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Revised December 16, 2021

Jessica Woodruff
Community Development Partners
126 NE Alberta Street, Suite 202
Portland, Oregon 97211

Re: Geotechnical Investigation
Gateway Salem Development
Battle Creek Road SE
Salem, Oregon 97302

CGS Project No. 20-023

Ms. Woodruff,

Central Geotechnical Services, LLC (Central Geotech) is pleased to submit this Geotechnical Investigation Report for the proposed Gateway Salem Development at the Miller Site - Parcel 2 and 3 located in Salem, Oregon. The report was prepared in accordance with our Professional Services Agreement dated April 16, 2021.

The scope of our work included:

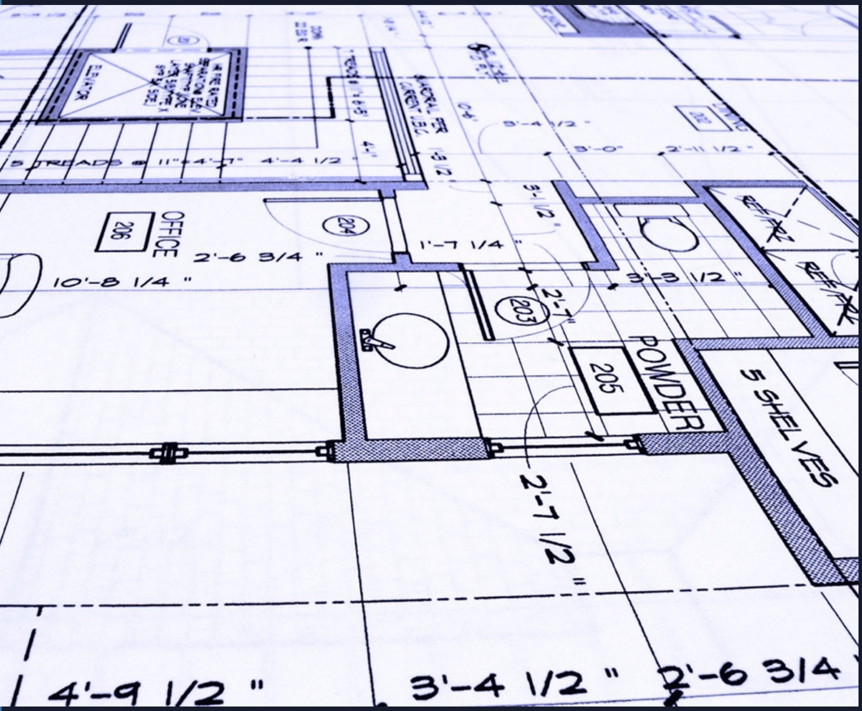
- ➢ Review of published geologic mapping
- ➢ Site reconnaissance
- ➢ Subsurface exploration consisting of fourteen exploratory test pits and three infiltration tests.
- ➢ Preparation of a Geotechnical Investigation Report presenting our conclusions and recommendations for geotechnical design and construction with specific regard to:
    - o Geologic hazards
    - o Allowable soil bearing for foundations
    - o Settlement estimates for foundations
    - o Retaining walls
    - o Storm water management
    - o Fill compaction

Thank you very much for the opportunity to work with you on this project. Please feel free to call our office with questions about this report.

Respectfully,

Central Geotechnical Services, LLC

Jose R. Serrano, P.E.
Associate Engineer



Geotechnical Investigation:

Gateway Salem Development
Miller Site – Parcel 2 & 3
Battle Creek Road
Salem, Oregon

Central Geotech Project No. 21-023

Prepared For:

Jessica Woodruff
Community Development Partners
126 NE Alberta Street, Suite 202
Portland, Oregon

Revised December 16, 2021

Submitted by:

CENTRAL
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1.0 INTRODUCTION AND PURPOSE OF REPORT

The purpose of this Geotechnical Investigation Report is to engage with the owner/developer and provide technical insight and analysis for the project, based on various public data, local findings onsite, and experience.

After receiving direction from Mr. Randy Boehm of Urban Resources, Inc., Central Geotechnical Services (CGS) was requested to provide a Geotechnical Investigation, along with general recommendations for the design and construction of the proposed multi-family residential development in Salem, Oregon. Mr. Boehm and members of the project design team provided preliminary construction concept documents for us to review and geotechnical scope requirements for the project.

Our conclusions and recommendations cover topics such as investigative soils data, allowable soil bearing pressure, lateral pressures, compaction requirements, design and alteration of existing foundations, foundation placement, pavement, and seismic considerations.

This report is intended to facilitate the preliminary focus of future development and initiate the requirements for the design and permitting of the proposed development project.

1.1 Project Description

Mr. Boehm provided us with grading and drainage, and street plans prepared by Western Engineering Inc., dated March 2021. Mr. Boehm also provided us with a preliminary site plan prepared by Scott Edwards Architecture LLP, dated February 25, 2021.

Based on phone discussions and review of documentation sent by your office, we understand that Community Development Partners (CDP) intends to build ten multi-family residential buildings with outdoor space at the site. Associated improvements will include pavement for parking and driveways, new streets, underground utilities and storm water management facilities.

The project is in the preliminary planning stages such that only tentative building plans are available at this time. We understand that the buildings will be 1- to 4-stories tall with wood frame construction. Structural loading information for the buildings is not available at this time. We expect that the design and construction of the development will be governed by the provisions of the 2019 Oregon Structural Specialty Code (OSSC).

The preliminary grading plan shows excavation cuts of up to 3 feet deep and fills of up to 3 feet thick for construction of roadway embankments. We presume that the buildings will be constructed on excavated building pads and that thin fills will be placed in localized areas to construct private driveways and parking areas.

Underground utilities will be constructed in street right-of-ways. Three water quality facilities that will receive storm water runoff are planned adjacent to open space areas.

2.0 INVESTIGATION SUMMARY

2.1 Site Location and Surface Conditions

The proposed development site is located off Battle Creek Road SE, about 1,100 feet southwest of Boone Road SE, in Salem, Oregon. The site is a 15.54-acre, polygon-shaped property that is made up of two contiguous lots identified as Marion County Tax Lots 083W140000118 and 083W140000300. A vicinity map of the site is shown in Figure 2-1, below.

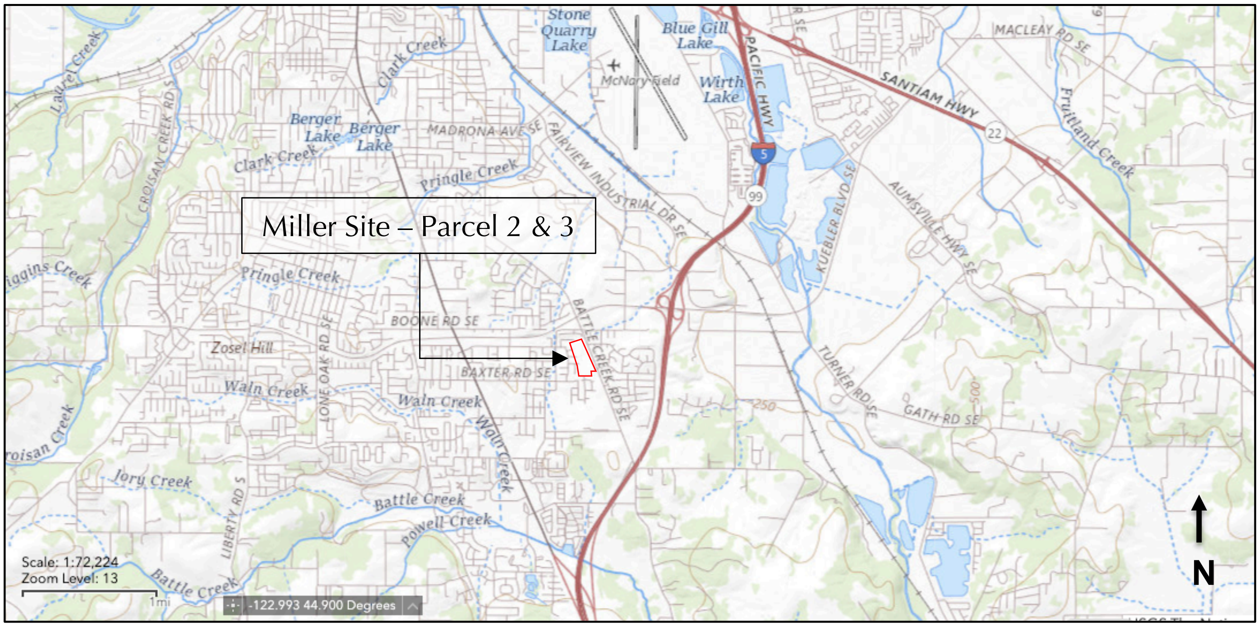

Figure 2-1: Vicinity map of project site (Source: USGS National Map)

The site is located on a broad drainage with gentle topography that inclines to the north. Slopes on the site are generally inclined at less than 10% grade. The elevation at the site ranges from about 367 feet above mean sea level at the northeast corner to about 412 feet at the southeast corner. At the time of our exploration, the site was mostly an open grass field.

The general topography in the site vicinity is shown in Figure 2-2, on the next page.

