Carlson Geotechnical
A Division of Carlson Testing, Inc.
Phone: (503) 601-8250
www.carlsontesting.com

Bend Office	**(541) 330-9155**
Eugene Office	**(541) 345-0289**
Salem Office	**(503) 589-1252**
Tigard Office	**(503) 684-3460**

Report of
Preliminary Geotechnical Investigation
Baxter Road Apartments
1709 Baxter Road SE
Salem, Oregon

CGT Project Number G2406149

Prepared for

Laura Robinson
Neighborly Development
2925 River Road South, Suite 100
Salem, Oregon 97302

July 12, 2024

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Dear Laura Robinson:

Carlson Geotechnical (CGT), a division of Carlson Testing, Inc. (CTI), is pleased to submit this report summarizing the results of our preliminary geotechnical investigation for the proposed Baxter Road Apartments project. The proposed site is located at 1709 Baxter Road SE in Salem, Oregon. We performed our work in general accordance with CGT Proposal GP24-138, dated May 23, 2024. Written authorization for our services was received on June 3, 2024.

We appreciate the opportunity to work with you on this project. Please contact us at (503) 601-8250 if you have any questions regarding this report.

Respectfully Submitted,
CARLSON GEOTECHNICAL

REGISTERED PROFESSIONAL ENGINEER
70773PE
OREGON
Jan. 10, 2007
BRAD MICHAEL WILCOX
EXPIRES: 6/30/2024

Ariana Tenold, G.I.T.
Geotechnical Project Manager
atenold@carlsontesting.com

Brad M. Wilcox, P.E., G.E.
Principal Geotechnical Engineer
bwilcox@carlsontesting.com

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ATTACHMENTS

1.0 INTRODUCTION

Carlson Geotechnical (CGT), a division of Carlson Testing, Inc. (CTI), is pleased to submit this report summarizing the results of our preliminary geotechnical investigation for the proposed Baxter Road Apartments project. The proposed site is located at 1709 Baxter Road SE in Salem, Oregon, as shown on the attached Site Location, Figure 1.

1.1 Project Information

CGT developed an understanding of the proposed project based on our correspondence with our client and review of two preliminary site layout plans, prepared by Scott Beck Architecture LLC, dated May 7 and May 29, 2024, respectively. Based on our review, we understand the project is in the preliminary stages of planning, but is anticipated to include:

- Demolition and removal of an existing single family residential structure, shop buildings, and an asphalt-paved driveway.
- Construction of a new apartment complex at the approximate 10.8-acre site. Current plans for the complex include the construction of ten multi-story apartment buildings and a clubhouse building. Although no architectural information has been provided, we anticipate the buildings will be two- to three stories, wood-framed, and incorporate slab-on-grade ground floors. No below-grade levels (basements) are anticipated for this project. We have assumed maximum column, continuous wall, and uniform floor slab loads will be on the order of 50 kips, 3 kips per lineal foot (klf), and 150 pounds per square foot (psf), respectively.
- Construction of new private drive lanes and passenger car parking lots to serve the new buildings. We assume the drive lanes and parking areas will be surfaced with asphalt concrete (AC).
- Construction of extensions to three public streets, including Abbie Lane SE, Snowball Avenue SE, and a street yet to be named.
- Installation of appurtenant utilities to serve the new buildings.
- Although no stormwater management plans have been provided, we anticipate stormwater collected from new impervious areas of the site will be routed to the nearest storm drain or other suitable discharge point. Infiltration testing was not requested as part of this assignment.
- Although no grading plans have been provided, we anticipate permanent grade changes at the site will include cuts and fills up to about 5 feet relative to existing grades.

1.2 Scope of Services

Our scope of work included the following:

- Contact the Oregon Utilities Notification Center to mark the locations of public utilities within a 20-foot radius of our explorations at the site.
- Explore subsurface conditions at the site by observing the excavation of thirteen test pits to depths of up to 10 feet below ground surface (bgs). As requested by our client, the test pits were advanced within the footprint of each of the proposed apartment buildings, where access allowed. Details of the subsurface investigation are presented in Appendix A.
- Classify the soils encountered in the explorations in general accordance with ASTM D2488 (Visual-Manual Procedure).
- Provide a technical narrative describing surface and subsurface deposits, and local geology of the site, based on the results of our explorations and published geologic mapping.

- Provide recommendations for the Seismic Site Class, mapped maximum considered earthquake spectral response accelerations, and site seismic coefficients.
- Provide a qualitative evaluation of seismic hazards at the site, including earthquake-induced liquefaction, landsliding, and surface rupture due to faulting or lateral spread.
- Provide geotechnical recommendations for site preparation and earthwork.
- Provide *preliminary* geotechnical engineering recommendations for use in design and construction of shallow foundations, floor slabs, retaining walls, and pavements.
- Provide this written report summarizing the results of our geotechnical investigation and our preliminary recommendations for the project.

This report is considered preliminary, as we have not reviewed final grading plans, finished floor elevations, or detailed structural information for the proposed development. The recommendations presented later in this report for use in design of planned structures are presented on a preliminary basis, and may be considered final once we have had a chance to review finalized earthwork and structural plans.

2.0 SITE DESCRIPTION

2.1 Site Geology

Based on available geologic mapping[1] of the area, the site is underlain by Columbia River Basalt (Tcr), which consists of numerous fine-grained lava flows that primarily erupted from fissures in eastern Washington and Oregon and western Idaho during the Miocene (23.8 to 5.3 million years ago). Many individual flows are interbedded with thin paleosols that consist of clay-rich soils or sediments formed during periods of volcanic inactivity. The basalt, which has a flow thickness between 40 and 100 feet thick, features jointed patterns ranging from columnar to entablature/colonnade, and is described as having fresh exposures that are dark gray to black, while weathered exposures are red-brown to gray-brown. The basalt is variably weathered, producing a red clay/silt laterite layer that is 30 to as much as 175 feet thick in the vicinity of the site.

2.2 Site Surface Conditions

The site consists of one tax lot totaling approximately 10.8 acres. The site was bordered by residential properties to the north, east, and west, and by South Baxter Road to the south. At the time of our field investigation, the site was occupied by a residential structure, a few outbuildings, and an asphalt-paved driveway. The remainder of the site was surfaced with short grasses and several moderate- to large deciduous trees. In terms of topography, the west central portion of the site (encompassing the existing residence and outbuildings) was relatively level while the remainder of the site gently descended to the north and east. Site layout and surface conditions at the time of our field investigation are shown on the attached Site Plan (Figure 2) and Site Photographs (Figure 3).

2.3 Subsurface Conditions

2.3.1 Subsurface Investigation & Laboratory Testing

Our subsurface investigation consisted of thirteen test pits (TP-1 through TP-13) completed on June 20, 2024. The approximate exploration locations are shown on the Site Plan, attached as Figure 2. In summary, the test pits were excavated to depths ranging from about 8 to 10 feet bgs. Details regarding the subsurface

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[1] Bela, J.L., 1981, Geology of the Rickreall, Salem West, Monmouth, and Sidney 7.5-minute Quadrangles, Oregon Department of Geology and Mineral Industries, GMS 18.

investigation, logs of the explorations, and results of laboratory testing are presented in Appendix A. Subsurface conditions encountered during our investigation are summarized below.

2.3.2 Subsurface Materials

The following describes each of the subsurface materials encountered at the site.

Organic Soil (OL)
Organic soil (topsoil) was encountered at the surface of all thirteen test pits and extended to about ¼-foot bgs. This soil was typically brown, dry, exhibited low plasticity, and contained abundant rootlets.

Elastic Silt (MH)
Underlying the topsoil in all thirteen test pits was native, elastic silt. This soil was medium stiff to very stiff, red with some yellow/gray mottling, moist, exhibited medium to high plasticity, and contained a varying amount of weathered basalt gravel and cobbles up to 10-inches in diameter. This soil extended the full depths explored in the test pits, up to 10 feet bgs. This soil is interpreted to consist of residual soil from the in-place weathering of basalt bedrock.

2.3.3 Groundwater

We did not encounter groundwater within the depths explored at the site on June 20, 2024. To determine approximate regional groundwater levels in the area, we researched well logs available on the Oregon Water Resources Department (OWRD)[2] website for wells located within Section 14, Township 8 South, Range 3 West, Willamette Meridian. Our review indicated that groundwater levels in the area generally ranged from about 67 to 97 feet bgs. It should be noted groundwater levels vary with local topography. In addition, the groundwater levels reported on the OWRD logs often reflect the purpose of the well, so water well logs may only report deeper, confined groundwater, while geotechnical or environmental borings will often report any groundwater encountered, including shallow, unconfined groundwater. Therefore, the levels reported on the OWRD well logs referenced above are considered generally indicative of local water levels and may not reflect actual groundwater levels at the project site. We anticipate that groundwater levels will fluctuate due to seasonal and annual variations in precipitation, changes in site utilization, or other factors. Additionally, the on-site, elastic silt is conducive to formation of perched groundwater.

3.0 SEISMIC CONSIDERATIONS

3.1 Seismic Design

Section 1613.2.2 of the 2022 Oregon Structural Specialty Code (2022 OSSC) requires that the determination of the seismic site class be in accordance with Chapter 20 of the American Society of Civil Engineers Minimum Design Loads for Buildings and Other Structures (ASCE 7-16). We have assigned the site as Site Class D ("Stiff Soil") based on geologic mapping and subsurface conditions encountered during our investigation.

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[2] Oregon Water Resources Department, 2024. Well Log Records, *accessed July 2024*, from OWRD web site: http://apps.wrd.state.or.us/apps/gw/well_log/.

Earthquake ground motion parameters for the site were obtained in accordance with the 2022 OSSC using the Seismic Hazards by Location calculator on the ATC website[3]. The site Latitude 44.87853363° North and Longitude 123.021754° West were input as the site location. The following table shows the recommended seismic design parameters for the site.

Table 1 Seismic Ground Motion Values

Parameter		Value
Mapped Acceleration Parameters	Spectral Acceleration, 0.2 second (S_s)	0.813g
	Spectral Acceleration, 1.0 second (S_1)	0.411g
Coefficients (Site Class D)	Site Coefficient, 0.2 second (F_A)	1.175
	Site Coefficient, 1.0 second (F_V)[1]	1.889
Adjusted MCE Spectral Response Parameters	MCE Spectral Acceleration, 0.2 second (S_{MS})	0.955g
	MCE Spectral Acceleration, 1.0 second (S_{M1})	0.776g
Design Spectral Response Accelerations	Design Spectral Acceleration, 0.2 second (S_{DS})	0.637g
	Design Spectral Acceleration, 1.0 second (S_{D1})	0.517g
Seismic Design Category (Risk Category II)		D

[1] Value determined from 2022 OSSC Table 1613.2.3(2).

