotechnical Services, LLC	Figure-2

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APPENDIX A: FIELD EXPLORATIONS AND LABORATORY TESTING

APPENDIX A
FIELD EXPLORATIONS AND LABORATORY TESTING

FIELD EXPLORATIONS

Soil and groundwater conditions at the proposed project were explored on September 16 and 17, 2024, by completing eight drilled borings (B-1 through B-6, INF-1 and INF-2) at the approximate locations shown on the Site Plan, Figure 2. The exploratory borings were extended to final depths of between 6.5- to 31.5-feet below ground surface using a 4-3/8-inch diameter mud rotary drilling technique and 8-1/4-inch hollow stem auger drilling technique. The borings were completed with a drill rig owned and operated by Western States Soil Conservation, Inc. (WSSC).

The explorations were continuously monitored by a qualified staff from our office who maintained detailed logs of subsurface explorations, visually classified the soil encountered and obtained representative soil samples from the explorations. Samples were collected from the borings using 1½-inch-inside-diameter, split-spoon SPT samplers in general accordance with ASTM D1586. The samplers were driven into the soil with a 140-pound automative trip hammer free-falling 30 inches. The samplers were driven a total distance of 18 inches. The number of blows required to drive the sampler the final 12 inches is recorded on the exploration logs, unless otherwise noted. The average efficiency of the automatic SPT hammer used by WSSC was 81.4 percent. The calibration testing results are presented at the end of this appendix.

Recovered soil samples from exploratory borings were visually classified in the field in general accordance with ASTM D 2488 and the classification chart listed in Key to Exploration Logs. Logs of the borings are presented in this Appendix. The logs are based on interpretation of the field and laboratory data and indicate the depth at which subsurface materials, or their characteristics change, although these changes may actually be gradual.

LABORATORY TESTING

Soil samples obtained from the explorations were visually classified in the field and in our laboratory using the USCS and ASTM classification methods. ASTM Test Method D2488 was used to visually classify the soil samples, while ASTM D2487 was used to classify the soils based on laboratory tests results.

Moisture Content

Moisture content determinations were performed in general accordance with ASTM D2216. Results of the moisture content testing are presented on the appropriate exploration logs at the respective sample depths.

Particle-size Analysis

Selected samples were “washed” through the U.S. No. 200 mesh sieve to estimate the relative percentages of coarse- and fine-grained particles in the soil. The percent passing value represents the percentage by weight of the sample finer than the U.S. No. 200 sieve. These tests were conducted to verify field descriptions and to estimate the fines content for analysis purposes. The tests were conducted in accordance with ASTM D1140 and/or C117, and the results are shown on the exploration logs in Appendix A at the respective sample depths.

Atterberg Limits Test

Atterberg limits (plastic and liquid limits) testing was performed on select soil samples in general accordance with ASTM D4318. The plastic limit is defined as the moisture content where the soil becomes brittle. The liquid limit is defined as the moisture content where the soil begins to act similar to a liquid. The plasticity index is the difference between the liquid and plastic limits. The test results are presented on the appropriate exploration log at the respective sample depth.

CENTRAL GEOTECHNICAL SERVICES

Exploration and Soil Classification Key

rev. 051624

Relative Density - Coarse-Grained Soil

Term	SPT (140-lb Hammer)*	D&M Sampler (140-lb Hammer)*	D&M Sampler (300-lb Hammer)*
Very-loose	0-4	0-11	0-4
Loose	4-10	11-26	4-10
Medium-dense	10-30	26-74	10-30
Dense	30-50	74-120	30-47
Very-dense	>50	>120	>47