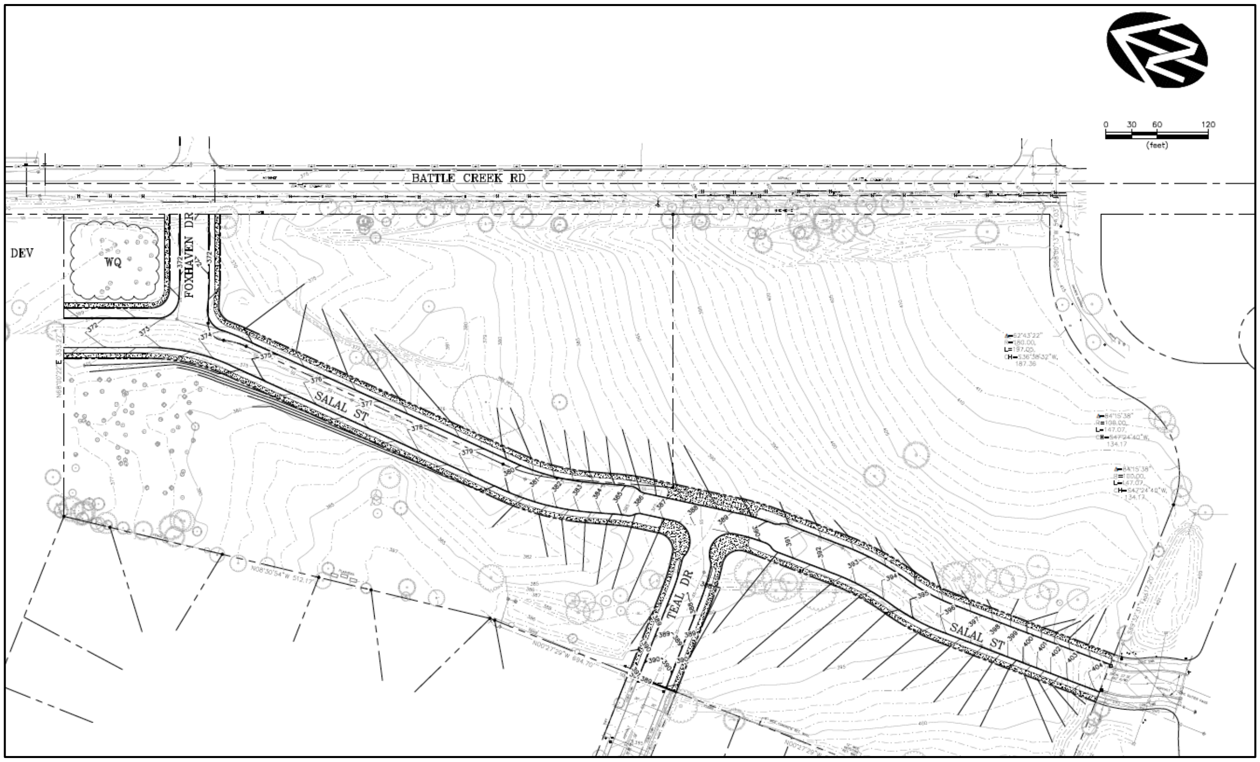

Figure 2-2: Topographic map of CDP Salem Gateway. Contour interval is 1-foot.
(Source: Overall Grading & Drainage Plan by Westech Engineering, Inc., dated March 2021)

2.2 Site Geology

The South Salem area is underlain by a thick and widespread sequence of basalt flows belonging to the Miocene age Columbia River Basalt Group, deposited 6 to 17 million years ago.[1] The basalt is a dense, finely-crystalline rock that is commonly fractured along blocky and columnar joints. The fractures formed as the lava cooled and contracted, and with subsequent tectonic deformation. Tectonic forces fractured, folded, uplifted and faulted the basalt to form broad hills with deeply-incised gullies.

Individual basalt flows range from 15 to 150 feet thick and are sometimes separated by thin interflow zones of sedimentary deposits or residual soil. The total thickness of the Columbia River Basalt Group in the site vicinity exceeds several hundred feet. Basalt at the ground surface is typically decomposed to a clayey silt with a distinctive, red-brown color, known as residual soil or laterite.

In the late Quaternary (80,000 to 10,000 years ago), the Columbia River Basin and Willamette Valley were repeatedly inundated by episodic glacial outburst floods, known as the Missoula Floods. The flood waters scoured the bedrock along river channels, and deposited gravel, silt, and sand up to several

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[1] Walker, G.W., and Duncan, R.A., 1989 Geologic map of the Salem 1° by 2° Quadrangle, western Oregon: U.S. Geologic Survey Miscellaneous Investigations Series Map I-1893, scale 1:250,000.

hundred feet thick at elevations below 400 feet[2]. Strong winds subsequently transported the silt as loess (wind-blown silt) onto slopes in upland areas above 400 feet. The last flooding event occurred at the end of the last glacial period about 9,000 to 10,000 years ago[3].

2.3 Seismic Setting

The Salem area is subject to seismic events stemming from three possible sources: the Cascadia Subduction Zone (CSZ) at the interface between the Juan de Fuca Plate and the North American Plate, intraslab faults within the Juan de Fuca Plate, and crustal faults in the North American Plate.

The CSZ is seismically active. Intraslab events with inland epicenters, such as the 6.8 MW Nisqually earthquake in 2001, have occurred on a frequent basis in the Puget Sound, contributing small to moderate magnitude ground motions in southern Washington. The maximum magnitude for a CSZ interface event is expected to be in the range of moment magnitude (M_W) 9.0 with a nearshore or offshore epicenter located about 45 miles west of the project site.

Inland crustal faults in the North American Plate are considered potentially active. Five moderate magnitude earthquakes attributed to crustal faults have occurred in the Portland-Vancouver-Salem metropolitan Area since 1877 including a 5.2 MW earthquake in 1962. Slip rates for the crustal faults are very low (i.e., less than about 0.2 mm per year) and no documented surface rupture has occurred in the last 10,000 years.

Quaternary age (last 1.6 million years) crustal faults inventoried in the USGS National Fault and Fold Database that lie within 10 miles of the is the Salem-Eola Hills Fault about 4.8 miles to the southwest, respectively, the Waldo and Turner and Mill Creek faults about 1.4 and 3.8 miles to the southeast.

The contribution of potential earthquake-induced ground motion from known sources, including the faults described above are provided by the seismic design parameters for the project site presented in the recommendations section of this report.

2.4 Geological Hazard Review

We reviewed comprehensive landslide inventory mapping of Oregon by the Oregon Department of Geology and Mineral Industries (DOGAMI) compiled from regional geologic mapping, LiDAR imagery, and other sources[4]. LiDAR imagery provides high-resolution digital elevations of the ground surface, revealing potential landslide features. Features identified from LiDAR imagery are validated with site reconnaissance and knowledge of the site geology.

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[2] Madin, I., 1990, Earthquake-Hazard Geology Maps of the Portland Metropolitan Area, Oregon: Oregon Department of Geology and Mineral Industries Open File Report O-90-02, map scale 1:24,000.

[3] Waitt, R. B. Jr., 1985, Case for Periodic Colossal Jokulhlaups from Pleistocene Lake Missoula; Geological Society of America Bulletin, v. 96, no. 10, p. 1271-1286.

[4] Oregon Department of Geology and Mineral Industries, 2014, Statewide Landslide Information Database for Oregon (SLIDO 4.2): DOGAMI GIS website, updated October 30, 2020, map scale 1:9,028.

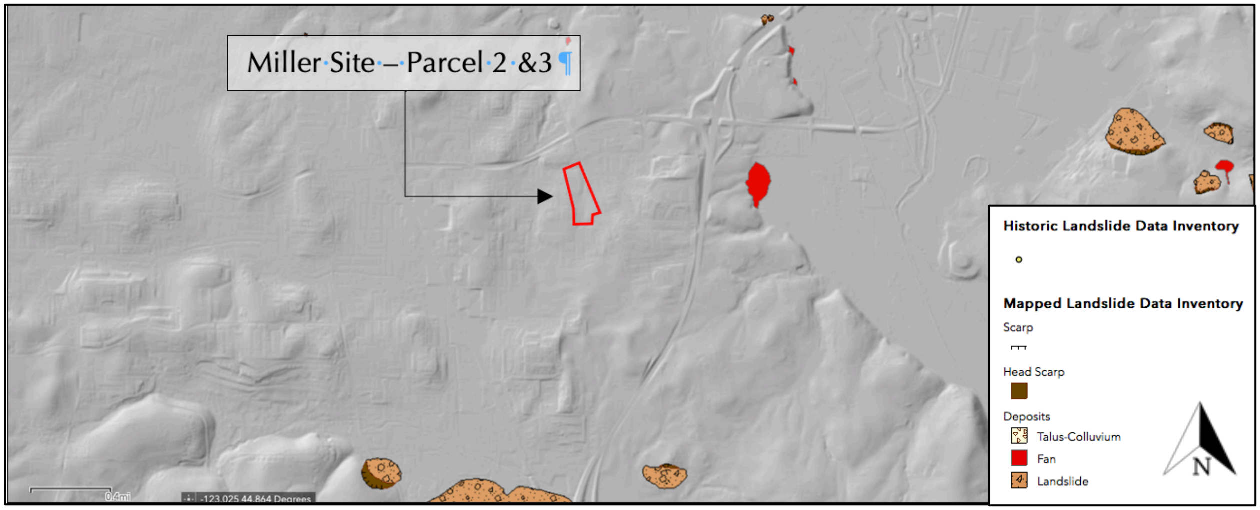

Figure 2-3: Landslide Inventory Map on LiDAR Bare Earth Imagery. Approximate Site Boundary Shown as Red Line. (Source: DOGAMI SLIDO 4.2)

The DOGAMI landslide inventory shows no mapped landslides on or in the vicinity of the Miller Site, and DOGAMI designates the site as a "low landslide susceptibility" area. The Oregon HazVu GIS database does not identify the property as having any Earthquake Liquefaction Hazard. The property does not appear to have any additional mapped geologic hazards.

2.5 Subsurface Exploration

We explored subsurface soil conditions at the site in fourteen exploratory test pits excavated to depths of up to 10 feet below the ground surface (bgs) on February 27, 2021. The test pits were completed with a Hitachi 40u, 8,000-pound, tracked-excavator operated by Dan Fischer Excavating of Forest Grove, Oregon. At the completion of logging and sampling, the test pits were loosely backfilled. The approximate locations of the explorations relative to the proposed development area are presented in Figure 2-4.

Summary logs of the test pit explorations are presented in Appendix A.

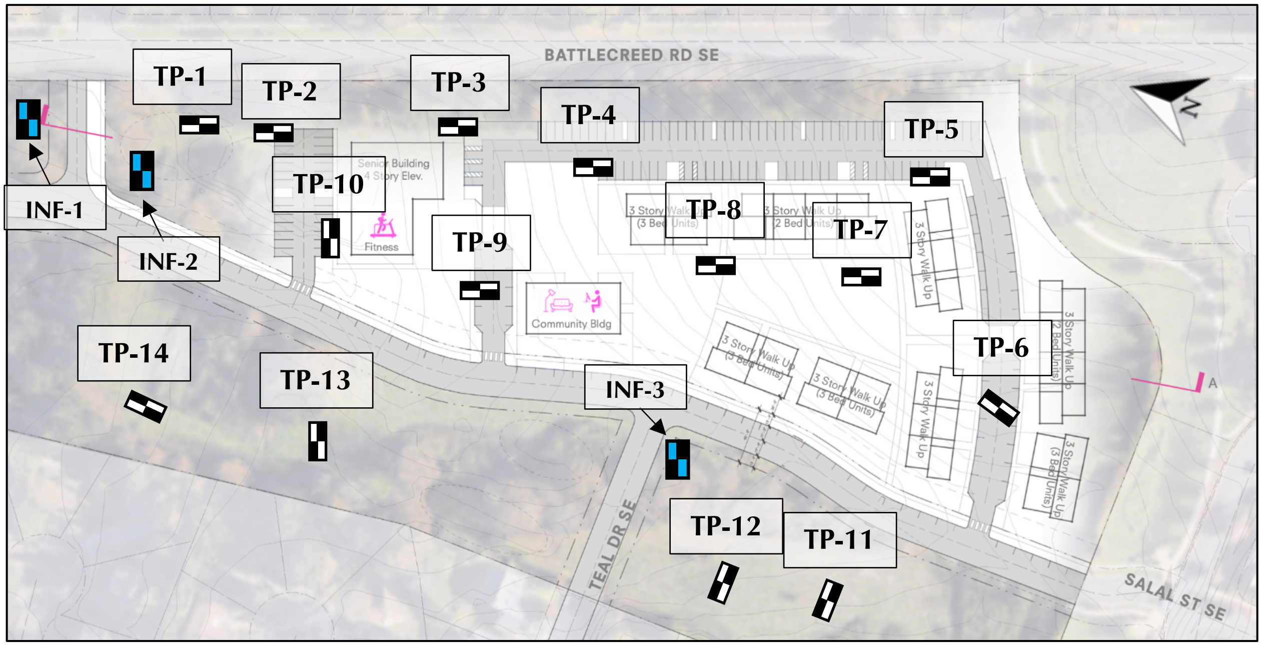

Figure 2-4: Phase 1 – Site Layout with Test Pit Locations. All locations are approximate.
(Source: CDP Salem Presentation, prepared by Scott Edwards Architecture, LLP, dated February 25, 2021, page 5 of 10)

2.6 Subsurface Conditions

We encountered three soil layers on the site; an upper layer of fill and/or disturbed topsoil, a middle layer of clayey silt residual soil, and a lower layer of weathered basalt rock. Each soil unit is described below.