3.2 Seismic Hazards

3.2.1 Liquefaction

In general, liquefaction occurs when deposits of loose/soft, saturated, cohesionless soils, generally sands and silts, are subjected to strong earthquake shaking. If these deposits cannot drain quickly enough, pore water pressures can increase, approaching the value of the overburden pressure. The shear strength of a cohesionless soil is directly proportional to the effective stress, which is equal to the difference between the overburden pressure and the pore water pressure. When the pore water pressure increases to the value of the overburden pressure, the shear strength of the soil approaches zero, and the soil can liquefy. The liquefied soils can undergo rapid consolidation or, if unconfined, can flow as a liquid. Structures supported by the liquefied soils can experience rapid, excessive settlement, shearing, or even catastrophic failure.

For fine-grained soils, susceptibility to liquefaction is evaluated based on penetration resistance and plasticity, among other characteristics. Criteria for identifying non-liquefiable, fine-grained soils are constantly evolving. Current practice to identify non-liquefiable, fine-grained soils is based on moisture content and plasticity characteristics of the soils[4,5,6]. The susceptibility of sands, gravels, and sand-gravel mixtures to liquefaction is typically assessed based on penetration resistance, as measured using SPTs, CPTs, or Becker Hammer Penetration tests (BPTs).

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[3] Applied Technology Council (ATC), 2024. USGS seismic design parameters determined using “Seismic Hazards by Location,” *accessed July 2024*, from the ATC website https://hazards.atcouncil.org/.

[4] Seed, R.B. et al., 2003. Recent Advances in Soil Liquefaction Engineering: A Unified and Consistent Framework. Earthquake Engineering Research Center Report No. EERC 2003-06.

[5] Bray, Jonathan D., Sancio, Rodolfo B., et al., 2006. Liquefaction Susceptibility of Fine-Grained Soils, Journal of Geotechnical and Geoenvironmental Engineering, Volume 132, Issue 9, September 2006.

[6] Idriss, I.M., Boulanger, R.W., 2008. Soil Liquefaction During Earthquakes, Earthquakes Engineering Research Institute Monograph MNO-12.

The Oregon Department of Geology and Mineral Industries' Oregon Statewide Geohazards Viewer (HazVu)[7] shows a *low* hazard for liquefaction at the site. Based on their plasticity characteristics and lack of saturated conditions, the silty soils encountered within our explorations are considered non-liquefiable. Based on review of geologic mapping and our previous experience in the area, we do not anticipate liquefiable conditions are present at depths below those explored as part of this assignment.

3.2.2 Slope Instability

Due to the relatively level to gently sloped topography at and surrounding the site, the risk of slope instability at the site is considered low. Provided the recommendations presented later in this report regarding grading and drainage are incorporated into design and construction, the proposed development is not anticipated to increase the hazard associated with seismically-induced slope instability.

3.2.3 Surface Rupture

3.2.3.1 Faulting

Although the site is situated in a region of the country with known active faults and historic seismic activity, no known faults exist on or immediately adjacent to the site. Therefore, the risk of surface rupture at the site due to faulting is considered low.

3.2.3.2 Lateral Spread

Surface rupture due to lateral spread can occur on sites underlain by liquefiable soils that are located on or immediately adjacent to slopes steeper than about 3 degrees (20H:1V), and/or adjacent to a free face, such as a stream bank or the shore of an open body of water. During lateral spread, the materials overlying the liquefied soils are subject to lateral movement downslope or toward the free face. Based on the non-liquefiable nature of the site soils, the risk of damage associated with lateral spread at the site is negligible.

4.0 CONCLUSIONS

Based on the results of our field explorations and analyses, the site may be developed as described in Section 1.1 of this report, provided the recommendations presented in this report are incorporated into the design and development. Satisfactory subgrade support for planned shallow foundations, floor slabs, retaining walls and pavements can be achieved by the native, medium stiff to better, elastic silt (MH) or structural fill that is properly placed and compacted on that material during construction. This soil was encountered at depths of about ¼ foot bgs within the test pits.

The near surface silty soils are susceptible to disturbance during wet weather. Trafficability of these soils may be difficult, and significant damage to the subgrade could occur, if earthwork is undertaken without proper precautions at times when the exposed soils are more than a few percentage points above optimum moisture content. In the event that construction occurs during wet weather, CGT recommends that measures be implemented to protect the fine-grained subgrade in areas of repeated construction traffic and within footing excavations. Geotechnical recommendations for wet weather construction are presented in Section 5.3 of this report. Re-use of the on-site silty soils as structural fill during wet weather conditions is anticipated to be very difficult as discussed in Section 5.4.1.1 of this report.

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[7] Oregon Department of Geology and Mineral Industries, 2024. Oregon Statewide Geohazards Viewer, *accessed July 2024*, from DOGAMI web site: http://www.oregongeology.org/sub/hazvu/index.htm.

5.0 PRELIMINARY RECOMMENDATIONS

The recommendations presented in this report are based on the information provided to us, results of our field investigation and analyses, laboratory data, and professional judgment. CGT has observed only a small portion of the pertinent subsurface conditions. The recommendations are based on the assumptions that the subsurface conditions do not deviate appreciably from those found during the field investigation. CGT should be consulted for further recommendations if the design of the proposed development changes and/or variations or undesirable geotechnical conditions are encountered during site development.

5.1 Site Preparation

5.1.1 Demolition

Demolition of the existing buildings and appurtenant structures should include complete removal of all structural elements, including foundations and concrete slabs. Abandoned buried utilities should similarly be removed or grouted full. Concrete or asphalt concrete debris resulting from demolition activities should be hauled off site for disposal.

5.1.2 Stripping

Existing vegetation, topsoil, rooted soils, and undocumented fill soils (where/if encountered) should be removed from within, and for a minimum 5-foot margin around, the proposed building pads, structural fill, and pavement areas. Based on the results of our field explorations, topsoil stripping depths are anticipated to be on the order of about ¼ foot bgs. These materials may be deeper or shallower at locations away from the completed explorations. The geotechnical engineer’s representative should provide recommendations for actual stripping depths based on observations during site stripping. Stripped surface vegetation, rooted soils and fill materials should be transported off-site for disposal, or stockpiled for later use in landscaped areas.

5.1.3 Grubbing

Grubbing of trees should include the removal of the root mass and roots greater than ½ inch in diameter. Grubbed materials should be transported off-site for disposal. Root masses from larger trees may extend greater than 3 feet bgs. Where root masses are removed, the resulting excavation should be properly backfilled with structural fill in conformance with Section 5.4 of this report.

5.1.4 Test Pit Backfills

The test pits conducted at the site were loosely backfilled during our field investigation. Where test pits are located within finalized building, structural fill, or pavement areas, the loose backfill materials should be re-excavated. The resulting excavations should be backfilled with structural fill in conformance with Section 5.4 of this report.

5.1.5 Existing Utilities & Below-Grade Structures

All existing utilities at the site should be identified prior to excavation. Abandoned utility lines beneath the new buildings, pavements, and hardscaping features should be completely removed or grouted full. Soft, loose, or otherwise unsuitable soils encountered in utility trench excavations should be removed and replaced with structural fill in conformance with Section 5.4 this report. Buried structures (i.e. footings, foundation walls, retaining walls, slabs-on-grade, tanks, etc.), if encountered during site development, should be completely removed and replaced with structural fill in conformance with Section 5.4 of this report.

5.1.6 Subgrade Preparation – Building Pads, Private Pavements & Areas to Receive Structural Fill

After site preparation as recommended above, but prior to placement of structural fill and/or aggregate base, the geotechnical engineer's representative should observe the exposed subgrade soils in order to identify areas of excessive yielding through either proof rolling or probing. Proof rolling of subgrade soils is typically conducted during dry weather using a fully-loaded, 10- to 12-cubic-yard, tandem-axle, tire-mounted, dump truck or equivalent weighted water truck. Areas of limited access or that appear too soft or wet to support proof rolling equipment should be evaluated by probing. During wet weather, subgrade preparation should be performed in general accordance with the recommendations presented in Section 5.3 of this report. If areas of soft soil or excessive yielding are identified, the affected material should be over-excavated to firm, unyielding subgrade, and replaced with imported granular structural fill in conformance with Section 5.4.2 of this report.

The elastic silt (MH) soils should be kept moist, near optimum moisture content, and not allowed to dry out. If allowed to dry below optimum moisture content, to a point where surface cracking appears in the subgrade, the affected material should be over-excavated and replaced with imported granular structural fill.

5.1.7 Erosion Control

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

5.2 Temporary Excavations

5.2.1 Overview

Conventional earthmoving equipment in proper working condition should be capable of making necessary excavations for the anticipated site cuts as described earlier in this report. All excavations should be in accordance with applicable OSHA and state regulations. It is the contractor's responsibility to select the excavation methods, to monitor site excavations for safety, and to provide any shoring required to protect personnel and adjacent improvements. A "competent person," as defined by OR-OSHA, should be on-site during construction in accordance with regulations presented by OR-OSHA. CGT's current role on the project does not include review or oversight of excavation safety.

5.2.2 OSHA Soil Type

For use in the planning and construction of temporary excavations up to 10 feet in depth, an OSHA soil type "A" may be used for the native elastic silt (MH) encountered within our explorations.

5.2.3 Utility Trenches

Temporary trench cuts should stand near vertical to depths of approximately 4 feet in the native, silty soils (MH) encountered near the surface of the site. If groundwater seepage undermines the stability of the trench, or if sidewall caving is observed during excavation, the sidewalls should be flattened or shored. Depending on the time of year trench excavations occur, trench dewatering may be required in order to maintain dry working conditions. Pumping from sumps located within the trench will likely be effective in removing water resulting from seepage. If groundwater is encountered, we recommend placing trench stabilization material at the base of the excavations. Trench stabilization material should be in conformance with Section 5.4.3.

5.2.4 Excavations Near Foundations

Excavations near footings should not extend within a 1 horizontal to 1 vertical (1H:1V) plane projected out and down from the outside, bottom edge of the footings. In the event excavation needs to extend below the referenced plane, temporary shoring of the excavation and/or underpinning of the subject footing may be required. The geotechnical engineer should be consulted to review proposed excavation plans for this design case to provide specific recommendations.