Consistency - Fine-Grained Soil

Term	SPT (140-lb Hammer)*	Sampler (140-lb Hammer)*	Sampler (300-lb Hammer)*	Pocket Pen (tsf)	Torvane (tsf)
Very-soft	0-2	0-3	0-2	<0.25	<0.13
Soft	2-4	3-6	2-5	0.25-0.5	0.13-0.25
Medium-stiff	4-8	6-12	5-9	0.5-1	0.25-0.5
Stiff	8-15	12-25	9-19	1.0-2.0	0.5-1.0
Very-stiff	15-30	25-65	19-31	2.0-4.0	1.0-2.0
Hard	>30	>65	>31	>4.0	>2.0

SPT N-value correlation based off ASTM D1586

Unified Soil Classification System (USCS)

USCS Symbols	Graph	Typical Descriptions
GP		Poorly graded GRAVEL, <5% fines
GP-GM/GP-GC		Poorly graded GRAVEL w/ silt/clay, 5 to 12% fines
GM		silty GRAVEL, over 12% fines
GC		clayey GRAVEL, over 12% fines
GW		well graded GRAVEL, <5% fines
SP		poorly graded SAND, <5% fines
SP-SM/SP-SC		poorly graded SAND w/ silt/clay, 5 to 12% fines
SM		silty SAND, over 12% fines
SC		clayey SAND, over 12% fines
SW		well graded SAND, <5% fines
ML		SILT, low plasticity
MH		SILT, high plasticity
CL		CLAY, low plasticity
CH		CLAY, high plasticity
OL		ORGANIC SILT, low plasticity
OH		ORGANIC CLAY, medium to high plasticity
PT		PEAT

ADDITIONAL CONSTITUENTS

Silt/Clay in:			Sand/Gravel in:		
Percent*	Fine-Grained	Coarse-Grained	Percent*	Fine-Grained	Coarse-Grained
<5	trace	trace	<5	trace	trace
5-12	minor	with	5-15	minor	minor
>12	some	silty/clayey	15-30	with	with
			>30	sandy/gravelly	with

GEOTECHNICAL TESTING EXPLANATIONS

ATT	Atterberg Limits
CBR	California Bearing Ratio
CON	Consolidation
DD	Dry Density
DS	Direct Shear
HYD	Hydrometer Gradation
LL	Liquid Limit
PL	Plastic Limit
PI	Plasticity Index
MC	Moisture Content
MD	Moisture-Density
NP	Non-Plastic
OC	Organic Content
P	Pushed Sample
PP	Pocket Penetrometer
Passing No.200	Percent Passing U.S. Std. No.200 Sieve
RES	Resilient Modulus
SIEV	Sieve Gradation
TOR	Torvane
UC	Unconfined Compressive Strength
VS	Vane Shear

CONTACT LINES

———	Distinct contact between soil strata (approximate location)
– – – –	Approximate contact between soil strata

WATER LEVELS

▽	Water Level at Time of Drilling, or as labeled
▼	Water Level at End of Drilling, or as labeled
▼	Static Water Level, or as labeled

Moisture (ASTM D2488)

Dry	Very low moisture, dry to touch
Moist	Damp, without visible moisture
Wet	Visible free water, usually saturated

ADDITIONAL MATERIALS

AC		ASPHALT CONCRETE
CC		CEMENT CONCRETE
CR		CRUSHED ROCK
SOD		SOD/FOREST DUFF
FILL		FILL

Sampler Descriptions

SYMBOL	SAMPLER DESCRIPTIONS	SYMBOL	SAMPLER DESCRIPTIONS
	Location of grab sample (GS)		Location of sample collected using Standard Penetration Test with recovery (SS)
	No Recovery		Location of sample collected using Shelby tube/Geoprobe sample with recovery (ST)
	Location of rock coring interval (RC)		Location of sample collected using Dames & Moore sampler or pushed with recovery (D&M)

Key to Exploration Logs and Soil Classification

Figure A-1

CENTRAL Geotechnical Services
Central Engineering Services
7662 SW Mohawk Street
Tualatin, OR 97062
Telephone: (503) 616-9419