2.6.1 Fill and/or Disturbed Native Topsoil

We encountered a thin layer of fill and/or disturbed native topsoil in all fourteen exploratory test pits that extended from the ground surface to a depth of 1.0 to 3.5 feet. The fill and/or disturbed native topsoil consists of clayey SILT (ML-OL) with variable amounts of sand and gravel with mixed organics. The gravel and sand was generally limited to the upper 18 inches. In general, the clayey SILT is soft to stiff. Pocket penetrometer measurements indicate an unconfined compressive strength of 0.5 to 1.5 tsf, consistent with a soft to stiff consistency. The moisture content of four of the fill and/or disturbed native topsoil samples was generally between 25% and 38%.

Additional unrecognized fill may be present around the existing foundations, subsurface structures, and other existing or abandoned improvements.

2.6.2 Residual Clayey SILT

Beneath the topsoil and/or disturbed native topsoil, we encountered native, residual soil derived from decomposition in-place of the underlying basalt in ten of our fourteen explorations. The residual soil consists of clayey SILT (ML) with trace to fine coarse sand and subangular, gravel to cobble-sized fragments of weathered rock. The amount of rock fragments increases with depth as the layer transitions to basalt at its base. In general, the clayey SILT is stiff to very-hard. Pocket penetrometer measurements indicate an unconfined compressive strength of 1.0 to 5.0 tsf, consistent with a stiff to very-hard consistency. The moisture content of five of the residual soil samples was generally between 28% and 31%.

In explorations, the clayey SILT generally extended to depths of 1.5 to 8 feet.

2.6.3 BASALT

Beneath the residual soil is weathered BASALT belonging to the Columbia River Basalt Group. The BASALT is red-brown to brown, moderately-weathered, fractured and vesicular. The estimated ODOT Rock Hardness Classification of the BASALT is generally Medium-hard (R3) (see ODOT Rock Hardness Classification Chart at end of Appendix A).

Practical refusal on Medium-hard (R3) basalt with an approximate 8,000 pound (GVW) tracked-excavator was encountered in eleven of the fourteen of our test pits at depths of 7.5 to 9 feet bgs. In two test pits, TP-7 and TP-8, the exploration was terminated at 9 feet bgs in soft (R2) weathered Columbia River Basalt. In TP-13, the exploration was terminated at 10 feet bgs in a stiff to very-stiff CLAY (CH). The clay is likely a thin interflow deposit of mudstone. The moisture content of three weathered BASALT samples was generally between 38% and 51%.

2.6.4 Groundwater

We encountered groundwater seepage in seven of our exploratory test pits between 5.5 and 7 feet bgs, which were excavated to a maximum depth of 10 feet in February 2021. The groundwater appeared to be perched on top of the underlying BASALT. These conditions, however, are specific to the locations of our explorations as well as the time of our exploration. Groundwater levels are generally higher (at shallower depths) during the wet season (October through June).

We expect that temporary perched groundwater conditions typically occur near to the ground surface during the wet-weather season in response to heavy rainfall events, due to the presence of low permeability clayey silt soil and shallow basalt.

Central Geotech is not currently engaged to provide observations of groundwater conditions on an ongoing basis. Due to the shallow water table at the site, further investigation of the perched groundwater and monitoring of groundwater levels may be required to determine appropriate shoring design, excavation, and de-watering measures for the project.

2.7 Infiltration Test Results

We conducted infiltration testing at the site on July 22, 2021 in general accordance with the methodology provided in the “City of Salem Department of Public Works Administrative Rules Chapter 109 Division

004 Appendix C Infiltration Testing", dated January 2014, at the approximate locations shown in Figure 2-4, on the previous page, and depths shown in Table 2-1, next page.

Infiltration testing for the proposed stormwater facility was conducted using the "Encased Falling Head Procedure". This procedure utilizes a 6-inch-inside diameter casing or hollow stem auger seated and sealed approximately six inches into native soil. The goal of this field test is to evaluate the vertical infiltration rate through a 6-inch plug of soil without allowing lateral infiltration. This test attempts to mimic lab procedures to determine the infiltration rate for use in the design of infiltration features onsite. Testing was performed in a 6-inch-inside-diameter casing seated approximately six inches below the bottom of the hand auger borings, at the approximate depth of 2 feet below existing grade.

A 24-hour period of pre-saturation was performed prior to the final test runs. A total of two one-hour test runs were performed at each test location. Approximately 6 inches of water was added prior to each test, water levels were measured at periodic intervals from a fixed reference point.

Based on the test results, the recorded infiltration drawdown rates at depths of 2 feet bgs were low with a rate ranging from 1.60 to 2.75 inches per hour. The recorded drawdown rates and test parameters are summarized in Table 2-1.

Table 2-1 - Infiltration Test Parameters and Summary of Test Results

Test Number	Soil Type	Test Depth (feet)	Pressure Head (inches)	Infiltration Drawdown Rate (inches/hour)
INF-1	Clayey SILT	2.0	6	2.75
INF-2	Clayey SILT	2.0	6	2.00
INF-3	Clayey SILT	2.0	6	1.50

3.0 CONCLUSIONS AND RECOMMENDATIONS

3.1 General

Based on the results of our geotechnical investigation, we consider the site suitable for multi-family residential development as proposed. Buildings and associated improvements may be supported on shallow foundations designed and constructed in accordance with our recommendations and applicable building codes. We expect a minimum excavation depth of 2 to 3.5 feet will be necessary to remove existing topsoil/fill and reach native subgrade that is suitable for footing support. In localized areas, additional depth of excavation may be required to remove unsatisfactory soils or existing uncontrolled fills.

The primary geotechnical concerns for the project are the presence of low permeability soil and shallow basalt bedrock that will pose difficult excavating conditions for underground structures. Excavations deeper than 7 feet will likely encounter Medium-hard (R3) BASALT. These conditions are shared by the majority of developments in the area and can be mitigated with proper design and construction.

The following sections present our conclusions and recommendations for project design and construction.

3.2 Landslide Hazard

Based on the results of our geotechnical investigation, there are no serious slope stability concerns for the proposed development. Slopes within the proposed development area are smooth and uniform in topography, consistent with stable slope conditions, and we observed no landform evidence of prior slope movement or landsliding in the vicinity of the proposed development. The proposed development is underlain by native stiff to very-hard clayey SILT and medium-hard (R3) BASALT. These earth materials are generally resistant to instability at slope inclinations of 50% grade or less, under well-drained conditions.

In our opinion, the proposed development will not pose any adverse effects on slope stability at the site or on adjacent properties, provided that the site is developed in accordance with our recommendations. No further evaluation of landslide hazard is considered necessary for conformance with SRC Chapter 810.

3.3 Site Preparation

The heavily rooted topsoil zone should be stripped and removed from the site in all proposed building and pavement areas and for a minimum 2-foot margin around such areas. Based on our explorations, the minimum depth of stripping will be approximately 18 inches. Greater stripping depths will be required to remove tree stumps or isolated zones of loose or organic soil. Stripped material should be transported off-site for disposal or stockpiled for use in landscaped areas.

All brush, trees, and shrubs should be removed in building and paved areas to the depth of roots greater than 1/2-inch in diameter. Depending on the methods used to remove the root balls, considerable disturbance and loosening of the subgrade could occur during site grubbing. Disturbed soil is to be removed to expose firm subgrade. The resulting excavations should be backfilled with engineered structural fill and compacted as described in this report, and evaluated by our office during construction phases.

After stripping and the required site cuts have been completed, we recommend the areas be observed by a member of our geotechnical staff who will evaluate the subgrade by probing or other applicable means. If soft areas are identified, the material should be excavated and replaced with compacted engineered structural fill as described in this report.

It is possible that unrecognized areas of undocumented fill may be encountered on the site during construction. It is recommended that all uncontrolled fill soils be removed completely in preparation for foundations or other construction and be replaced with engineered structural fill in accordance with *Section 3.8 Engineered Structural Fill*.

3.4 Site Grading

We expect that the project will include limited site grading to construct building pads for foundations, pavement areas and storm water facilities. Site grading should be designed and performed in accordance with Section 1804 and Appendix J of the OSSC. CGS should observe prepared subgrade in areas to receive Engineered Structural Fill prior to fill placement. Fill should be placed in accordance with Section *3.8 Engineered Structural Fill*.

Permanent cut and fill slopes should not exceed a grade of 2H:1V (Horizontal to Vertical). Slopes that will be maintained by mowing should not be constructed steeper than 3H:1V. CGS recommends that fill slopes be overbuilt by about 3 feet and trimmed back to finish grade in order to construct a stable slope face that is resistant to shallow sloughing and erosion. Structures and paved surfaces should be located at least 3 feet horizontal from the slope face. Finish slope faces should be planted with appropriate vegetation to provide protection against erosion. Surface water runoff should be collected and directed away from slopes steeper than 3H:1V to prevent water from running down the slope face.

3.5 Difficult Excavating Conditions on Basalt Rock

Based on our program of test pit exploration, we expect that excavations deeper than 7 feet bgs will encounter difficult excavating conditions on BASALT rock. The estimated rock hardness classification of BASALT encountered in eleven of the fourteen test pits was Medium-hard (R3). For reference purposes, Appendix A presents a modified ODOT rock hardness classification chart with typical excavation methods for each rock hardness class.