5.3 Wet Weather Considerations

For planning purposes, the wet season should be considered to extend from late September to late June. It is our experience that dry weather working conditions should prevail between early July and mid-September. Notwithstanding the above, soil conditions should be evaluated in the field by the geotechnical engineer's representative at the initial stage of site preparation to determine whether the recommendations within this section should be incorporated into construction.

5.3.1 Overview

As mentioned above, due to its fines content, the on-site elastic silt (MH) is susceptible to disturbance during wet weather. Trafficability of these soils may be difficult, and significant damage to subgrade soils could occur, if earthwork is undertaken without proper precautions at times when the exposed soils are more than a few percentage points above optimum moisture content. For wet weather construction, site preparation activities may need to be accomplished using track-mounted equipment, loading removed material onto trucks supported on granular haul roads, or other methods to limit soil disturbance. The geotechnical engineer's representative should evaluate the subgrade during excavation by probing rather than proof rolling. Soils that have been disturbed during site preparation activities, or soft or loose areas identified during probing, should be over-excavated to firm, unyielding subgrade, and replaced with imported granular structural fill in conformance with Section 5.4.2.

5.3.2 Geotextile Separation Fabric

We recommend a geotextile separation fabric be placed to serve as a barrier between the prepared subgrade and granular fill/base rock in areas of repeated or heavy construction traffic. The geotextile fabric should meet the requirements presented in the current Oregon Department of Transportation (ODOT) Standard Specification for Construction (ODOT SSC), Section 02320.

5.3.3 Granular Working Surfaces (Haul Roads & Staging Areas)

Haul roads subjected to repeated heavy, tire-mounted, construction traffic (e.g. dump trucks, concrete trucks, etc.) will require a minimum of 18 inches of imported granular material. For light staging areas, 12 inches of imported granular material is typically sufficient. Additional granular material, cement amendment, or geo-grid reinforcement may be recommended based on site conditions and/or loading at the time of construction. The imported granular material should be in conformance with Section 5.4.2 and have less than 5 percent material passing the U.S. Standard No. 200 Sieve. The prepared subgrade should be covered with geotextile fabric (Section 5.3.2) prior to placement of the imported granular material. The imported granular material should be placed in a single lift (up to 24 inches deep) and compacted using a smooth-drum, non-vibratory roller until well-keyed.

5.3.4 Cement Amendment

It is sometimes less costly to amend near-surface, moisture-sensitive, fine-grained soils with Portland cement than to remove and replace those soils with imported granular material. Successful use of soil cement amendment depends on use of correct techniques and equipment, soil moisture content, and the amount of cement added to the subgrade (mix design). We anticipate the on-site native clayey soils (CH) are conducive for cement amendment due to their generally moderate plasticity and experience with similar soils.

The recommended percentage of cement is based on soil moisture contents at the time the work is performed. Based on our experience, 3 percent cement by weight of dry soil can generally be used when the soil moisture content does not exceed approximately 20 percent. If the soil moisture content is in the range of 25 to 35 percent, 4 to 6 percent by weight of dry soil is recommended. Similarly, if the soil moisture content is in the range of 35 to 45 percent, 7 to 8 percent by weight of dry soil is recommended. It is difficult to accurately predict field performance due to the variability in soil response to cement amendment. The amount of cement added to the soil may need to be adjusted based on field observations and performance.

If cement amendment is considered, we recommend additional sampling, laboratory testing, and a mix design be performed to determine the level of improvement in engineering properties (strength, stiffness) of the on-site soils when blended with Portland cement. We recommend project scheduling allow for a minimum of 4 weeks for this testing and design to be completed, prior to initiating cement amendment.

5.3.5 Footing Subgrade Protection

A minimum of 3 inches of imported granular material (crushed rock) is recommended to protect fine-grained (silty), footing subgrades from foot traffic during inclement weather. The imported granular material should be in conformance with Section 5.4.2. The maximum particle size should be limited to 1 inch. The imported granular material should be placed in one lift over the prepared, undisturbed subgrade, and compacted using non-vibratory equipment until well keyed.

Surface water should not be allowed to collect in footing excavations. The excavations should be draped and/or provided with sumps to preclude water accumulation during inclement weather.

5.4 Structural Fill

The geotechnical engineer should be provided the opportunity to review all materials considered for use as structural fill (prior to placement). Samples of the proposed fill materials should be submitted to the geotechnical engineer a minimum of 5 business days prior their use on site[8]. The geotechnical engineer's representative should be contacted to evaluate compaction of structural fill as the material is being placed. Evaluation of compaction may take the form of in-place density tests and/or proof roll tests with suitable equipment. Structural fill should be evaluated at intervals not exceeding every 2 vertical feet as the fill is being placed.

5.4.1 On-Site Soils – General Use

5.4.1.1 Elastic Silt (MH)

Re-use of this soil as structural fill may be difficult because this soil is sensitive to small changes in moisture content and is difficult, if not impossible, to adequately compact during wet weather. We anticipate the

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[8] Laboratory testing for moisture density relationship (Proctor) is required. Tests for gradation may be required.

moisture content of this soil will be higher than the optimum moisture content for satisfactory compaction. Therefore, moisture conditioning (drying) should be expected in order to achieve adequate compaction. If used as structural fill, this soil should be free of organic matter, debris, and particles larger than 4 inches. When used as structural fill, this soil should be placed in lifts with a maximum pre-compaction thickness of about 8 inches at moisture contents within –1 and +3 percent of optimum, and compacted to not less than 92 percent of the material's maximum dry density, as determined in general accordance with ASTM D1557 (Modified Proctor).

If the on-site materials cannot be properly moisture-conditioned and/or processed, we recommend using imported granular material for structural fill.

5.4.2 Imported Granular Structural Fill – General Use

Imported granular structural fill should consist of angular pit or quarry run rock, crushed rock, or crushed gravel that is fairly well graded between coarse and fine particle sizes. The granular fill should contain no organic matter, debris, or particles larger than 4 inches, and have less than 5 percent material passing the U.S. Standard No. 200 Sieve. For fine-grading purposes, the maximum particle size should be limited to 1½ inches. The percentage of fines can be increased to 12 percent of the material passing the U.S. Standard No. 200 Sieve if placed during dry weather, and provided the fill material is moisture-conditioned, as necessary, for proper compaction. Imported granular fill material should be placed in lifts with a maximum thickness of about 12 inches, and compacted to not less than 95 percent of the material's maximum dry density, as determined in general accordance with ASTM D1557 (Modified Proctor). Proper moisture conditioning and the use of vibratory equipment will facilitate compaction of these materials.

Granular fill materials with high percentages of particle sizes in excess of 1½ inches are considered non-moisture-density testable materials. As an alternative to conventional density testing, compaction of these materials should be evaluated by proof roll test observation (deflection tests), where accepted by the geotechnical engineer.

5.4.3 Trench Base Stabilization Material

If groundwater is present at the base of utility excavations, trench base stabilization material should be placed. Trench base stabilization material should consist of a minimum of 1 foot of well-graded granular material with a maximum particle size of 4 inches and less than 5 percent material passing the U.S. Standard No. 4 Sieve. The material should be free of organic matter and other deleterious material, placed in one lift, and compacted until well-keyed.

5.4.4 Trench Backfill Material

Trench backfill for the utility pipe base and pipe zone should consist of granular material as recommended by the utility pipe manufacturer. Trench backfill above the pipe zone should consist of well-graded granular material containing no organic matter or debris, have a maximum particle size of ¾ inch, and have less than 8 percent material passing the U.S. Standard No. 200 Sieve. As a guideline, trench backfill should be placed in maximum 12-inch-thick lifts. The earthwork contractor may elect to use alternative lift thicknesses based on their experience with specific equipment and fill material conditions during construction in order to achieve the required compaction. The following table presents recommended relative compaction percentages for utility trench backfill.

Table 2 Utility Trench Backfill Compaction Recommendations

Backfill Zone	Recommended Minimum Relative Compaction	
	Structural Areas[1,2]	**Landscaping Areas**
Pipe Base and Within Pipe Zone	90% ASTM D1557 or pipe manufacturer's recommendation	85% ASTM D1557 or pipe manufacturer's recommendation
Above Pipe Zone	92% ASTM D1557	88% ASTM D1557
Within 3 Feet of Design Subgrade	95% ASTM D1557	90% ASTM D1557

[1] Includes proposed buildings, pavement areas, structural fill areas, exterior hardscaping, etc.

[2] Or as specified by the local jurisdiction where located in the public right of way.

5.4.5 Controlled Low-Strength Material (CLSM)

CLSM is a self-compacting, cementitious material that is typically considered when backfilling localized areas. CLSM is sometimes referred to as "controlled density fill" or CDF. Due to its flowable characteristics, CLSM typically can be placed in restricted-access excavations where placing and compacting fill is difficult. If chosen for use at this site, we recommend the CLSM be in conformance with Section 00442 of the most recent, ODOT SSC. The geotechnical engineer's representative should observe placement of the CLSM and obtain samples for compression testing in accordance with ASTM D4832. As a guideline, for each day's placement, two compressive strength specimens from the same CLSM sample should be tested. The results of the two individual compressive strength tests should be averaged to obtain the reported 28-day compressive strength. If CLSM is considered for use on this site, please contact the geotechnical engineer for site-specific and application-specific recommendations.

5.5 Permanent Slopes

Permanent cut or fill slopes constructed at the site, if any, should be graded at 2H:1V or flatter. Constructed slopes should be overbuilt by a few feet depending on their size and gradient so that they can be properly compacted prior to being cut to final grade. The surface of all slopes should be protected from erosion by seeding, sodding, or other acceptable means. Adjacent on-site and off-site structures should be located at least 5 feet from the top of slopes.

5.6 Shallow Foundations

5.6.1 Subgrade Preparation

Satisfactory subgrade support for shallow foundations can be obtained from the native, medium stiff to better elastic silt (MH), or new structural fill that is properly placed and compacted on that material during construction. The geotechnical engineer's representative should be contacted to observe subgrade conditions prior to placement of forms, reinforcement steel, or granular backfill (if required). If soft, loose, or otherwise unsuitable soils are encountered, they should be over-excavated as recommended by the geotechnical representative at the time of construction. The resulting over-excavation should be brought back to grade with imported granular structural fill in conformance with Section 5.4.2. The maximum particle size of over-excavation backfill should be limited to 1½ inches. All granular pads for footings should be constructed a minimum of 6 inches wider on each side of the footing for every vertical foot of over-excavation.