Project No: Westech-19-01

BORING LOG B-1

Client: Westech Engineering, Inc
Project: Macleay and Cordon
Location: Macleay Road SE & Cordon Road SE, Salem, OR

Date Started: 9/16/24
Date Completed: 9/16/24

Approximate Ground Elevation:
Groundwater first observed: ---
Groundwater at end of drilling: ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY (in.)	MOISTURE (%)	N-Value	LAB RESULTS/ REMARKS
0 – 0.5		ASPHALT CONCRETE (5.5 inches) Pavement fabric at 1.5 inches					
0.5 – 1.0		Dense, gray to dark gray, GRAVEL with silt and sand (GP-GM), moist, gravel is subangular (6.5 inches of AGGREGATE BASE)					
1.0 – 2.0		Medium-stiff, dark gray, SILT (ML), trace gravel and organics (decomposed rootlets), moist, gravel is subangular, organic odor (FILL)					
2.0 – 6.5		Stiff, gray to brown, SILT (ML), moist, moderate plasticity					
2.5 – 4.0			SS S-1	14	35	10	
5.0 – 6.5			SS S-2	18		7	

Boring completed at 6.5 feet bgs.
Groundwater not observed.
Hammer efficiency factor is 84.1%

Operator: Western States Soil Conservation Inc.
Equipment: CME 75 HT Truck Rig **Rig Number:** Truck #5
Drilling Method: 4 7/8" Mud Rotary

Logged By: Ruslan P.
Checked By: Jessica P.
Approximate Location Coordinates:
Lat: Long:

Remarks: Macleay Road SE, EB Lane

Central Engineering Services
7662 SW Mohawk Street
Tualatin, OR 97062
Telephone: (503) 616-9419

Project No:
Westech-19-01

BORING LOG B-2

Client: Westech Engineering, Inc
Project: Macleay and Cordon
Location: Macleay Road SE & Cordon Road SE, Salem, OR

Date Started: 9/16/24
Date Completed: 9/16/24

Approximate Ground Elevation:
Groundwater first observed: ---
Groundwater at end of drilling: ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY (in.)	MOISTURE (%)	N-Value	LAB RESULTS/ REMARKS
0		ASPHALT CONCRETE (5.5 inches)					
0.5		Pavement fabric at 1.5 inches					
0.7		Dense, gray to dark gray, GRAVEL with silt and sand (GP-GM), moist, gravel is subangular (6.5 inches of AGGREGATE BASE)					
		Medium-stiff, dark gray, SILT (ML), trace gravel and organics (decomposed rootlets), moist, moderate plasticity, gravel is angular, organic odor (FILL)	SS S-1	11		6	
			SS S-2	12		6	
4.3		Medium-stiff, gray to brown, SILT (ML), moist, moderate plasticity					
			SS S-3	18		8	
6.5							

Boring completed at 6.5 feet bgs.
Groundwater not observed.
Hammer efficiency factor is 84.1%

Operator: Western States Soil Conservation Inc.
Equipment: CME 75 HT Truck Rig **Rig Number:** Truck #5
Drilling Method: 4 7/8" Mud Rotary

Logged By: Ruslan P.
Checked By: Jessica P.
Approximate Location Coordinates:
Lat: Long:

Remarks: Macleay Road SE, EB Lane

CENTRAL Geotechnical Services
Central Engineering Services
7662 SW Mohawk Street
Tualatin, OR 97062
Telephone: (503) 616-9419

Project No:
Westech-19-01

BORING LOG B-3

Client: Westech Engineering, Inc
Project: Macleay and Cordon
Location: Macleay Road SE & Cordon Road SE, Salem, OR

Date Started: 9/16/24
Date Completed: 9/16/24

Approximate Ground Elevation:
Groundwater first observed: ---
Groundwater at end of drilling: ---