We encountered practical refusal on BASALT rock with a medium-sized (8,000 GVW) tracked-excavator at depths of 7.5 to 9 feet bgs. In one test pit where a localized seam of stiff to very-stiff silty CLAY (CH) was present, we achieved an excavation depth of 10 feet bgs, the maximum reach of the tracked-excavator.

Excavations deeper than 7 feet bgs will likely require large excavating equipment with ripper teeth and/or use of a hydraulic rock-chipper attachment. Excavation to depths greater than about 10 feet may require costly, time-consuming methods such as extensive hydraulic rock chipping or demolition with non-explosive expansion compounds such as Bristar.

3.6 Temporary Excavations

The stability of temporary excavation slopes is a function of many factors, including soil type, soil density, slope inclination, slope height, the presence of groundwater, and the duration of exposure. Generally, the likelihood of slope failure increases as the cut is deepened and as the duration of exposure increases. For this reason, temporary slope safety should remain the responsibility of the contractor, who is continually present at the site and is able to monitor the performance of the excavation and modify construction practices to reflect varying conditions.

Regardless of inclination, temporary slopes should be protected from surface runoff of storm water. This can typically be accomplished using berms or swales located along the top of the slope, and by placing plastic tarpaulins over the slope.

We recommend that the excavation contractor maintain adequate slopes and setbacks in conformance with OSHA Excavation Guidelines and all applicable regulations. Temporary cut slopes for the construction of basements or retaining walls should be limited to 1H:1V.

3.7 Utility Trench Backfill

Utility trench backfill in structural areas should consist of well-graded, granular fill limited to a maximum particle size of 1½ inches. Granular trench backfill should be compacted to at least 92% of the maximum dry density as determined by ASTM D1557. Excavator-mounted, vibratory-plate compactors are typically the most efficient for compaction of trench backfill. Lift thicknesses should be evaluated based on field density tests; however, care should be taken when operating vibratory compactors to prevent damage to pipes. An initial lift thickness over pipe may need to be up to 4 feet to protect the pipe from damage during compaction; however, thick lifts of loosely placed backfill should not be the standard practice for utility trench backfill. Native materials can be used for trench backfill in non-structural areas where a soft trench and future settlement of the backfill can be tolerated.

3.8 Engineered Structural Fill

Structural fill is any fill material used for support of foundations, retaining walls, slab-on-grade floors, sidewalks, embankments, pavements, and similar features. Upon approval by our office, the on-site soil is suitable for use as structural fill provided it can be separated from unsuitable material, be properly moisture conditioned, and compacted to the specified density as determined by standard testing in a soils lab. On-site soil used as structural fill should be placed in lifts with a maximum compacted thickness of 8 inches. Unsuitable, deleterious materials such as organics, wood, construction debris and oversize material should be removed prior to placement of the on-site soil as engineered structural fill.

Imported granular material should be used for engineered structural fill if the on-site material cannot be properly moisture conditioned or if fill is to be placed in tight access locations not accessible by appropriate compaction equipment. Imported granular fill should consist of crushed aggregate that is fairly well-graded between coarse and fine material and have less than 5 percent by weight passing the U.S. Standard No. 200 Sieve. Use of alternative granular fill material such as pit-run or quarry-run rock or sand should be evaluated for suitability by CGS prior to its use. Granular fill should be placed in lifts with a maximum compacted thickness of 12 inches.

All engineered structural fill should be compacted to at least 92% of the maximum dry density determined by Modified Proctor ASTM D1557 or equivalent. CGS should perform density testing of engineered structural fill with a nuclear density gauge to evaluate the compaction and moisture content of the tested soils. Moisture content at the time of compaction should be no more than 2% dry of optimum and 4% wet of optimum. Acceptable moisture contents may be adjusted by CGS personnel based on field performance observed at the time of construction. Proof-rolling with a loaded dump truck or water truck to evaluate fill compaction may be allowed in certain circumstances under the guidance of CGS.

Regardless of material or location, structural fill should be placed over firm, unyielding subgrade prepared in accordance with the "Site Preparation" section of this report. The condition of the subgrade should be evaluated by a CGS representative before filling or construction begins. Fill compaction should be verified by in-place density tests taken during fill placement to confirm that compaction meets project specifications.

3.9 Shallow Foundations

In our opinion, the proposed buildings and associated structures can be supported on shallow, spread footings bearing on a minimum 8-inch-thick layer of new engineered structural fill placed over stiff, native soil. Foundation design, construction, and setback requirements should conform to the Oregon Structural Specialty Code (OSSC) and other governing codes as applicable.

We recommend an allowable bearing pressure of 2,000 pounds per square foot (psf) for design of footings. The recommended allowable bearing pressure applies to the total of dead plus long-term live loads. The allowable bearing pressure may be increased by a factor of 1.33 for short-term loads, such as those resulting from wind or seismic forces.

Total static settlement of footings founded as recommended is expected to be less than 1 inch. Differential settlement is estimated to be less than ¾ inches over a horizontal span of 20 feet. Most of the settlement will occur during construction as the loads are applied. These estimates are based on maximum wall loads of 2,500 pounds per lineal foot and a maximum column load of 60 kips. For heavier loads, CGS should be consulted.

CGS should review the preliminary structural foundation loading plan, once it becomes available so that we can refine our settlement estimates. We expect that construction of granular engineered structural fill pads beneath footing areas may be necessary to reduce footing settlement to within structural tolerances for heavier column loads.

For protection against frost heave and maximizing bearing strength, perimeter footings should be embedded at least 18 inches below exterior finish grade. Interior footings should be embedded at least 12 inches below floor slabs. Minimum footing widths should be determined by the project architect/designer/structural engineer in accordance with applicable design codes. The OSSC specifies a minimum footing width of 12 inches for one-story, 15 inches for two-story, and 18 inches for three-story, light-frame structures. Excavations adjacent to footings should not extend beneath a 1H:1V plane projected downwards from the bottom edge of the footing or be backfilled with engineered structural fill.

Footing excavations should be trimmed neat and carefully prepared. Loose, wet or otherwise softened subgrade should be removed from footing areas prior to placing crushed rock backfill, forms and reinforcing steel. In wet weather conditions, we recommend that a several-inch-thick layer of granular material (typically 3/4"-0 crushed aggregate) be placed at the base of footing excavations. The granular material reduces water softening of subgrade soils, reduces subgrade disturbance during placement of forms and reinforcement, and provides a clean environment for reinforcing steel. To be effective, the granular material should be placed on firm, well-drained subgrade and lightly compacted until well-keyed using a small vibratory plate compactor.

We recommended that CGS observe the foundation excavation subgrade prior to placing structural fill, formwork, or reinforcing steel to evaluate subgrade support conditions are within recommended specifications.

3.9.1 Lateral Resistance

Lateral loads on the proposed structure imposed by wind or seismic forces 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 an ultimate coefficient of friction of 0.35 for footings bearing on silt and 0.5 for footings bearing on structural granular fill.

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

3.10 Slab on Grade Floors

Satisfactory subgrade support for lightly-loaded building floor slabs can be obtained on undisturbed native soil or on newly placed structural fill. The modulus of subgrade reaction for design of floor slabs may be taken as 100 pounds per cubic inch.

A minimum 8-inch-thick layer of imported granular material should be placed and compacted over the prepared subgrade to assist as a capillary break and blanket drain. Imported granular material should consist of crushed rock, crushed gravel or sand that is fairly well-graded between coarse and fine, contains no deleterious materials, has a maximum particle size of 1½ inches, and less than 5 percent by weight passing the U.S. Standard No. 200 Sieve. The imported granular material may be placed in one lift and should be compacted until well-keyed, to about 85 percent of the maximum dry density as determined by ASTM D 1557. An underslab drainage pipe system placed at the base of the granular material with a minimum 0.5% fall to a lowpoint drain outlet is recommended for living areas with concrete slab floors.

A vapor retarder manufactured for use beneath floor slabs should be installed above the base rock and according to the manufacturer's recommendations. Careful attention should be made during construction to prevent perforating the retarder, and to seal edges and utility penetrations. We recommend following ACI 302.1, Chapter 3 with regard to installing a vapor retarder.

3.11 Retaining Walls

Because the project is in the preliminary design phase, it is unclear whether structural retaining walls will be included. Lateral pressures presented in this report are to be considered as general guidelines, should retaining walls be included. CGS should be consulted for feature-specific recommendations.

The design engineer for the retaining wall must take into consideration the state at which the soil retention walls will be placed, whether under active, passive, or at-rest pressures. Walls that may deflect by at least 0.01 times their height may be designed with active earth pressures. Walls that may not deflect should be designed with at-rest pressures. The possibility of additional non-seismic surcharge loading should also be considered.

Our recommended lateral earth pressures for design of retaining walls presented as equivalent fluid pressures are summarized in Table 3-1, below. Active and at-rest pressures should be modelled as a static triangular pressure profile with the resultant total force acting at one-third height of the exposed wall face. The recommended values are based on free-draining granular backfill, a wet density of 135 pounds per cubic foot and a friction angle of 35 degrees for the retained soils. The tabulated design parameters are to be used only for well-drained backfill conditions with no hydrostatic pressures behind the walls.

Table 3-1 - Equivalent Fluid Pressure Acting on Retaining Walls

Wall Type	Backfill Slope	Backfill Equivalent Fluid Pressure (pcf)
Active (Yielding wall)	Level	35
	2H:1V	50
At-Rest (Non-yielding wall)	Level	50
	2H:1V	70

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

If the wall will be subjected to the influence of surcharge loading, the wall should be designed for an additional horizontal pressure. For uniform surcharge pressures, a uniformly distributed lateral pressure of 0.3 times the vertical surcharge pressure should be added. The influence zone of an applied vertical load is generally considered to be a 45 degree plane projected downward from the bottom edge of the footing. Traffic surcharges may be estimated using an additional vertical load of 250 psf (2 feet of additional fill), in accordance with local practice, or as determined by the type of traffic expected to apply the surcharge loads.

It is difficult to accurately predict the additional lateral forces that will be generated on a retaining wall during an earthquake. Some factors affecting the magnitude of earthquake forces on the wall are the size and duration of the earthquake, the distance from the earthquake epicenter of the site, and the mass of soil retained by the wall. Retaining walls that are designed only for active earth pressures may fail when additional forces are generated by an earthquake.

A simple approach based on the work of Seed and Whitman (1970), is to include in the design analysis an additional horizontal force (P_E) to account for the additional loads imposed on the retaining wall by the earthquake (dynamic load)[5]. In this case, the static force is calculated and then an additional dynamic force (as shown below) is added to the wall for failure analysis.

$$P_E = \frac{3}{8}(0.5 * PGA_M)\gamma_t H^2$$

Where PGA_M = Peak Ground Acceleration (see Table 3-3)
γ_t = total unit weight of soil
H = height of retaining wall

The resultant of this equation is given in pounds per linear foot of wall. The location of this earthquake-induced force can be assumed to act at a distance of 0.6H up from the base of the wall.