5.6.2 Minimum Footing Width & Embedment

Minimum footing widths should be in conformance with the current OSSC. As a guideline, CGT recommends individual spread footings have a minimum width of 24 inches. For two- and three-story, light framed structures, we recommend continuous wall footings have a minimum width of 15 to 18 inches, respectively. All footings should be founded at least 18 inches below the lowest, permanent adjacent grade to develop lateral capacity and for frost protection.

5.6.3 Horizontal Setback from Descending Slopes

Foundations constructed within or near descending slopes should be setback a minimum of 5 feet from the slope surface. This distance should be measured between the face of the slope and the bottom, outside edge of the respective foundation. Organic topsoil and loose surface soils (if present) should not be included when determining this distance. The geotechnical engineer or his representative should be contacted to observe foundation subgrade conditions and confirm this recommended minimum setback is achieved.

5.6.4 Bearing Pressure & Settlement

Footings founded as recommended above should be proportioned for a maximum allowable soil bearing pressure of 2,500 pounds per square foot (psf). This bearing pressure is a net bearing pressure, applies to the total of dead and long-term live loads, and may be increased by one-third when considering seismic or wind loads. For foundations founded as recommended above, total settlement of foundations is anticipated to be less than 1 inch. Differential settlements between adjacent columns and/or bearing walls should not exceed ½ inch. If an increased allowable soil bearing pressure is desired, the geotechnical engineer should be consulted.

5.6.5 Lateral Capacity

A maximum passive (equivalent fluid) earth pressure of 150 pounds per cubic foot (pcf) is recommended for design of footings cast neat into excavations in suitable native soil or confined by imported granular structural fill that is properly placed and compacted during construction. The recommended earth pressure was computed using a factor of safety of 1½, which is appropriate due to the amount of movement required to develop full passive resistance. In order to develop the above capacity, the following should be understood:

1. Concrete must be poured neat in excavations or the foundations must be backfilled with imported granular structural fill,
2. The adjacent grade must be level,
3. The static ground water level must remain below the base of the footings throughout the year.
4. Adjacent floor slabs, pavements, or the upper 12-inch-depth of adjacent, unpaved areas should not be considered when calculating passive resistance.

An ultimate coefficient of friction equal to 0.35 may be used when calculating resistance to sliding for footings founded on the native soils described above. An ultimate coefficient of friction equal to 0.45 may be used when calculating resistance to sliding for footings founded on a minimum of 6 inches of imported granular structural fill (crushed rock) that is properly placed and compacted during construction.

5.6.6 Subsurface Drainage

Recognizing the fine-grained (silty) soils encountered at this site, we recommend placing foundation drains at the exterior, base elevations of perimeter continuous wall footings. Foundation drains should consist of a minimum 4-inch diameter, perforated, PVC drainpipe wrapped with a non-woven geotextile filter fabric. The drains should be backfilled with a minimum of 2 cubic feet of open graded drain rock per lineal foot of pipe. The drain rock should also be encased in a geotextile fabric in order to provide separation from the surrounding fine-grained soils. Foundation drains should be positively sloped and should outlet to a suitable discharge point. The geotechnical engineer's representative should observe the drains prior to backfilling. Roof drains should not be tied into foundation drains.

5.7 Rigid Retaining Walls

5.7.1 Footings

Retaining wall footings should be designed and constructed in conformance with the recommendations presented in Section 5.6, as applicable.

5.7.2 Wall Drains

We recommend placing retaining wall drains at the base elevation of the heel of retaining wall footings. Retaining wall drains should consist of a minimum 4-inch-diameter, perforated, HDPE (High Density Polyethylene) drainpipe wrapped with a non-woven geotextile filter fabric. The drains should be backfilled with a minimum of 2 cubic feet of open graded drain rock per lineal foot of pipe. The drain rock should be encased in a geotextile fabric in order to provide separation from the surrounding soils. Retaining wall drains should be positively sloped and should outlet to a suitable discharge point. The geotechnical engineer's representative should be contacted to observe the drains prior to backfilling. Roof or area drains should not be tied into retaining wall drains.

5.7.3 Wall Backfill

Retaining walls should be backfilled with imported granular structural fill in conformance with Section 5.4.2 and contain less than 5 percent passing the U.S. Standard No. 200 Sieve. The backfill should be compacted to a minimum of 90 percent of the material's maximum dry density as determined in general accordance with ASTM D1557 (Modified Proctor). When placing fill behind walls, care must be taken to minimize undue lateral loads on the walls. Heavy compaction equipment should be kept at least "H" feet from the back of the walls, where "H" is the height of the wall. Light mechanical or hand tamping equipment should be used for compaction of backfill materials within "H" feet of the back of the walls.

5.7.4 Design Parameters & Limitations

For rigid retaining walls founded, backfilled, and drained as recommended above, the following table presents parameters recommended for design.

Table 3 Design Parameters for Rigid Retaining Walls

Retaining Wall Condition	Modeled Backfill Condition	Static Equivalent Fluid Pressure (S_A)[1]	Seismic Equivalent Fluid Pressure (S_{AE}) [1,2]	Surcharge from Uniform Load, q, Acting on Backfill Behind Retaining Wall
Not Restrained from Rotation	Level (i=0)	28 pcf	38 pcf	0.22*q
Restrained from Rotation	Level (i=0)	50 pcf	51 pcf	0.38*q

[1] Refer to the attached Figure 4 for a graphical representation of static and seismic loading conditions. Seismic resultant force acts at 0.6H above the base of the wall.

[2] Seismic (dynamic) lateral loads were computed using the Mononobe-Okabe Equation as presented in the 1997 Federal Highway Administration (FHWA) design manual. Static and seismic equivalent fluid pressures are not additive.

The above design recommendations are based on the assumptions that:

- The walls consist of concrete cantilevered retaining walls ($\beta = 0$ and $\delta = 24$ degrees, see Figure 4).
- The walls are 10 feet or less in height.
- The backfill is drained and consists of imported granular structural fill ($\phi = 38$ degrees).
- No point, line, or strip load surcharges are imposed behind the walls.
- The grade behind the wall is level, or sloping down and away from the wall, for a distance of 10 feet or more from the wall.
- The grade in front of the walls is level or ascending for a distance of at least 5 feet from the wall.

Re-evaluation of our recommendations will be required if the retaining wall design criteria for the project vary from these assumptions.

5.7.5 Surcharge Loads

Where present, surcharges from adjacent site features (i.e. buildings, slabs, pavements, etc.) should be evaluated in design of retaining walls at the site. Methods for calculating lateral pressures on rigid retaining walls from strip, line, and vertical point loads are presented on the attached Figure 5.

5.8 Floor Slabs

5.8.1 Subgrade Preparation

Satisfactory subgrade support for slabs constructed on grade, supporting up to 150 psf area loading, can be obtained from the native, medium stiff to better elastic silt (MH), or new structural fill that is properly placed and compacted on that material during construction. The geotechnical engineer's representative should observe floor slab subgrade soils to evaluate surface consistencies. If soft, loose, or otherwise unsuitable soils are encountered, they should be over-excavated as recommended by the CGT geotechnical representative at the time of construction. The resulting over-excavation should be brought back to grade with imported granular structural fill as described in Section 5.4.2.

5.8.2 Crushed Rock Base

Concrete floor slabs should be supported on a minimum 6-inch-thick layer of crushed rock (base rock).

5.8.2.1 Conventional Base Rock

Floor slab base rock should consist of well-graded granular material (crushed rock) containing no organic matter or debris, have a maximum particle size of ¾ inch, and have less than 5 percent material passing the U.S. Standard No. 200 Sieve. Floor slab base rock should be placed in one lift and compacted to not less than 95 percent of the material's maximum dry density as determined in general accordance with ASTM D1557 (Modified Proctor). We recommend "choking" the surface of the base rock with sand just prior to concrete placement. Choking means the voids between the largest aggregate particles are filled with sand, but does not provide a layer of sand above the base rock. Choking the base rock surface reduces the lateral restraint on the bottom of the concrete during curing. Choking the base rock also reduces punctures in vapor retarding membranes due to foot traffic where such membranes are used.

5.8.2.2 Gas Permeable Base Rock

Floor slab base rock in areas where radon gas mitigation is desired should consist of open-graded crushed rock containing no organic matter or debris, with all material passing through a 2-inch sieve and retained on the ¼-inch sieve, in accordance with Section 1812.2.1, Bullet 1, of the 2022 OSSC.

CGT recommends that a minimum 10-mil polyethylene sheeting or equivalent material with equal or greater tensile strength, resistance to puncture, resistance to deterioration, and resistance to water-vapor transmission be placed on top of the gas-permeable base rock to act as a soil-gas-retarder. Placement and installation of this sheeting should be in conformance with that indicated in Section 1812.2.2 of the 2022 OSSC.

The geotechnical engineer or their representative should be contacted to observe gas-permeable base rock conditions prior to placement of the soil-gas-retarder.

5.8.3 Design Considerations

For floor slabs constructed with a 6-inch thick base rock layer as recommended, an effective modulus of subgrade reaction of 150 pounds per cubic inch (pci) is recommended for the design of the floor slab. A higher effective modulus of subgrade reaction can be obtained by increasing the base rock thickness. Please contact the geotechnical engineer for additional recommendations if a higher modulus is desired. Floor slabs constructed as recommended will likely settle less than ½ inch. For general floor slab construction, slabs should be jointed around columns and walls to permit slabs and foundations to settle differentially.

5.8.4 Subgrade Moisture Considerations

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

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

5.9 Pavements

5.9.1 Subgrade Preparation

5.9.1.1 Private Pavements

Pavement subgrade preparation should be performed in general accordance with the recommendations presented in Section 5.1.6 above. Subgrade surfaces should be crowned (or sloped) for proper drainage in accordance with specifications provided by the project civil engineer.

5.9.1.2 Public Pavements

Pavement subgrade preparation for public pavements (i.e., pavements in the public right-of-way) should be in conformance with the approved civil plans, or as specified by the City of Salem.

5.9.2 Pavement Sections

Pavement section design was not included as part of this assignment, but can be provided, upon request, for an additional fee.

5.10 Additional Considerations

5.10.1 Drainage

Subsurface drains should be connected to the nearest storm drain, or other suitable discharge point. Paved surfaces and grading near or adjacent to the buildings should be sloped to drain away from the buildings. Surface water from paved surfaces and open spaces should be collected and routed to a suitable discharge point. Surface water should not be directed into foundation drains, retaining wall drains, or onto site slopes.