DEPTH (ft)	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY (in.)	MOISTURE (%)	N-Value	LAB RESULTS/ REMARKS
0–0.5	ASPHALT CONCRETE (6.0 inches) Pavement fabric at 1.5 inches					
0.5–1.3	Dense, gray to dark gray, GRAVEL with silt and sand (GP-GM), moist, gravel is subangular (10 inches of AGGREGATE BASE)					
1.3–6.0	Soft, dark gray, SILT (ML), trace organics (decomposed rootlets), moist, moderate plasticity, organic odor (FILL)	SS S-1	10		4	
		SS S-2	9		3	
		SS S-3	14	35	8	
6.0–6.5	Medium-stiff, gray to brown, SILT (ML), moist, moderate plasticity					

Boring completed at 6.5 feet bgs.
Groundwater not observed.
Hammer efficiency factor is 84.1%

Operator: Western States Soil Conservation Inc.
Equipment: CME 75 HT Truck Rig **Rig Number:** Truck #5
Drilling Method: 4 7/8" Mud Rotary

Logged By: Ruslan P.
Checked By: Jessica P.
Approximate Location Coordinates:
Lat: Long:

Remarks: Macleay Road SE, EB Lane

CENTRAL Geotechnical Services
Central Engineering Services
7662 SW Mohawk Street
Tualatin, OR 97062
Telephone: (503) 616-9419

Project No:
Westech-19-01

BORING LOG B-4

Client: Westech Engineering, Inc
Project: Macleay and Cordon
Location: Macleay Road SE & Cordon Road SE, Salem, OR

Date Started: 9/16/24
Date Completed: 9/16/24

Approximate Ground Elevation:
Groundwater first observed: ---
Groundwater at end of drilling: ---

DEPTH (ft)	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY (in.)	MOISTURE (%)	N-Value	LAB RESULTS/ REMARKS
0–1.0	Very-stiff, brown to dark-brown, SILT (ML), trace fine roots, dry to moist (4-inch-thick root zone)	SS S-1	11		20	
1.0	Stiff, brown, SILT (ML), trace sand, moist	SS S-2	18		12	
		SS S-3	18		10	
	Grades to soft, with sand at 7.5 feet bgs	SS S-4	18		4	
	Grades to very-soft, no sand at 10.0 feet bgs	SS S-5	18	40	0	Passing No. 200: 98.9%
	Grades to medium-stiff, blue-gray at 15.0 feet bgs	SS S-6	18	38	5	
	Grades to soft at 20.0 feet bgs	SS S-7	18		5	
25.0–26.5	Very-dense, dark gray, GRAVEL with sand and silt (GP-GM), moist to wet, sand is medium, gravel is subrounded	SS S-8	18	21	44/50 for 5"	

Boring completed at 26.5 feet bgs.
Groundwater not observed due to the drilling method.
Hammer efficiency factor is 84.1%

Operator: Western States Soil Conservation Inc.
Equipment: CME 75 HT Truck Rig **Rig Number:** Truck #5
Drilling Method: 4 7/8" Mud Rotary

Logged By: Ruslan P.
Checked By: Jessica P.

Approximate Location Coordinates:
Lat: Long:

Remarks:

Central Engineering Services
7662 SW Mohawk Street
Tualatin, OR 97062
Telephone: (503) 616-9419

Project No:
Westech-19-01

BORING LOG B-5

Client: Westech Engineering, Inc
Project: Macleay and Cordon
Location: Macleay Road SE & Cordon Road SE, Salem, OR

Date Started: 9/17/24
Date Completed: 9/17/24

Approximate Ground Elevation:
Groundwater first observed: ---
Groundwater at end of drilling: ---