Because P_E is a short-term loading that may never occur during the life of the retaining wall, it is common to allow a one third increase in the bearing pressure and passive resistance for the earthquake analysis. Also, for the analysis of sliding and overturning of the retaining wall, it is common to accept a lower factor of safety (1.1 to 1.2) under the combined static and earthquake loads.[6]

A layer of compacted aggregate that is a minimum of 1-foot-wide should be placed behind all retaining walls to allow for proper drainage, and placed utilizing the compaction recommendations described in this report. All structural retaining walls should be backfilled with an imported, free-draining granular material such as ¾"-0 crushed rock with no more than 5% passing the No. 200 sieve. Only light-weight compaction equipment should be used immediately behind retaining walls, so that compactive effort does not damage the wall.

At the base of the retaining walls and continuous with the wall backfill aggregate, a wall subdrain should be installed to divert water from the retaining the structures. The wall subdrain should consist of a 3- or 4-inch-diameter, perforated, gravity drain pipe (ADS Highway Grade or better) enveloped in at least 4 cubic feet per lineal foot of clean, drain rock. The drain rock should be wrapped within geotextile filter

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[5] Seed, H.B. and Whitman, R.V., 1970, Design of Earth Retaining Structures for Dynamic Loads: ASCE Specialty Conference, Lateral Stresses in the Ground and Design of Earth Retaining Structures, Cornell University, Ithaca, New York, p. 103-147.

[6] Day, Robert W. "Geotechnical Engineer's Portable Handbook". Second Edition, 2012. Pg. 16.18, Table 16.5, Topic (1).

fabric with a minimum 1 foot overlap at joints to prevent fines from washing into the drain rock. A diagram of a typical wall subdrain can be found in Appendix B as a recommended guideline for construction.

Retaining walls in living areas or other moisture sensitive areas should include water proofing and wall panel drains as specified by the wall designer in accordance with Section 1805 of the OSSC.

3.12 Site Drainage

Site drainage should include foundation drainage, surface runoff collection, and conveyance to a properly designed and permitted storm water drainage facility. As a matter of good construction practice, we recommend that perimeter footing subdrains be installed for all buildings. Perimeter subdrains should conform to the requirements of Section 1805.4.2 of the OSSC and should consist of perforated drainpipe enveloped in a zone of drain rock that is wrapped in a non-woven geotextile filter fabric. The subdrain should be connected to a non-perforated drainpipe conveyance to storm drain facilities. A diagram of a typical footing subdrain is presented in Appendix B as a recommended guideline for construction.

Water should not be allowed to pond beneath floor slabs or within crawl spaces. Floor slab and crawl space subgrade should be sloped to drain to a suitable low point drain outlet or sump to provide positive drainage from the area under the building in accordance with Section 1804.8 of the OSSC. The drain location and routing should be carefully considered to ensure drainage occurs as intended. It might be necessary to install underslab drainage and provide for sump pumps, depending on the below grade depth of floor slabs.

We recommend that all roof drains be connected to a non-perforated drainpipe leading to storm drain outlet facilities. Pavement surfaces and open space areas should be sloped such that surface water runoff is collected and routed to suitable discharge points. Ground surfaces adjacent to buildings should be sloped to drain away from the buildings in accordance with Section 1804.4 of the OSSC.

3.13 Storm Water Infiltration Facilities

Based on the results of field testing, we consider the project site suitable for limited subsurface disposal of storm water from a geotechnical perspective. Vertical infiltration is restricted by the presence of shallow, low permeability silt soil and basalt bedrock below the proposed storm water facilities.

For shallow infiltration systems constructed near a depth of 2 to 3 feet bgs, we recommend a design infiltration drawdown rate of 1.5 inches per hour. A field performance test of infiltration facilities is recommended at the time of construction to verify that the effective infiltration rate meets or exceeds the recommended rate. It should be noted that infiltration rates of the in-situ soils may vary across the site, and as such testing is generally required by the permitting agency at the actual location and depth of the facility to be constructed.

Because infiltration rates tend to decrease over time due to siltation clogging, an appropriate factor of safety (correction factor) should be applied to the recommended rate by the system designer. We recommend a factor of safety of 3 to 5 be applied to the recommended rates, because fracture permeability in basalt has shown to be susceptible to clogging and reduction in the rate of infiltration over a short period of time. Incorporation of silt traps and pre-filter elements will extend the service life of the system. All systems should include overflow outlets that discharge potential overflow to a suitable dispersal area.

3.14 Pavement Profiles

We do not have specific information on the frequency and type of vehicles that will use the development on a daily basis. Typically, pavement design requirements are controlled by the Fire Code, which states fire apparatus access roads shall be capable of supporting not less than 12,500 pounds point load (wheel load or gross wheel position weight) and 75,000 pounds live load (gross vehicle weight).

For design purposes, we assumed that post-construction traffic will be primarily light duty passenger vehicles averaging no more than five heavy trucks per day. The thickness of the driveway pavement profile is governed by the fire apparatus support requirements. If actual traffic loading will exceed those described above, we should be contacted to revise our recommended pavement sections.

We recommend the minimum pavement section profiles presented in Table 3-2 to support the anticipated traffic loads over a design life of 20 years. Our pavement recommendations are based on a typical subgrade stiffness for compacted soil at the site using a California Bearing Ratio value of 3. For loading dock approaches or areas where service trucks back and turn, a Portland Cement Concrete (PCC) pavement section should be used or the AC pavement thickness increased to 5 inches. The recommended minimum PCC section is 6 inches of PCC over 8 inches of 1½"-0 crushed rock compacted to at least 95% of ASTM D1557.

These thicknesses are intended to be the minimum acceptable for construction completed during an extended period of dry weather. If pavement areas are constructed during wet weather, CGS should review the subgrade and proposed construction methods immediately prior to the placement of base course so that specific recommendations can be provided. Wet-weather pavement construction may require cement amendment or an additional 6 inches of crushed aggregate base.

Table 3-2 - Recommended Minimum Dry-Weather Pavement Sections

Material	Driveway Areas Thickness (in)	Parking Areas Thickness (in)	Compaction Standard
Asphaltic Concrete (AC)	4	3	92% of Rice Density AASHTO T-209
Crushed Aggregate Base ¾"-0 (leveling course)	2	2	95% of Modified Proctor
Crushed Aggregate Base 1½ "-0	8	6	95% of Modified Proctor

AC pavement should conform to Section 0074 of the Standard Specification for Highway Construction, Oregon Highway Specifications, and Marion County requirements. We recommend graded half-inch or three-quarter-inch, Dense Hot Mix Asphalt Concrete for Design Level 2 using Performance Grade Asphalt PG-64-22 which is appropriate for low to moderate volume pavements in Western Oregon. The aggregate base should conform to Section 02630 of the 2018 ODOT Oregon Standard Specifications for Construction with the addition that no more than 5 percent of the material by dry weight passes the U.S. Standard No. 200 Sieve. Aggregate base contaminated with soil during construction should be removed and replaced before paving.

As a matter of good construction practice, we recommend placing a woven separation fabric between the soil subgrade and the aggregate such as Contech C200 or US200. The fabric should conform to the minimum property values presented in Table 02320-4 – Subgrade Geotextile (Separation), in Section 02320 of the 2018 ODOT Oregon Standard Specifications for Construction.

We recommend that CGS conduct density testing and a proof roll performance test of the pavement subgrade prior to placement. Subgrade and base rock should be compacted to at least 95% of the maximum dry density obtained from ASTM D1557. Subgrade strength should be evaluated visually by proof-rolling directly on the subgrade with a loaded dump truck during dry weather and on top of base course in wet weather. Soft areas which rut, pump, or weave by more than ¼ inch should be stabilized prior to paving.

3.15 Seismic Design Considerations

At this time, we presume that the building will be designed to resist earthquake loading in accordance with the 2017 ASCE 7-16 standard methodology and as prescribed by the 2019 OSSC. Based on the results of our test pit explorations, pocket penetrometer readings of the soil strength, and laboratory tests, we designate the building site to be Seismic Site Class C.

Site coefficients and spectral response acceleration parameters determined for the site using the ASCE Hazard Tool in accordance with the standard ASCE 7-16 methodology are presented in Table 3-3, on the following page. These values are based on risk-targeted maximum considered earthquake (MCE_r) ground motions for the 0.2 and 1 second spectral response accelerations provided in the 2019 OSSC. The values are the lessor of deterministic and probabilistic estimates (2% chance of exceedance in 50 years at 5% critical dampening) of ground motion based on USGS hazard map data available in 2008 and updated in 2014.

Table 3-3 - Seismic Design Parameters (ASCE7-16)

Parameter	Value
Location (Lat., Lon. in degrees)	44.8795, -123.0118
Mapped Maximum Considered Earthquake Spectral Response Acceleration (USGS Mapping Standardized to Site Class B)	
Short Period, S_S	0.809 g
1 Second Period, S_1	0.409 g
Design Site Coefficients (Site Class C)	
F_a	1.2
F_v	1.5
Design Spectral Response Acceleration Parameter (Site Class C)	
S_{DS} (2/3 x F_a x S_s)	0.647 g
S_{D1} (2/3 x F_v x S_1)	0.409 g
Seismic Design Category	D
Peak Ground Acceleration (PGA_M)	0.451 g

4.0 LIMITATIONS OF REPORT

We have prepared this report for the exclusive use of Community Development Partners and members of the design team, for this specific project only. The full geotechnical 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 Geotech should be notified for review of the recommendations of this report, and revision of such if necessary.

We recommend that Central Geotech 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. Should Central Geotech not be retained for Design or Construction related services further into the development process, this report and its recommendations should be considered void, as we cannot take on responsibility for construction operations that were unobserved by our office.

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. No warranty, express or implied, is made. The scope of our work did not include environmental assessments or evaluations regarding the presence or absence of wetlands or hazardous or toxic substances in the soil, surface water, or groundwater at this site.