5.10.2 Expansive Potential

The near surface native soils consist of generally moderate plasticity silty soils. Based on our experience with similar soils in the vicinity of the site, these soils are not considered to be susceptible to appreciable movements from changes in moisture content. Accordingly, no special considerations are required to mitigate expansive potential of the near surface soils at the site.

6.0 RECOMMENDED ADDITIONAL SERVICES

6.1 Design Review

Geotechnical design review is of paramount importance. We recommend the geotechnical design review take place prior to releasing bid packets to contractors.

6.2 Observation of Construction

Satisfactory earthwork, foundation, floor slab, and pavement performance depends to a large degree on the quality of construction. Sufficient observation of the contractor’s activities is a key part of determining that the work is completed in accordance with the construction drawings and specifications. Subsurface conditions observed during construction should be compared with those encountered during subsurface explorations, and recognition of changed conditions often requires experience. We recommend that qualified personnel visit the site with sufficient frequency to detect whether subsurface conditions change significantly from those observed to date and anticipated in this report. We recommend geotechnical engineer's representative attend a pre-construction meeting coordinated by the contractor and/or developer. The project geotechnical engineer's representative should provide observations and/or testing of at least the following earthwork elements during construction:

- Site Stripping and Demolition
- Subgrade Preparation for Shallow Foundations, Retaining Walls, Structural Fills, Floor Slabs, and Pavements
- Compaction of Structural Fill, Retaining Wall Backfill, and Utility Trench Backfill
- Compaction of Base Rock for Floor Slabs and Pavements
- Compaction of Asphalt Concrete for Pavements

It is imperative that the owner and/or contractor request earthwork observations and testing at a frequency sufficient to allow the geotechnical engineer to provide a final letter of compliance for the earthwork activities.

7.0 LIMITATIONS

We have prepared this report for use by the owner/developer and other members of the design and construction team for the proposed development. The opinions and recommendations contained within this report are forwarded to assist in the planning and design process and are not intended to be, nor should they be construed as, a warranty of subsurface conditions.

We have made observations based on our explorations that indicate the soil conditions at only those specific locations and only to the depths penetrated. These observations do not necessarily reflect soil types, strata thickness, or water level variations that may exist between or away from our explorations. If subsurface conditions vary from those encountered in our site explorations, CGT should be alerted to the change in conditions so that we may provide additional geotechnical recommendations, if necessary. Observation by experienced geotechnical personnel should be considered an integral part of the construction process.

The owner/developer is responsible for ensuring that the project designers and contractors implement our recommendations. When the design has been finalized, prior to releasing bid packets to contractors, we recommend that the design drawings and specifications be reviewed by our firm to see that our recommendations have been interpreted and implemented as intended. If design changes are made, we request that we be retained to review our conclusions and recommendations and to provide a written modification or verification. Design review and construction phase testing and observation services are beyond the scope of our current assignment, but will be provided for an additional fee.

The scope of our services does not include services related to construction safety precautions, and our recommendations are not intended to direct the contractor's methods, techniques, sequences, or procedures, except as specifically described in our report for consideration in design.

Geotechnical engineering and the geologic sciences are characterized by a degree of uncertainty. Professional judgments presented in this report are based on our understanding of the proposed construction, familiarity with similar projects in the area, and on general experience. Within the limitations of scope, schedule, and budget, our services have been executed in accordance with the generally accepted practices in this area at the time this report was prepared; no warranty, expressed or implied, is made. This report is subject to review and should not be relied upon after a period of three years.

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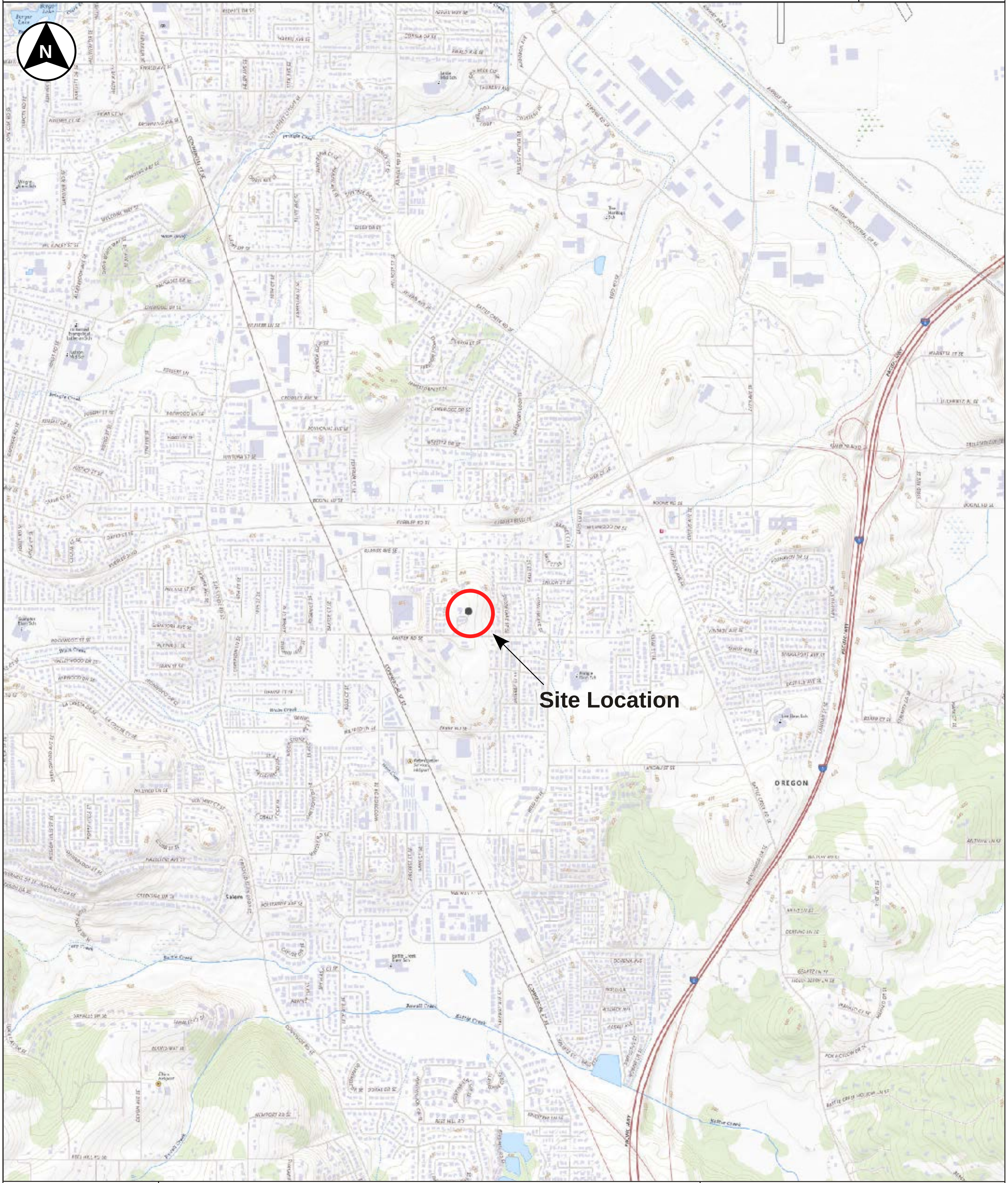
FIGURE 1
Site Location



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Drafted by:AET

USGS Topographic base map created with The National Map, 2024, at https://apps.nationalmap.gov/viewer/

Township 8 South, Range 4 West, Section 14, Willamette Meridian

Latitude: 44.87853363° North
Longitude: 123.021754º West

1 Inch = 2,000 feet

0 2000 4000

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FIGURE 2

Site Plan

LEGEND

TP-1 ▨ Test pit exploration

◁ 1 Orientation of site photographs shown on Figure 3

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Drafted by: AET

NOTES: 2023 aerial photograph from City of Salem Mapping System https://salem.maps.arcgis.com/. All locations are approximate.

1 Inch = 100 Feet

0 100 200

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FIGURE 3

Site Photographs

Photograph 1

Photograph 2

Photograph 3

Photograph 4

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Drafted by: BJG

See Figure 2 for approximate photograph locations and directions. Photographs were taken at the time of our fieldwork.

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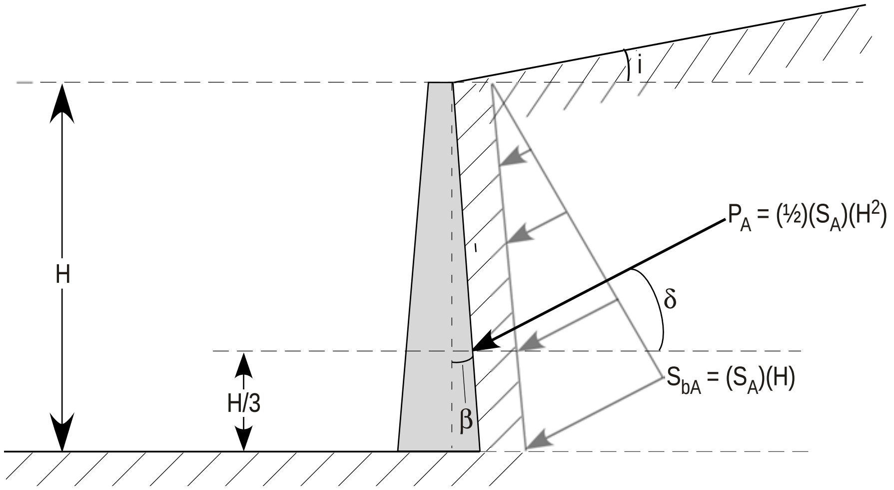
FIGURE 4

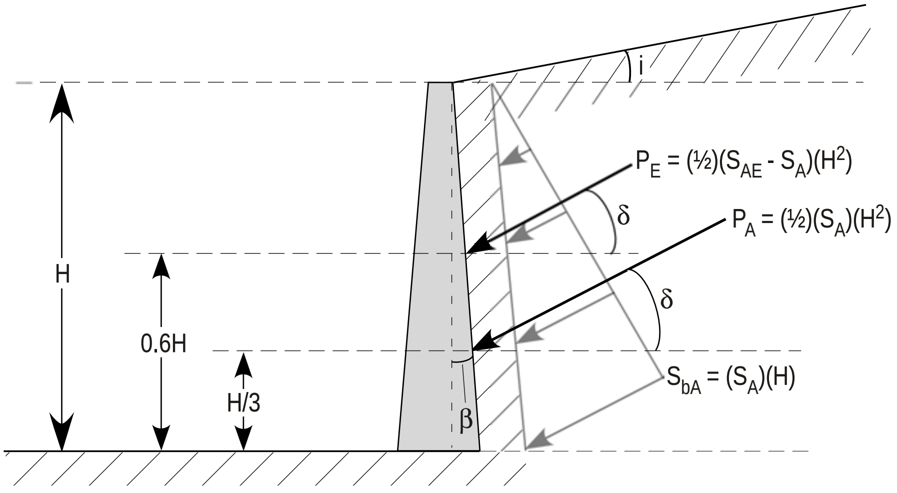
Retaining Walls

ACTIVE LATERAL PRESSURE DISTRIBUTION

STATIC LOADING CONDITIONS



SEISMIC LOADING CONDITIONS



LEGEND

S_A = Active lateral equivalent fluid pressure (lb/ft^3)*

S_{bA} = Active lateral earth pressure (static) at the bottom of wall (lb/ft^3)