DEPTH (ft)	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY (in.)	MOISTURE (%)	N-Value	LAB RESULTS/ REMARKS
0–1.0	Very-stiff, brown to dark-brown, SILT (ML), trace fine roots, sand and debris, dry (4-inch-thick root zone)(FILL)	SS S-1	12		14	
1.0	Stiff, brown, SILT (ML), moist	SS S-2	16		9	
	Grades to soft, trace sand at 5.0 feet bgs	SS S-3	18	37	4	Passing No. 200: 99.5%
	Grades to gray at 7.5 feet bgs	SS S-4	18		2	
	Grades to very-soft at 10.0 feet bgs	SS S-5	18		1	
	Grades to medium-stiff, blue-gray at 15.0 feet bgs	SS S-6	17		7	
	Grades to soft at 20.0 feet bgs	SS S-7	18		5	
25.5	Very-dense, dark gray, GRAVEL with sand and silt (GP-GM), moist to wet, sand is medium, gravel is subrounded	SS S-8	18	26	49/50 for 5"	
31.5		SS S-9	9		46/50 for 6"	

Boring completed at 31.5 feet bgs.
Groundwater not observed due to the drilling method.
Hammer efficiency factor is 84.1%

Operator: Western States Soil Conservation Inc.
Equipment: CME 75 HT Truck Rig **Rig Number:** Truck #5
Drilling Method: 4 7/8" Mud Rotary

Logged By: Ruslan P.
Checked By: Jessica P.

Approximate Location Coordinates:
Lat: Long:

Remarks:

Central Engineering Services
7662 SW Mohawk Street
Tualatin, OR 97062
Telephone: (503) 616-9419

Project No:
Westech-19-01

BORING LOG B-6

Client: Westech Engineering, Inc
Project: Macleay and Cordon
Location: Macleay Road SE & Cordon Road SE, Salem, OR

Date Started: 9/16/24
Date Completed: 9/16/24

Approximate Ground Elevation:
Groundwater first observed: ---
Groundwater at end of drilling: ---

DEPTH (ft)	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY (in.)	MOISTURE (%)	N-Value	LAB RESULTS/ REMARKS
0–1.0	Stiff, brown to dark brown, SILT (ML), trace sand and fine roots, dry (4-inch-thick root zone)					
1.0	Medium-stiff, brown SILT (ML), trace sand, dry					
3–4.5		SS 1	12		6	
5.5–7		SS 2	12	31	7	Passing No. 200: 98.5%
7.0	Grades to soft at 7.0 feet bgs					
8–9.5		SS 3	0		4	
10.0	Grades to very-soft at 10.0 feet bgs					
10.5–12		SS 4	18	38	0	Passing No. 200: 98.9%
15.0	Grades to wet at 15.0 feet bgs					
15.5–17		SS 5	18		1	
17–19		SS 6	24	38		
19–20.5		SS 7	18		0	
20.5–22		SS 8	18		0	
24.5	Very-dense, dark gray, GRAVEL with sand and silt (GP-GM), moist to wet, sand is medium, gravel is subrounded					
25.5–27		SS 9	16	13	74	Passing No. 200: 8.7%
30–31		SS 10	0		50	
31.0						

Terminated at 31.0 feet bgs.
Groundwater not observed due to the drilling method.
Hammer efficiency factor is 84.1%

Operator: Western States Soil Conservation Inc.
Equipment: CME 75 HT Truck Rig **Rig Number:** Truck #5
Drilling Method: 4 7/8" Mud Rotary

Logged By: Ruslan P.
Checked By: Jessica P.

Approximate Location Coordinates:
Lat: Long:

Remarks:

Central Engineering Services
7662 SW Mohawk Street
Tualatin, OR 97062
Telephone: (503) 616-9419