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

97231PE
JOSE R. SERRANO
5/11/2021

EXPIRES: 06/30/22

Jose R. Serrano, P.E.
Associate Engineer

Kyle Warren
Staff Geologist

PAUL A. CRENNA
E1766

RENEWS 11-1-22

Paul A. Crenna C.E.G.
Principal Engineering Geologist

APPENDIX A: LOGS OF EXPLORATORY TEST PITS

CENTRAL
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10240 SW NIMBUS AVENUE, SUITE L6
PORTLAND, OR 97223 - 503.616.9419
WWW.CENTRALGEOTECH.COM

TEST PIT LOG

PROJECT: MILLER SITE – BATTLE CREEK ROAD
SALEM, OREGON

CGS PROJECT NO. 21-023

TEST PIT NO. **TP-1**

DEPTH (FT)	P. PENE-TROMETER (TSF)	CORRELATE D N-VALUE	SAMPLE NO.-TYPE	MOISTURE/ GROUND WATER	MATERIAL DESCRIPTION	LAB TEST RESULT
1	1.0				Medium-stiff, gravelly SILT (ML-OL), mixed dark brown to brown, fine organics in growth position in upper 18-inches, gravel is up to 6-inches in diameter and subrounded, damp (FILL)	
2	0.5		1-GS	38%		
3	2.5				Stiff to very-stiff, clayey SILT (ML), brown to orange-brown, trace fine to coarse sand, includes abundant gravel to cobble-sized, sub-angular rock fragments in lower few feet, damp (RESIDUAL SOIL)	
4	2.0		2-GS 3-GS	28%		
5						
6			4-GS	▨		
7				▨ 38%		
8			5-GS		Soft (R2) to medium-hard (R3) BASALT, fresh surfaces are gray with orange-brown staining on fracture faces, includes seams of silt and clay in upper 6 inches, damp (COLUMBIA RIVER BASALT)	
9						
10					Practical refusal on Medium-Hard (R3) BASALT at 7.5 feet bgs Slow groundwater seepage at 5.5 feet bgs Test pit walls standing vertical	
11						
12						
13						
14						
15						

LEGEND

20% SOIL MOISTURE CONTENT	0-10-18 ▼ STATIC GROUND WATER LEVEL WITH DATE OF MEASUREMENT	▼ GROUND WATER LEVEL AT END OF DRILLING	▨ GROUND WATER SEEPAGE ZONE

DATE EXCAVATED: 2-27-2021
LOGGED BY: K. Warren
SURFACE ELEVATION:
EQUIPMENT: Hitachi Z-Axis 40u

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TEST PIT NO. **TP-2**

DEPTH (FT)	P. PENE-TROMETER (TSF)	CORRELATE D N-VALUE	SAMPLE NO.-TYPE	MOISTURE/ GROUND WATER	MATERIAL DESCRIPTION	LAB TEST RESULT
1	1.0				Soft to medium-stiff, clayey SILT (ML-OL), mixed dark brown to brown, fine organics in growth position in upper 24-inches, gravel up to 6-inches in diameter and subrounded in upper 15-inches, damp (FILL/DISTURBED NATIVE)	
2	0.5		1-GS			
3	1.0				Stiff to very-stiff, clayey SILT (ML), brown to orange-brown, trace fine to coarse sand, includes abundant gravel to cobble-sized, sub-angular rock fragments in lower few feet, damp (RESIDUAL SOIL)	
4	1.0		2-GS			
5						
6			3-GS			
7				▨		
8					Soft (R2) to medium-hard (R3) BASALT, fresh surfaces are gray with orange-brown staining on fracture faces, includes seams of silt and clay in upper 6 inches, damp (COLUMBIA RIVER BASALT)	
9						
10						
11					Practical refusal on Medium-Hard (R3) BASALT at 9.5 feet bgs Slow groundwater seepage at 6.5 feet bgs Test pit walls standing vertical	
12						
13						
14						
15						

LEGEND

20% SOIL MOISTURE CONTENT	0-10-18 ▼ STATIC GROUND WATER LEVEL WITH DATE OF MEASUREMENT	▼ GROUND WATER LEVEL AT END OF DRILLING	▨ GROUND WATER SEEPAGE ZONE

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TEST PIT NO. **TP-3**

DEPTH (FT)	P. PENE-TROMETER (TSF)	CORRELATED N-VALUE	SAMPLE NO.-TYPE	MOISTURE/ GROUND WATER	MATERIAL DESCRIPTION	LAB TEST RESULT
1	0.5				Soft to medium-stiff, clayey SILT (ML-OL), mixed dark brown to brown, fine organics in growth position in upper 24-inches, gravel up to 6-inches in diameter and subrounded in upper 15-inches, damp (FILL/DISTURBED NATIVE)	
2	0.5		1-GS	31%		
3	1.0				Soft (R2) to medium-hard (R3) BASALT, brown, gray, black, fresh surfaces are gray with orange-brown staining on fracture faces, includes seams of silt and clay in upper 12 inches, damp (COLUMBIA RIVER BASALT)	
4	5.0		2-GS	40%		
5						
6						
7			3-GS	▨ 38%	Practical refusal on Medium-Hard (R3) BASALT at 7 feet bgs Slow groundwater seepage at 7 feet bgs Test pit walls standing vertical	
8						
9						
10						
11						
12						
13						
14						
15						

LEGEND

20%	SOIL MOISTURE CONTENT	0-10-18 ▼	STATIC GROUND WATER LEVEL WITH DATE OF MEASUREMENT
▼	GROUND WATER LEVEL AT END OF DRILLING	▨	GROUND WATER SEEPAGE ZONE

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TEST PIT NO. **TP-4**

DEPTH (FT)	P. PENE-TROMETER (TSF)	CORRELATED N-VALUE	SAMPLE NO.-TYPE	MOISTURE/ GROUND WATER	MATERIAL DESCRIPTION	LAB TEST RESULT
1	0.5				Soft to medium-stiff, clayey SILT (ML-OL), mixed dark brown to brown, fine organics and roots up to $^{1}/_{2}$-inch diameter in growth position in upper 18-inches, gravel up to 6-inches in diameter and subrounded in upper 12-inches, damp (FILL/DISTURBED NATIVE)	
2	0.5		1-GS			
3	1.0				Stiff to very-stiff, clayey SILT (ML), brown to orange-brown, trace fine to coarse sand, includes abundant gravel to cobble-sized, sub-angular rock fragments in lower few feet, damp (RESIDUAL SOIL)	
4	5.0		2-GS			
5						
6			3-GS			
7					Soft (R2) to medium-hard (R3) BASALT, brown, gray, black, fresh surfaces are gray with orange-brown staining on fracture faces, includes seams of silt and clay in upper 12 inches, damp (COLUMBIA RIVER BASALT)	
8			4-GS			
9						
10					Practical refusal on Medium-Hard (R3) BASALT at 8 feet bgs No groundwater encountered Test pit walls standing vertical	
11						
12						
13						
14						
15						

LEGEND
20% SOIL MOISTURE CONTENT
0-10-18 ▽ STATIC GROUND WATER LEVEL WITH DATE OF MEASUREMENT
▼ GROUND WATER LEVEL AT END OF DRILLING
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TEST PIT NO. **TP-5**

DEPTH (FT)	P. PENETROMETER (TSF)	CORRELATED N-VALUE	SAMPLE NO.-TYPE	MOISTURE/ GROUND WATER	MATERIAL DESCRIPTION	LAB TEST RESULT
1	0.5		1-GS	29%	Soft to medium-stiff, clayey SILT (ML-OL), mixed dark brown to brown, fine organics and roots up to $^{1}/_{2}$-inch diameter in growth position in upper 18-inches, gravel up to 6-inches in diameter and subrounded in upper 12-inches, damp (FILL/DISTURBED NATIVE)	
2						
3			2-GS	51%	Soft (R2) to medium-hard (R3) BASALT, brown, gray, black, red, fresh surfaces are gray with orange-brown staining on fracture faces, includes seams of silt and clay in upper 12 inches, damp (COLUMBIA RIVER BASALT)	
4						
5						
6						
7						
8			3-GS		Practical refusal on Medium-Hard (R3) BASALT at 8 feet bgs No groundwater encountered Test pit walls standing vertical	
9						
10						
11						
12						
13						
14						
15						

LEGEND

20%	SOIL MOISTURE CONTENT	0-10-18 ▼	STATIC GROUND WATER LEVEL WITH DATE OF MEASUREMENT
▼	GROUND WATER LEVEL AT END OF DRILLING	▨	GROUND WATER SEEPAGE ZONE

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TEST PIT NO. **TP-6**

DEPTH (FT)	P. PENE-TROMETER (TSF)	CORRELATED N-VALUE	SAMPLE NO.-TYPE	MOISTURE/ GROUND WATER	MATERIAL DESCRIPTION	LAB TEST RESULT
1	0.5		1-GS		Soft to medium-stiff, clayey SILT (ML-OL), mixed dark brown to brown, fine organics and roots up to $^{1}/_{2}$-inch diameter in growth position in upper 18-inches, damp (FILL/DISTURBED NATIVE)	
2						
3			2-GS		Soft (R2) to medium-hard (R3) BASALT, brown, gray, black, red, fresh surfaces are gray with orange-brown staining on fracture faces, includes seams of silt and clay in upper 12 inches, vesicles present in rock fragments, damp (COLUMBIA RIVER BASALT)	
4						
5						
6						
7						
8						
9			3-GS		Practical refusal on Medium-Hard (R3) BASALT at 9 feet bgs No groundwater encountered Test pit walls standing vertical	
10						
11						
12						
13						
14						
15						

LEGEND

20%	SOIL MOISTURE CONTENT	0-10-18 ▼	STATIC GROUND WATER LEVEL WITH DATE OF MEASUREMENT
▼	GROUND WATER LEVEL AT END OF DRILLING	▨	GROUND WATER SEEPAGE ZONE

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TEST PIT NO. **TP-7**

DEPTH (FT)	P. PENE-TROMETER (TSF)	CORRELATE D N-VALUE	SAMPLE NO.-TYPE	MOISTURE/ GROUND WATER	MATERIAL DESCRIPTION	LAB TEST RESULT
1	0.5		1-GS	24%	Soft to medium-stiff, clayey SILT (ML-OL), mixed dark brown to brown, fine organics and roots up to $^1/_2$-inch diameter in growth position in upper 18-inches, damp	
2	0.5				(FILL/DISTURBED NATIVE)	
3			2-GS	42%	Very-soft (R1) to medium-hard (R3) BASALT, brown, gray, black, red, fresh surfaces are gray with orange-brown staining on fracture faces, includes seams of silt and clay in upper 12 inches, vesicles present in rock fragments, damp	
4						
5					(COLUMBIA RIVER BASALT) Color transitioned to gray	
6					Color transitioned to red	
7						
8			3-GS			
9			4-GS		Terminated at 9 feet bgs No groundwater encountered Test pit walls standing vertical	
10						
11						
12						
13						
14						
15						