S_{AE} = Active total (static + seismic) equivalent fluid pressure (lb/ft^3)*

i = Slope of backfill, relative to horizontal (degrees)**

β = Slope of back of wall, relative to vertical (degrees)**

P_A = Static active thrust force acting at H/3 from bottom of retaining wall (lb/ft)

P_E = Dynamic active thrust force acting at 0.6H from bottom of retaining wall (lb/ft)

δ = Angle from normal of back of wall (degrees). Based on friction developing between wall and backfill**

*Refer to report text for calculated values **Refer to report text for modeled/assumed values

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Notes

1. Uniform pressure distribution of seismic loading is based on empirical evaluations [Sherif et al, 1982 and Whitman, 1990].
2. Placement of seismic resultant force at 0.6H is based on wall behavior and model test results [Whitman, 1990].

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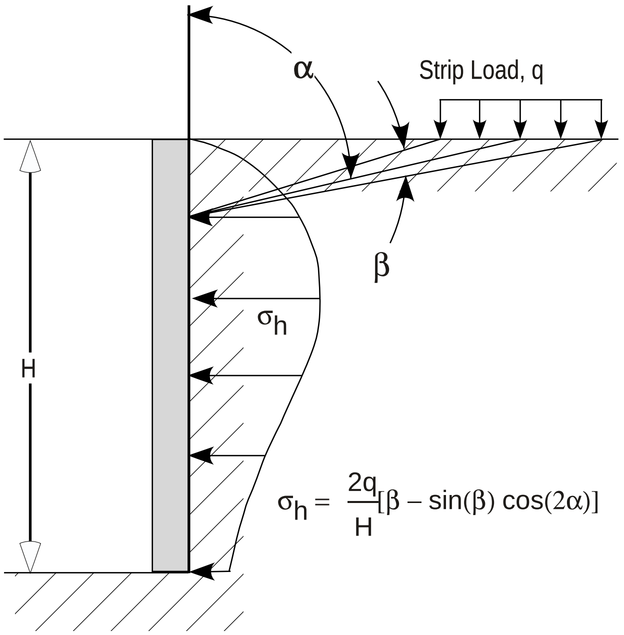
FIGURE 5

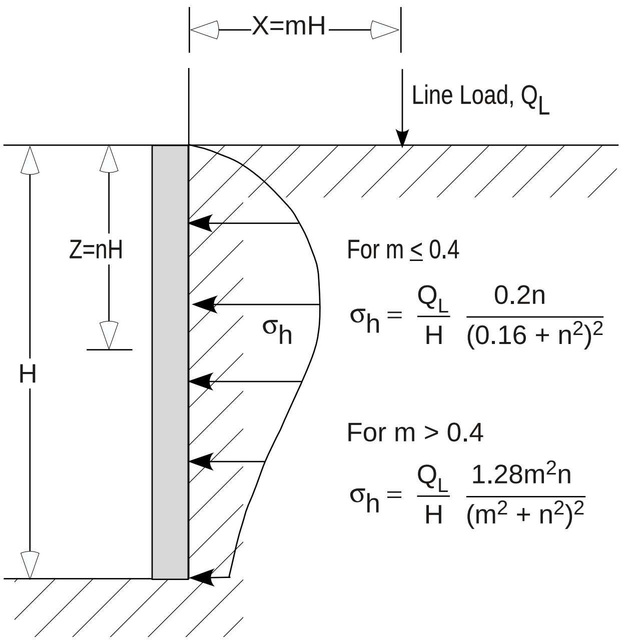
Retaining Wall Surcharge

STRIP LOAD PARALLEL TO WALL[1]



$$\sigma_h = \frac{2q}{H}[\beta - \sin(\beta)\cos(2\alpha)]$$

LINE LOAD PARALLEL TO WALL[2]



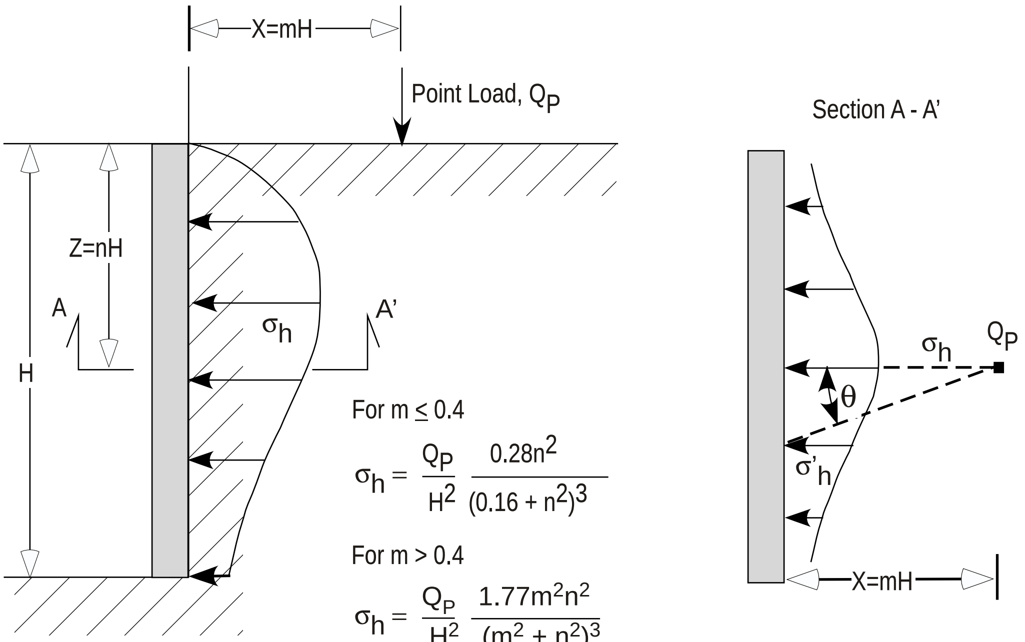
For m ≤ 0.4

$$\sigma_h = \frac{Q_L}{H}\,\frac{0.2n}{(0.16 + n^2)^2}$$

For m > 0.4

$$\sigma_h = \frac{Q_L}{H}\,\frac{1.28m^2n}{(m^2 + n^2)^2}$$

VERTICAL POINT LOAD[2]



For m ≤ 0.4

$$\sigma_h = \frac{Q_P}{H^2}\,\frac{0.28n^2}{(0.16 + n^2)^3}$$

For m > 0.4

$$\sigma_h = \frac{Q_P}{H^2}\,\frac{1.77m^2n^2}{(m^2 + n^2)^3}$$

$$\sigma'_h = \sigma_h \cos^2(1.1\,\theta)$$

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Notes: 1. Das, Principles of Geotechnical Engineering, 1990 Edition.
2. NAVFAC Design Manual 7.06.

Refer to the referenced design manuals for additional guidance. Contact CGT if there are any questions with modeling surcharge loads.

Carlson Geotechnical
A Division of Carlson Testing, Inc.
Phone: (503) 601-8250
www.carlsontesting.com

Bend Office **(541) 330-9155**
Eugene Office **(541) 345-0289**
Salem Office **(503) 589-1252**
Tigard Office **(503) 684-3460**

Appendix A: Subsurface Investigation and Laboratory Testing

Baxter Road Apartments
1709 Baxter Road SE
Salem, Oregon

CGT Project Number G2406149

July 12, 2024

Prepared For:

Laura Robinson
Neighborly Development
2925 River Road South, Suite 100
Salem, Oregon 97302

Prepared by
Carlson Geotechnical

A.1.0 SUBSURFACE INVESTIGATION

Our field investigation consisted of thirteen test pits completed on June 20, 2024. The exploration locations are shown on the Site Plan, attached to the geotechnical report as Figure 2. The exploration locations shown therein were determined based on measurements from existing site features (buildings, trees, etc.) and are approximate. Surface elevations indicated on the logs were estimated based on the topographic contours shown on the Existing Conditions ALTA plan, produced by Multi/Tech Engineering Services, Inc, provided by the client. The attached figures detail the exploration methods (Figure A1), soil classification criteria (Figure A2), and present detailed logs of the explorations (Figures A3 through A15), as discussed below.

A.1.1 Test Pits

CGT observed the excavation of thirteen test pits (TP-1 through TP-13) at the site to depths of about 8 to 10 feet bgs. The test pits were excavated using a CAT 350E mini-excavator provided and operated by our excavation subcontractor, Doug Shepherd's Dirtworks of Keizer, Oregon. The test pits were loosely backfilled with the excavated materials upon completion.

A.1.2 In-Situ Testing

A.1.2.1 Pocket Penetrometer Tests

Pocket penetrometer readings were generally taken at approximate ½-foot intervals in the upper four feet of each test pit. The pocket penetrometer is a hand-held instrument that provides an approximation of the unconfined compressive strength of cohesive, fine-grained soils. The correlation between pocket penetrometer readings and the consistency of cohesive, fine-grained soils is provided on the attached Figure A2.

A.1.3 Material Classification & Sampling

Representative disturbed (grab) samples of the soils encountered were obtained at select intervals within the test pits. A qualified member of CGT's geological staff collected the samples and logged the soils in general accordance with the Visual-Manual Procedure (ASTM D2488). An explanation of this classification system is attached as Figure A2. The grab samples were stored in sealable plastic bags and transported to our soils laboratory for further examination and testing. Our geotechnical staff visually examined all samples in order to refine the initial field classifications.

A.1.4 Subsurface Conditions

Subsurface conditions are summarized in Section 2.3 of the geotechnical report. Detailed logs of the explorations are presented on the attached exploration logs, Figures A3 through A15.