Project No:
Westech-19-01

INFILTRATION LOG INF-1

Client: Westech Engineering, Inc	**Date Started:** 9/17/24	**Approximate Ground Elevation:**	
Project: Macleay and Cordon		**Groundwater first observed:**	---
Location: Macleay Road SE & Cordon Road SE, Salem, OR	**Date Completed:** 9/17/24	**Groundwater at end of drilling:**	---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY (in.)	MOISTURE (%)	N-Value	LAB RESULTS/ REMARKS
0		Stiff, brown to dark brown, SILT (ML), trace sand and fine organics, dry (4-inch-thick root zone)					
1	1.0	Stiff, gray brown, SILT (ML), trace sand, moist					
2							
3			SS 1	12.5		9	
4							
5							
6			SS 2	18		26	
7							
8		Grades to brown with black staining at 7.5 feet bgs	SS 3	18		26	
9							
10							
11			SS 4	4	38	15	Passing No. 200: 95.1%
12							
13							
14		Grades to gray at 13.0 feet bgs					
15							
16	16.5		SS 5	18		26	

Boring completed at 16.5 feet bgs.
Groundwater not observed.

Infiltration test completed at 10 feet bgs.

Hammer efficiency factor is 84.1%

Operator: Western States Soil Conservation Inc.
Equipment: CME 75 HT Truck Rig **Rig Number:** Truck #5
Drilling Method: 6" HSA

Logged By: Ruslan P.
Checked By: Jessica P.

Approximate Location Coordinates:
Lat: Long:

Remarks:

CENTRAL Geotechnical Services
Central Engineering Services
7662 SW Mohawk Street
Tualatin, OR 97062
Telephone: (503) 616-9419

Project No: Westech-19-01

INFILTRATION LOG INF-2

Client: Westech Engineering, Inc
Project: Macleay and Cordon
Location: Macleay Road SE & Cordon Road SE, Salem, OR

Date Started: 9/17/24
Date Completed: 9/17/24

Approximate Ground Elevation:
Groundwater first observed: ---
Groundwater at end of drilling: ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY (in.)	MOISTURE (%)	N-Value	LAB RESULTS/ REMARKS
0–1.0		Stiff, brown to dark brown, SILT (ML) trace sand and fine organics (4-inch-thick root zone)	SS 1	12		14	
1.0–16.5		Stiff, brown gray, SILT (ML), moist					
2.5–4			SS 2	18		10	
5		Grades to dark brown with black staining, minor sand at 5.0 feet bgs	SS 3	18	36	9	Passing No. 200: 89.6%
7.5		Grades to gray-brown with orange mottles, with sand at 10.0 feet bgs	SS 4	18		4	
10			SS 5	18		2	
15		Grades to blue-gray at 15.0 feet bgs	SS 6	18		4	

Boring completed at 16.5 feet bgs.
Groundwater not observed.

Infiltration test completed at 5 feet bgs.