LEGEND

20% SOIL MOISTURE CONTENT

0-10-18 ▼ STATIC GROUND WATER LEVEL WITH DATE OF MEASUREMENT

▼ GROUND WATER LEVEL AT END OF DRILLING

▨ GROUND WATER SEEPAGE ZONE

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CGS PROJECT NO. 21-023

TEST PIT NO. **TP-8**

DEPTH (FT)	P. PENE-TROMETER (TSF)	CORRELATED N-VALUE	SAMPLE NO.-TYPE	MOISTURE/ GROUND WATER	MATERIAL DESCRIPTION	LAB TEST RESULT
1	0.5		1-GS		Soft to medium-stiff, clayey SILT (ML-OL), mixed dark brown to brown, fine organics and roots up to $^{1}/_{2}$-inch diameter in growth position in upper 18-inches, damp (FILL/DISTURBED NATIVE)	
2	0.5					
3	2.5		2-GS		Stiff to very-stiff, clayey SILT (ML), brown to orange-brown, trace fine to coarse sand, includes abundant gravel to cobble-sized, sub-angular rock fragments in lower few feet, damp (RESIDUAL SOIL)	
4	4.0					
5						
6						
7						
8					Very-soft (R1) to soft (R2) BASALT, brown, gray, black, red, fresh surfaces are gray with orange-brown staining on fracture faces, includes seams of silt and clay in upper 12 inches, vesicles present in rock fragments, damp (COLUMBIA RIVER BASALT)	
9			3-GS			
10					Terminated at 9 feet bgs No groundwater encountered Test pit walls standing vertical	
11						
12						
13						
14						
15						

LEGEND

20% SOIL MOISTURE CONTENT

0-10-18 ▼ STATIC GROUND WATER LEVEL WITH DATE OF MEASUREMENT

▼ GROUND WATER LEVEL AT END OF DRILLING

▨ GROUND WATER SEEPAGE ZONE

DATE EXCAVATED: 2-27-2021
LOGGED BY: K. Warren
SURFACE ELEVATION:
EQUIPMENT: Hitachi Z-Axis 40u

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TEST PIT LOG

PROJECT: MILLER SITE – BATTLE CREEK ROAD
SALEM, OREGON

CGS PROJECT NO. 21-023

TEST PIT NO. **TP-9**

DEPTH (FT)	P. PENE-TROMETER (TSF)	CORRELATED N-VALUE	SAMPLE NO.-TYPE	MOISTURE/ GROUND WATER	MATERIAL DESCRIPTION	LAB TEST RESULT
1	0.5				Soft to medium-stiff, clayey SILT (ML-OL), mixed dark brown to brown, fine organics and roots up to $^1/_2$-inch diameter in growth position in upper 18-inches, damp (FILL/DISTURBED NATIVE)	
2	0.5		1-GS	31%		
3	1.5				Stiff to very-stiff, clayey SILT (ML), brown to orange-brown, trace fine to coarse sand, includes abundant gravel to cobble-sized, sub-angular rock fragments in lower few feet, damp (RESIDUAL SOIL)	
4	3.0		2-GS	30%		
5						
6						
7				▨		
8			3-GS			
9					Soft (R2) to medium-hard (R3) BASALT, brown, gray, black, red, fresh surfaces are gray with orange-brown staining on fracture faces, includes seams of silt and clay in upper 12 inches, vesicles present in rock fragments, damp (COLUMBIA RIVER BASALT)	
10						
11					Practical refusal on Medium-Hard (R3) BASALT at 8.5 feet bgs Slow groundwater seepage at 7.5 feet bgs Test pit walls standing vertical	
12						
13						
14						
15						

LEGEND

20%	SOIL MOISTURE CONTENT		
0-10-18 ▼	STATIC GROUND WATER LEVEL WITH DATE OF MEASUREMENT		
▼	GROUND WATER LEVEL AT END OF DRILLING		
▨	GROUND WATER SEEPAGE ZONE		

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TEST PIT LOG

PROJECT: MILLER SITE – BATTLE CREEK ROAD
SALEM, OREGON

CGS PROJECT NO. 21-023

TEST PIT NO. **TP-10**

DEPTH (FT)	P. PENE-TROMETER (TSF)	CORRELATED N-VALUE	SAMPLE NO.-TYPE	MOISTURE/ GROUND WATER	MATERIAL DESCRIPTION	LAB TEST RESULT
1	1.5				Soft to medium-stiff, clayey SILT (ML-OL), mixed dark brown to brown, fine organics and roots up to $^{1}/_{2}$-inch diameter in growth position in upper 18-inches, damp	
2	2.0		1-GS		(FILL/DISTURBED NATIVE)	
3	2.0				Stiff to very-stiff, clayey SILT (ML), brown to orange-brown, trace fine to coarse sand, includes abundant gravel to cobble-sized, sub-angular rock fragments in lower few feet, damp	
4	5.0		2-GS		(RESIDUAL SOIL)	
5				▨		
6						
7					Soft (R2) to medium-hard (R3) BASALT, brown, gray, black, red, fresh surfaces are gray with orange-brown staining on fracture faces, includes seams of silt and clay in upper 12 inches, vesicles present in rock fragments, damp	
8					(COLUMBIA RIVER BASALT)	
9			3-GS		Practical refusal on Medium-Hard (R3) BASALT at 9 feet bgs	
10					Slow groundwater seepage at 5.5 feet bgs	
11					Test pit walls standing vertical	
12						
13						
14						
15						

LEGEND

20%	SOIL MOISTURE CONTENT	0-10-18 ▼	STATIC GROUND WATER LEVEL WITH DATE OF MEASUREMENT
▼	GROUND WATER LEVEL AT END OF DRILLING	▨	GROUND WATER SEEPAGE ZONE

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SURFACE ELEVATION:
EQUIPMENT: Hitachi Z-Axis 40u

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TEST PIT LOG

PROJECT: MILLER SITE – BATTLE CREEK ROAD SALEM, OREGON	CGS PROJECT NO. 21-023	TEST PIT NO. **TP-11**

DEPTH (FT)	P. PENE-TROMETER (TSF)	CORRELATE D N-VALUE	SAMPLE NO.-TYPE	MOISTURE/ GROUND WATER	MATERIAL DESCRIPTION	LAB TEST RESULT
1	1.0				Soft to medium-stiff, clayey SILT (ML-OL), mixed dark brown to brown, fine organics and roots up to $^1/_2$-inch diameter in growth position in upper 18-inches, damp (FILL/DISTURBED NATIVE)	
2	1.5		1-GS	29%		
3	2.5				Stiff to very-stiff, clayey SILT (ML), brown to orange-brown, trace fine to coarse sand, includes abundant gravel to cobble-sized, sub-angular rock fragments in lower few feet, damp (RESIDUAL SOIL)	
4	5.0		2-GS	29%		
5						
6				▨		
7					Soft (R2) to medium-hard (R3) BASALT, brown, gray, black, red, fresh surfaces are gray with orange-brown staining on fracture faces, includes seams of silt and clay in upper 12 inches, vesicles present in rock fragments, damp (COLUMBIA RIVER BASALT)	
8			3-GS			
9					Practical refusal on Medium-Hard (R3) BASALT at 8 feet bgs Slow groundwater seepage at 5.5 feet bgs Test pit walls standing vertical	
10						
11						
12						
13						
14						
15						

LEGEND

20% SOIL MOISTURE CONTENT	0-10-18 ▼ STATIC GROUND WATER LEVEL WITH DATE OF MEASUREMENT	▼ GROUND WATER LEVEL AT END OF DRILLING	▨ GROUND WATER SEEPAGE ZONE

DATE EXCAVATED: 2-27-2021
LOGGED BY: K. Warren
SURFACE ELEVATION:
EQUIPMENT: Hitachi Z-Axis 40u

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TEST PIT LOG

PROJECT: MILLER SITE – BATTLE CREEK ROAD
SALEM, OREGON

CGS PROJECT NO. 21-023

TEST PIT NO. **TP-12**

DEPTH (FT)	P. PENETROMETER (TSF)	CORRELATED N-VALUE	SAMPLE NO.-TYPE	MOISTURE/ GROUND WATER	MATERIAL DESCRIPTION	LAB TEST RESULT
1	1.5				Soft to medium-stiff, clayey SILT (ML-OL), mixed dark brown to brown, fine organics and roots up to $^1/_2$-inch diameter in growth position in upper 18-inches, damp (FILL/DISTURBED NATIVE)	
2	1.5		1-GS			
3	1.5					
4	1.5		2-GS		Stiff to very-stiff, clayey SILT (ML), brown to orange-brown, trace fine to coarse sand, includes abundant gravel to cobble-sized, sub-angular rock fragments in lower few feet, damp (RESIDUAL SOIL)	
5						
6						
7				▨	Soft (R2) to medium-hard (R3) BASALT, brown, gray, black, red, fresh surfaces are gray with orange-brown staining on fracture faces, includes seams of silt and clay in upper 12 inches, vesicles present in rock fragments, damp (COLUMBIA RIVER BASALT)	
8			3-GS			
9						
10						
11						
12						
13						
14						
15						

Practical refusal on Medium-Hard (R3) BASALT at 8 feet bgs

Slow groundwater seepage at 7.0 feet bgs

Test pit walls standing vertical

LEGEND

20%	SOIL MOISTURE CONTENT
0-10-18 ▼	STATIC GROUND WATER LEVEL WITH DATE OF MEASUREMENT
▼	GROUND WATER LEVEL AT END OF DRILLING
▨	GROUND WATER SEEPAGE ZONE

DATE EXCAVATED: 2-27-2021
LOGGED BY: K. Warren
SURFACE ELEVATION:
EQUIPMENT: Hitachi Z-Axis 40u

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TEST PIT LOG

PROJECT: MILLER SITE – BATTLE CREEK ROAD
SALEM, OREGON

CGS PROJECT NO. 21-023

TEST PIT NO. **TP-13**

DEPTH (FT)	P. PENE-TROMETER (TSF)	CORRELATED N-VALUE	SAMPLE NO.-TYPE	MOISTURE/ GROUND WATER	MATERIAL DESCRIPTION	LAB TEST RESULT
1	0.5				Soft to medium-stiff, clayey SILT (ML-OL), mixed dark brown to brown, fine organics and roots up to $^1/_2$-inch diameter in growth position in upper 18-inches, damp (FILL/DISTURBED NATIVE)	
2	1.5		1-GS			
3	1.5				Very-soft (R1) to soft (R2) BASALT, brown, gray, black, red, fresh surfaces are gray with orange-brown staining on fracture faces, includes seams of silt and clay in upper 12 inches, vesicles present in rock fragments, damp (COLUMBIA RIVER BASALT)	
4	1.5		2-GS			
5						
6			3-GS			
7						
8						
9					Stiff to very-stiff, silty CLAY (CH), white to light brown, trace fine to coarse sand-sized, sub-angular rock fragments, damp (MUDSTONE)	
10			4-GS			
11						
12					Terminated at 10 feet bgs No groundwater encountered Test pit walls standing vertical	
13						
14						
15						

LEGEND

20%	SOIL MOISTURE CONTENT	0-10-18 ▼	STATIC GROUND WATER LEVEL WITH DATE OF MEASUREMENT
▼	GROUND WATER LEVEL AT END OF DRILLING	▨	GROUND WATER SEEPAGE ZONE