A.2.0 LABORATORY TESTING

Laboratory testing was performed on samples collected in the field to refine our initial field classifications and determine in-situ parameters. Laboratory testing included the following:

- Twelve moisture content determinations (ASTM D2216).
- Two Atterberg limits (plasticity) tests (ASTM D4318).

Results of the laboratory tests are shown on the exploration logs.

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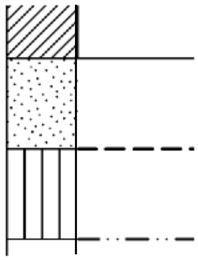
FIGURE A1
Exploration Key

PL —●— LL MC	Atterberg limits (plasticity) test results (ASTM D4318): PL = Plastic Limit, LL = Liquid Limit, and MC= Moisture Content (ASTM D2216)
☐ FINES CONTENT (%)	Percentage passing the U.S. Standard No. 200 Sieve (ASTM D1140)

SAMPLING

GRAB	Grab sample
BULK	Bulk sample
☒ SPT	**Standard Penetration Test** (SPT) consists of driving a 2-inch, outside-diameter, split-spoon sampler into the undisturbed formation with repeated blows of a 140-pound, hammer falling a vertical distance of 30 inches (ASTM D1586). The number of blows (N-value) required to drive the sampler the last 12 inches of an 18-inch sample interval is used to characterize the soil consistency or relative density. The drill rig was equipped with an cat-head or automatic hammer to conduct the SPTs. The observed N-values, hammer efficiency, and N_{60} are noted on the boring logs.
MC	**Modified California** sampling consists of 3-inch, outside-diameter, split-spoon sampler (ASTM D3550) driven similarly to the SPT sampling method described above. A sampler diameter correction factor of 0.44 is applied to calculate the equivalent SPT N_{60} value per Lacroix and Horn, 1973.
CORE	**Rock Coring** interval
■ SH	**Shelby Tube** is a 3-inch, inner-diameter, thin-walled, steel tube push sampler (ASTM D1587) used to collect relatively undisturbed samples of fine-grained soils.
WDCP	**Wildcat Dynamic Cone Penetrometer** (WDCP) test consists of driving 1.1-inch diameter, steel rods with a 1.4-inch diameter, cone tip into the ground using a 35-pound drop hammer with a 15-inch free-fall height. The number of blows required to drive the steel rods is recorded for each 10 centimeters (3.94 inches) of penetration. The blow count for each interval is then converted to the corresponding SPT N_{60} values.
DCP	**Dynamic Cone Penetrometer** (DCP) test consists of driving a 20-millimeter diameter, hardened steel cone on 16-millimeter diameter steel rods into the ground using a 10-kilogram drop hammer with a 460-millimeter free-fall height. The depth of penetration in millimeters is recorded for each drop of the hammer.
POCKET PEN. (tsf)	**Pocket Penetrometer** test is a hand-held instrument that provides an approximation of the unconfined compressive strength in tons per square foot (tsf) of cohesive, fine-grained soils.

CONTACTS



Observed (measured) contact between soil or rock units.

Inferred (approximate) contact between soil or rock units.

Transitional (gradational) contact between soil or rock units.

ADDITIONAL NOTATIONS

Italics	Notes drilling action or digging effort
{ Braces }	Interpretation of material origin/geologic formation (e.g. { Base Rock } or { Columbia River Basalt })

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All measurements are approximate.

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Project Number G2406149

FIGURE A2

Soil Classification

Classification of Terms and Content

NAME: Group Name and Symbol
Relative Density or Consistency
Color
Moisture Content
Plasticity
Other Constituents
Other: Grain Shape, Approximate Gradation
Organics, Cement, Structure, Odor, etc.
Geologic Name or Formation

Grain Size

		U.S. Standard Sieve
Fines		<#200 (0.075 mm)
Sand	Fine	#200 - #40 (0.425 mm)
	Medium	#40 - #10 (2 mm)
	Coarse	#10 - #4 (4.75 mm)
Gravel	Fine	#4 - 0.75 inch
	Coarse	0.75 inch - 3 inches
Cobbles		3 to 12 inches
Boulders		> 12 inches

Coarse-Grained (Granular) Soils

Relative Density

SPT N_{60}-Value	Density
0 - 4	Very Loose
4 - 10	Loose
10 - 30	Medium Dense
30 - 50	Dense
>50	Very Dense

Minor Constituents

Percent by Volume	Descriptor	Example
0 - 5%	"Trace" as part of soil description	"trace silt"
5 - 15%	"With" as part of group name	"**POORLY GRADED SAND WITH SILT**"
15 - 49%	Modifier to group name	"**SILTY SAND**"

Fine-Grained (Cohesive) Soils

SPT N_{60}-Value	Torvane tsf Shear Strength	Pocket Pen tsf Unconfined	Consistency	Manual Penetration Test
<2	<0.13	<0.25	Very Soft	Thumb penetrates more than 1 inch
2 - 4	0.13 - 0.25	0.25 - 0.50	Soft	Thumb penetrates about 1 inch
4 - 8	0.25 - 0.50	0.50 - 1.00	Medium Stiff	Thumb penetrates about ¼ inch
8 - 15	0.50 - 1.00	1.00 - 2.00	Stiff	Thumb penetrates less than ¼ inch
15 - 30	1.00 - 2.00	2.00 - 4.00	Very Stiff	Readily indented by thumbnail
>30	>2.00	>4.00	Hard	Difficult to indent by thumbnail

Minor Constituents

Percent by Volume	Descriptor	Example
0 - 5%	"Trace" as part of soil description	"trace fine-grained sand"
5 - 15%	"Some" as part of soil description	"some fine-grained sand"
15 - 30%	"With" as part of group name	"**SILT WITH SAND**"
30 - 49%	Modifier to group name	"**SANDY SILT**"

Moisture Content

Dry: Absence of moisture, dusty, dry to the touch

Moist: Leaves moisture on hand

Wet: Visible free water, likely from below water table

	Plasticity	Dry Strength	Dilatancy	Toughness
ML	Non to Low	Non to Low	Slow to Rapid	Low, can't roll
CL	Low to Medium	Medium to High	None to Slow	Medium
MH	Medium to High	Low to Medium	None to Slow	Low to Medium
CH	Medium to High	High to Very High	None	High

Structure

Stratified: Alternating layers of material or color >6 mm thick

Laminated: Alternating layers < 6 mm thick

Fissured: Breaks along definite fracture planes

Slickensided: Striated, polished, or glossy fracture planes

Blocky: Cohesive soil that can be broken down into small angular lumps which resist further breakdown

Lenses: Has small pockets of different soils, note thickness

Homogeneous: Same color and appearance throughout

Visual-Manual Classification

Major Divisions			Group Symbols	Typical Names
Coarse Grained Soils: More than 50% retained on No. 200 sieve	Gravels: 50% or more *retained* on the No. 4 sieve	Clean Gravels	GW	Well-graded gravels and gravel/sand mixtures, little or no fines
			GP	Poorly-graded gravels and gravel/sand mixtures, little or no fines
		Gravels with Fines	GM	Silty gravels, gravel/sand/silt mixtures
			GC	Clayey gravels, gravel/sand/clay mixtures
	Sands: More than 50% *passing* the No. 4 sieve	Clean Sands	SW	Well-graded sands and gravelly sands, little or no fines
			SP	Poorly-graded sands and gravelly sands, little or no fines
		Sands with Fines	SM	Silty sands, sand/silt mixtures
			SC	Clayey sands, sand/clay mixtures
Fine-Grained Soils: 50% or more Passes No. 200 Sieve	Silt and Clays Low Plasticity Fines		ML	Inorganic silts, rock flour, clayey silts
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, lean clays
			OL	Organic soil of low plasticity
	Silt and Clays High Plasticity Fines		MH	Inorganic silts, clayey silts
			CH	Inorganic clays of high plasticity, fat clays
			OH	Organic soil of medium to high plasticity
Highly Organic Soils			PT	Peat, muck, and other highly organic soils

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References:

ASTM D2487 Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System)
ASTM D2488 Standard Practice for Description and Identification of Soils (Visual-Manual Procedure)
Terzaghi, K., and Peck, R.B., 1948, Soil Mechanics in Engineering Practice, John Wiley & Sons.

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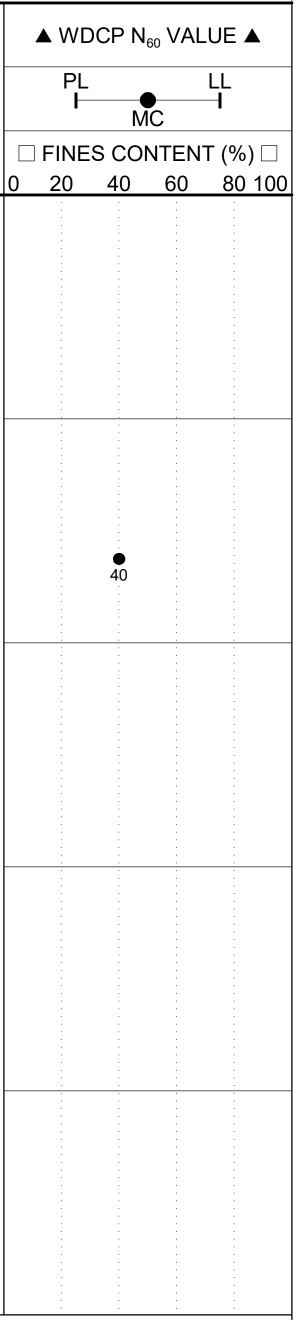
FIGURE A3

Test Pit TP-1

PAGE 1 OF 1

CLIENT Neighborly Development
PROJECT NAME Baxter Road Apartments
PROJECT NUMBER G2406149
PROJECT LOCATION 1709 Baxter Road Southeast - Salem, Oregon
DATE STARTED 6/20/24
GROUND ELEVATION 502 ft
ELEVATION DATUM Provided Topographic Survey
WEATHER Sunny, 80F
SURFACE Grass
LOGGED BY BJG
REVIEWED BY BMW
EXCAVATION CONTRACTOR Doug Shepherd Dirtworks
SEEPAGE ---
EQUIPMENT 350E CAT Mini Excavator
GROUNDWATER DURING DRILLING ---
EXCAVATION METHOD 2-foot toothed bucket
GROUNDWATER AFTER EXCAVATION ---

ELEVATION (ft)	GRAPHIC LOG	GROUP SYMBOL	MATERIAL DESCRIPTION	GROUNDWATER	DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	WDCP N_{60} VALUE	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)
		OL	**ORGANIC SOIL:** Brown, dry, low plasticity, with abundant rootlets.		0					
500 498 496 494 492		MH	**ELASTIC SILT:** Medium stiff, red, moist, medium to high plasticity, trace weathered basalt gravel up to 1-inch in diameter (5% by volume). {Residual soil} Very stiff below 1 foot bgs. With severely weathered basalt gravel and cobbles up to 5-inches in diameter below 6 feet bgs (15% by volume).		2 4 6 8 10	GRAB 1 GRAB 2	100 100		0.5 3.5 4.0 4.0 4.0 4.0 4.0 4.0	



490

- Test pit terminated at 10 feet bgs.
- No caving or groundwater encountered.
- Test pit loosely backfilled with spoils upon completion.