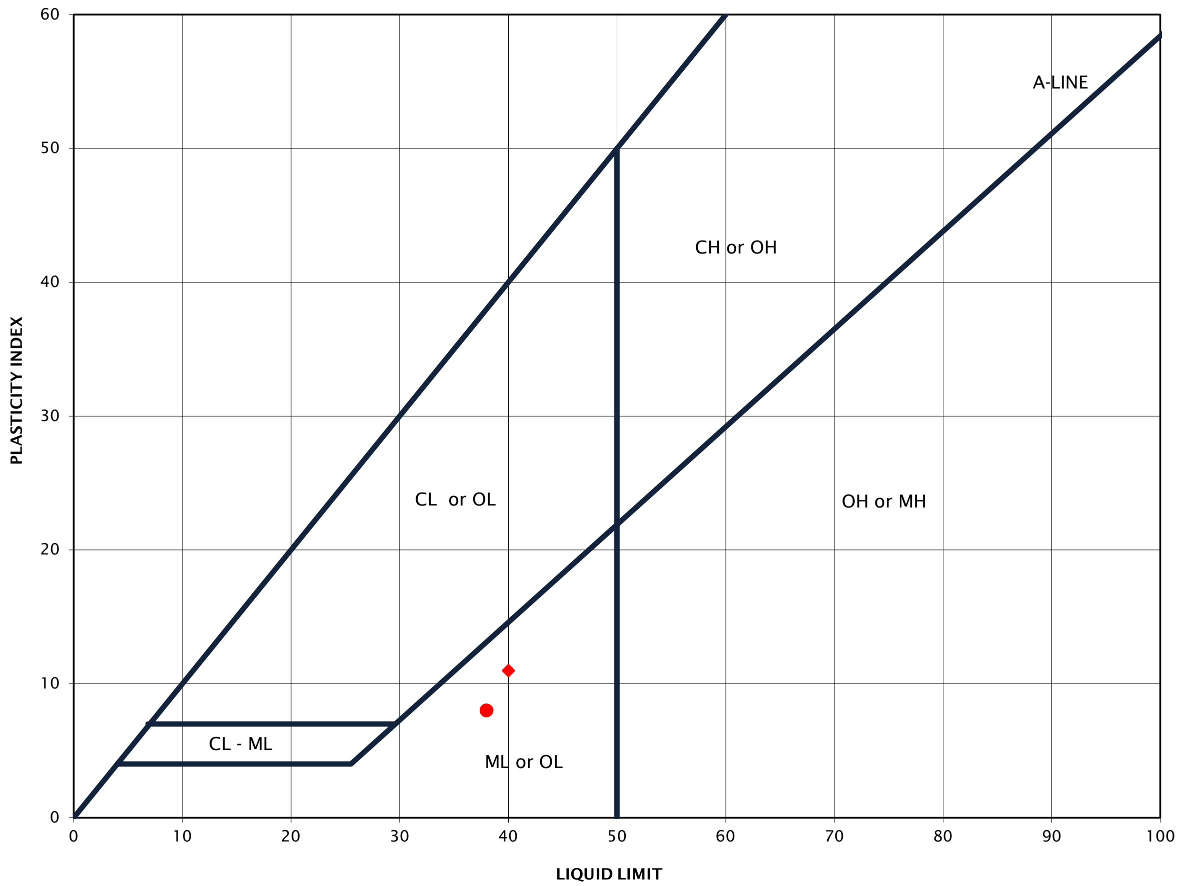
Hammer efficiency factor is 84.1%

Operator: Western States Soil Conservation Inc.
Equipment: CME 75 HT Truck Rig **Rig Number:** Truck #5
Drilling Method: 6" HSA

Logged By: Ruslan P.
Checked By: Jessica P.

Approximate Location Coordinates:
Lat: Long:

Remarks:



Key	Exploration Number	Sample Depth (Feet)	Moisture Content (Percent)	Liquid Limit	Plastic Limit	Plasticity Index
◆	B-5	5	36.6	40	29	11
●	B-6	10	37.5	38	30	8

CENTRAL GEOTECHNICAL SERVICES	Atterberg Limits Test Results	
	Westech-19-01	September 2024
	Macleay and Cordon	

Summary of SPT Test Results

Project: RIG5 TRUCK CME 75 SN373617, Test Date: 9/16/2023

FMX: Maximum Force
VMX: Maximum Velocity
BPM: Blows/Minute
EFV: Maximum Energy
ETR: Energy Transfer Ratio - Rated

Instr. Length ft	Blows Applied /6"	**N Value**	**N60 Value**	Average FMX kips	Average VMX ft/s	Average BPM bpm	Average EFV ft-lb	Average ETR %
35.00	**9-16-13**	**29**	**40**	47	16.5	52.0	301	86.1
38.00	**9-17-13**	**30**	**42**	48	15.0	51.7	288	82.3
40.00	**6-13-17**	**30**	**42**	48	14.9	51.7	299	85.4
43.00	**10-19-17**	**36**	**50**	52	14.7	51.6	300	85.8
45.00	**9-19-22**	**41**	**57**	47	14.1	51.6	285	81.4
			Overall Average Values:	49	15.0	51.7	294	84.1
			Standard Deviation:	2	0.9	0.1	9	2.7
			Overall Maximum Value:	55	17.6	52.1	314	89.8
			Overall Minimum Value:	44	13.6	51.5	275	78.4