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SURFACE ELEVATION:
EQUIPMENT: Hitachi Z-Axis 40u

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SALEM, OREGON

CGS PROJECT NO. 21-023

TEST PIT NO. **TP-14**

DEPTH (FT)	P. PENE-TROMETER (TSF)	CORRELATED N-VALUE	SAMPLE NO.-TYPE	MOISTURE/ GROUND WATER	MATERIAL DESCRIPTION	LAB TEST RESULT
1	1.5				Soft to medium-stiff, clayey SILT (ML-OL), mixed dark brown to brown, fine organics and roots up to $^1/_2$-inch diameter in growth position in upper 18-inches, damp (FILL/DISTURBED NATIVE)	
2	1.5		1-GS			
3	3.0					
4	5.0		2-GS		Stiff to very-stiff, clayey SILT (ML), brown to orange-brown, trace fine to coarse sand, includes abundant gravel to cobble-sized, sub-angular rock fragments in lower few feet, damp (RESIDUAL SOIL)	
5						
6						
7					Soft (R2) to medium-hard (R3) BASALT, brown, gray, black, red, fresh surfaces are gray with orange-brown staining on fracture faces, includes seams of silt and clay in upper 12 inches, vesicles present in rock fragments, damp (COLUMBIA RIVER BASALT)	
8			3-GS			
9					Practical refusal on Medium-Hard (R3) BASALT at 8 feet bgs No groundwater encountered Test pit walls standing vertical	
10						
11						
12						
13						
14						
15						

LEGEND

20% SOIL MOISTURE CONTENT	0-10-18 ▼ STATIC GROUND WATER LEVEL WITH DATE OF MEASUREMENT	▼ GROUND WATER LEVEL AT END OF DRILLING	▨ GROUND WATER SEEPAGE ZONE

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SURFACE ELEVATION:
EQUIPMENT: Hitachi Z-Axis 40u

SOIL CLASSIFICATION DESCRIPTION AND GUIDELINES

SOIL CLASSIFICATION

Major Divisions			Symbol	Typical Descriptions
Coarse Grained (More Than 50% Retained By No. 200 Sieve)	Gravel	Clean Gravels	GW	Well-Graded Gravels And Gravel/Sand Mixtures, Little Or No Fines
			GP	Poorly-Graded Gravels, Gravel/Sand Mixtures, Little Or No Fines
		Gravels With Fines	GM	Silty Gravels, Gravel/Sand/Silt Mixtures
			GC	Clayey Gravels, Gravel/Sand/Clay Mixtures
	Sand	Clean Sands	SW	Well-Graded Sand And Gravelly Sands, Little Or No Fines
			SP	Poorly-Graded Sand And Gravelly Sands, Little Or No Fines
		Sands With Fines	SM	Silty Sands, Sand/Silt Mixtures
			SC	Clayey Sands, Sand/Clay Mixtures
Fine Grained (More Than 50% Passing By No. 200 Sieve)	Silts And Clays	Liquid Limit Less Than 50	ML	Inorganic Silts, Silt With Slight Plasticity
			CL	Inorganic Clay, Clay With Low To Medium Plasticity
			OL	Organic Silts, Organic Silty Clays With Low Plasticity
		Liquid Limit More Than 50	MH	Inorganic Silts, Clayey Silts
			CH	Inorganic Clays Of High Plasticity, Fat Clays
			OH	Organinc Clays Of Medium To High Plasticity
Highly Organic Soils			PT	Peat, Humus And Other High Orgainc Soils

SOIL CHARACTERISTICS

Granular Soil		Cohesive Soil		
Relative Density	**Standard Penetration Test**	**Consistency**	**Standard Penetration Test**	**Unconfined Strength (Tsf)**
Very-Loose	0 - 4	Very-Soft	< 2	< 0.25
Loose	4 - 10	Soft	2 - 4	0.25 - 0.5
Medium-Dense	10 - 30	Medium-Stiff	4 - 8	0.5 - 1.0
Dense	30 - 50	Stiff	8 - 16	1.0 - 2.0
Very-Dense	> 50	Very-Stiff	16 - 32	2.0 - 4.0
Standard Penetration Tests Record The Number Of Blows Required To Drive A Split-Spoon Sampler 12 Inches (N-Value)		Hard	32 - 50	> 4.0
		Very-Hard	> 50	

ADDITIONAL SOIL CLASSIFICATION TERMS

Moisture Content	
Dry	Absence Of Moisture, Dusty, Dry To The Touch
Damp	Some Moisture But Leaves No Moisture On Hand
Moist	Leaves Moisture On Hand
Wet	Visible Free Water, Likely From Below Water Table

Groundwater Seepage	
Slow	< 1.0 gpm
Moderate	1.1 - 3.0 gpm
Rapid	> 3.0 gpm

Minor Fractions in Fine Grained Soil	
Trace (Clay, Silt, Sand, or Gravel)	< 15 percent
With (Clay, Silt, Sand, or Gravel)	16 to 30 percent
Clayey, Silty, Sandy, Gravelly	31 to 49 percent

Plasticity	
Nonplastic	Cannot Be Rolled At Any Water Content
Low	3 mm Thread Can Barely Be Rolled But Not Under The Plastic Limit
Medium	Can Be Rolled To 3 mm Thread , Crumbles When Drier Than Plastic Limit.
High	Can Easily Be Rolled To 3 mm Thread. Can Be Rerolled Several Times.

Structure	
Stratified	Alternating Layers of Material or Color > 6 mm
Laminated	Alternating Layers of Material or Color < 6 mm
Fissured	Breaks Along Definite Fracture Planes
Slickenslided	Striated, Polished Or Glossy Fracture Planes
Blocky	Cohesive Soil That Can Be Broken Down Into Angular Lumps Which Resist Further Breakdown
Lenses	Small Pockets Of Different Soils, Note Thickness
Homogeneous	Uniform Color And Appearance Througout

Caving	
Minor	Isolated Spalling
Moderate	Common Spalling
Severe	Will not stand vertical

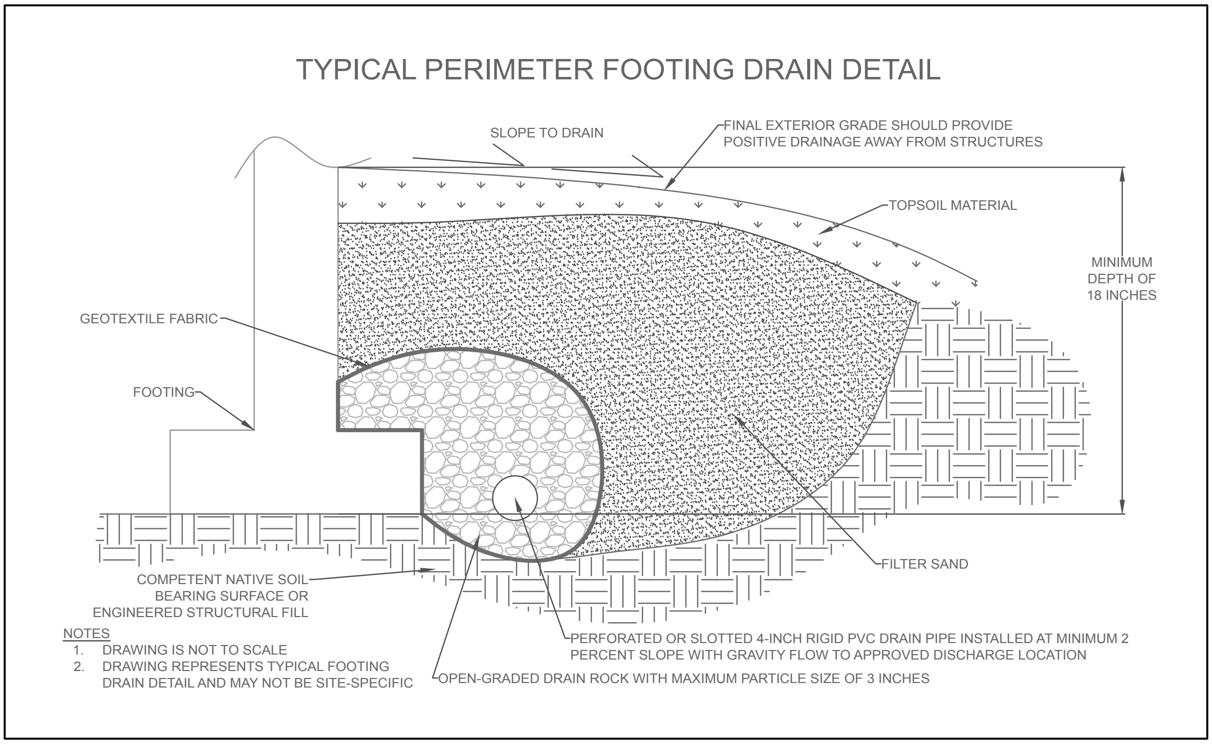
Dilatancy	
None	No Visible Changes in the Specimen
Slow	Water Slowly Appears and Dissapears
Rapid	Water Quickly Appears and Dissapears

ODOT ROCK HARDNESS CLASSIFICATION CHART

Hardness Designation	Field Identification/Excavation Methods	Approx. Strength (Unconfined Compressive Strength)
Extremely-Soft (RO)	Can Be Indented With Thumbnail. May Be Moldable Or Friable With Finger Pressure.	< 100 Psi
Very-Soft (R1)	Crumbles Under Firm Blows With Geology Pick. Scratched With Finger Nail.	100 - 1,000 psi
Soft (R2)	Can Be Peeled By Knife Or Pick. Shallow Indentation Made By Frim Blow Of Geology Pick.	1,000 - 4,000 psi
Medium-Hard (R3)	Can Be Scratched By Knife Or Pick, Specimen Can Be Fractured With A Single Blow Of Hamer Or Geology Pick / Excavation Often Requires Medium To Large Equipment With Ripper Teeth.	4,000 - 8,000 psi
Hard (R4)	Can Be Scratched With Knife Or Pick Only With Difficulty. Several Hammer Blows Required To Fracture Specimen / Excavation Requires Large Equipment, Rock Chipper, Expansive Compound Fracturing Or Blasting.	8,000 - 16,000 psi
Very-Hard (R5)	Cannont Be Scratched By Knife Or Sharp Pick. Specimen Requires Many Blows Of Hammer To Fracture Or Chip. Hammer Rebounds After Impact / Expansive Compound Fracturing Or Blasting Required To Excavate.	> 16,000 psi

Chart Taken From Oregon Department Of Transportation Soil/Rock Classification Manual. Modified To Include Typical Excavation Methods.

APPENDIX B: TYPICAL DETAIL FOR PERIMETER FOOTING SUBDRAIN



Guideline drawing for reference only