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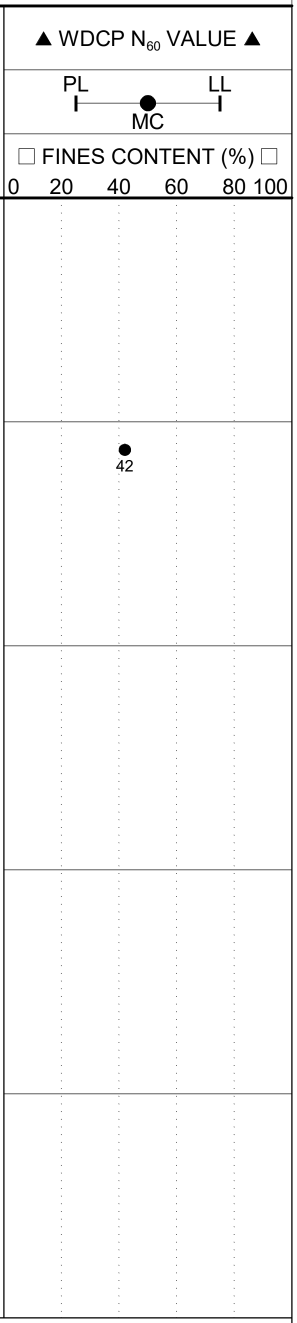
FIGURE A4

Test Pit TP-2

PAGE 1 OF 1

CLIENT Neighborly Development
PROJECT NAME Baxter Road Apartments
PROJECT NUMBER G2406149
PROJECT LOCATION 1709 Baxter Road Southeast - Salem, Oregon
DATE STARTED 6/20/24
GROUND ELEVATION 474 ft
ELEVATION DATUM Provided Topographic Survey
WEATHER Sunny, 80F
SURFACE Grass
LOGGED BY BJG
REVIEWED BY BMW
EXCAVATION CONTRACTOR Doug Shepherd Dirtworks
SEEPAGE ---
EQUIPMENT 350E CAT Mini Excavator
GROUNDWATER DURING DRILLING ---
EXCAVATION METHOD 2-foot toothed bucket
GROUNDWATER AFTER EXCAVATION ---

ELEVATION (ft)	GRAPHIC LOG	GROUP SYMBOL	MATERIAL DESCRIPTION	GROUNDWATER	DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	WDCP N_{60} VALUE	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)
		OL	**ORGANIC SOIL:** Brown, dry, low plasticity, with abundant rootlets.		0					
		MH	**ELASTIC SILT:** Medium stiff, red with yellow/gray mottling, moist, medium to high plasticity, trace weathered basalt gravel up to 3-inches in diameter (5% by volume). {Residual soil} Very stiff below 1 foot bgs.						0.5 3.5 4.0	
472					2	GRAB 1	100		4.0	
			No roots below 2 feet bgs.						4.0 4.0 4.0	
470					4				4.0	
468			With severely weathered basalt gravel and cobbles up to 8-inches in diameter below 5½ feet bgs (15% by volume).		6					
466					8	GRAB 2	100			
464					10					
462										



- Test pit terminated at 10 feet bgs.
- No caving or groundwater encountered.
- Test pit loosely backfilled with spoils upon completion.

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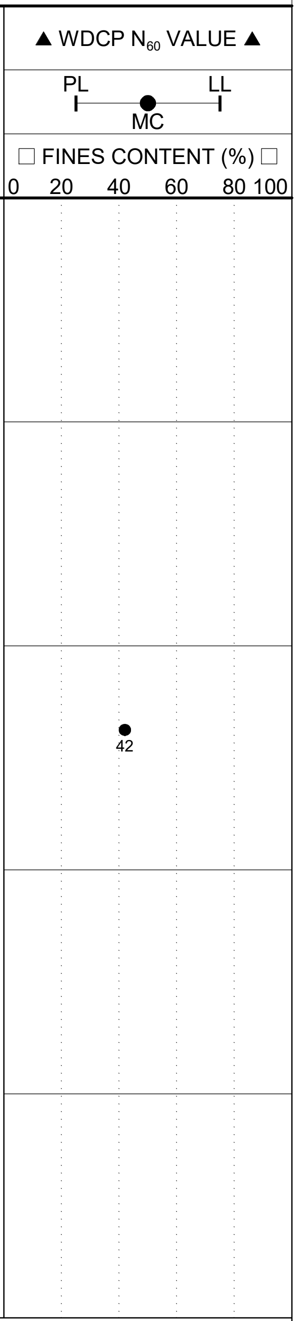
FIGURE A5

Test Pit TP-3

CLIENT Neighborly Development
PROJECT NAME Baxter Road Apartments
PROJECT NUMBER G2406149
PROJECT LOCATION 1709 Baxter Road Southeast - Salem, Oregon
DATE STARTED 6/20/24
GROUND ELEVATION 498 ft
ELEVATION DATUM Provided Topographic Survey
WEATHER Sunny, 80F
SURFACE Grass
LOGGED BY BJG
REVIEWED BY BMW
EXCAVATION CONTRACTOR Doug Shepherd Dirtworks
SEEPAGE ---
EQUIPMENT 350E CAT Mini Excavator
GROUNDWATER DURING DRILLING ---
EXCAVATION METHOD 2-foot toothed bucket
GROUNDWATER AFTER EXCAVATION ---

ELEVATION (ft)	GROUP SYMBOL	MATERIAL DESCRIPTION	GROUNDWATER	DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	WDCP N_{60} VALUE	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MC
498	OL	**ORGANIC SOIL:** Brown, dry, low plasticity, with abundant rootlets.		0						
	MH	**ELASTIC SILT:** Medium stiff, red with yellow mottling, moist, medium to high plasticity, trace roots up to 1-inch in diameter. {Residual soil} Very stiff below 1 foot bgs.						0.5 2.5 3.5		
496				2				4.0		
		No roots below 2½ feet bgs.						4.0 4.0 4.0		
494				4				4.0		
					GRAB 1	100				42
		Some severely weathered basalt gravel and cobbles up to 6-inches in diameter below 5 feet bgs (10% by volume).								
492				6						
490				8						
488				10	GRAB 2	100				
486										



- Test pit terminated at 10 feet bgs.
- No caving or groundwater encountered.
- Test pit loosely backfilled with spoils upon completion.

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FIGURE A6

Test Pit TP-4

PAGE 1 OF 1

CLIENT Neighborly Development

PROJECT NAME Baxter Road Apartments

PROJECT NUMBER G2406149

PROJECT LOCATION 1709 Baxter Road Southeast - Salem, Oregon

DATE STARTED 6/20/24

GROUND ELEVATION 498 ft

ELEVATION DATUM Provided Topographic Survey

WEATHER Sunny, 80F

SURFACE Grass

LOGGED BY BJG

REVIEWED BY BMW

EXCAVATION CONTRACTOR Doug Shepherd Dirtworks

SEEPAGE ---

EQUIPMENT 350E CAT Mini Excavator

GROUNDWATER DURING DRILLING ---

EXCAVATION METHOD 2-foot toothed bucket

GROUNDWATER AFTER EXCAVATION ---

ELEVATION (ft)	GRAPHIC LOG	GROUP SYMBOL	MATERIAL DESCRIPTION	GROUNDWATER	DEPTH (ft) 0	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	WDCP N_{60} VALUE	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	▲ WDCP N_{60} VALUE ▲ PL MC LL ☐ FINES CONTENT (%) ☐ 0 20 40 60 80 100
		OL	**ORGANIC SOIL:** Brown, dry, low plasticity, with abundant rootlets.								
496		MH	**ELASTIC SILT:** Very stiff, red with yellow/gray mottling, moist, medium to high plasticity, trace roots up to 1-inch in diameter and weathered basalt gravel up to ½-inch in diameter (5% by volume). {Residual soil}		2				2.0 2.0 3.5 4.0		
494			No roots below 2½ feet bgs.		4	GRAB 1	100		4.0 4.0 4.0 4.0		
492					6						
490			Some severely weathered basalt gravel and cobbles up to 8-inches in diameter below 6½ feet bgs (10% by volume).		8						
488					10	GRAB 2	100				● 45
486											

- Test pit terminated at 10 feet bgs.
- No caving or groundwater encountered.
- Test pit loosely backfilled with spoils upon completion.

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FIGURE A7

Test Pit TP-5

PAGE 1 OF 1

CLIENT Neighborly Development
PROJECT NAME Baxter Road Apartments
PROJECT NUMBER G2406149
PROJECT LOCATION 1709 Baxter Road Southeast - Salem, Oregon
DATE STARTED 6/20/24
GROUND ELEVATION 506 ft
ELEVATION DATUM Provided Topographic Survey
WEATHER Sunny, 80F
SURFACE Grass
LOGGED BY BJG
REVIEWED BY BMW
EXCAVATION CONTRACTOR Doug Shepherd Dirtworks
SEEPAGE ---
EQUIPMENT 350E CAT Mini Excavator
GROUNDWATER DURING DRILLING ---
EXCAVATION METHOD 2-foot toothed bucket
GROUNDWATER AFTER EXCAVATION ---

ELEVATION (ft)	GRAPHIC LOG	GROUP SYMBOL	MATERIAL DESCRIPTION	GROUNDWATER	DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	WDCP N_{60} VALUE	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	▲ WDCP N_{60} VALUE ▲ PL ● MC LL ☐ FINES CONTENT (%) ☐ 0 20 40 60 80 100
		OL	**ORGANIC SOIL:** Brown, dry, low plasticity, with abundant rootlets.		0						
		MH	**ELASTIC SILT:** Medium stiff, red with yellow mottling, moist, medium to high plasticity. {Residual soil} Very stiff below 1 foot bgs.						0.5 3.0 3.5		
504					2	GRAB 1	100		3.5		38
			Some severely weathered basalt gravels up to 2-inches in diameter below 3 feet bgs (10% by volume).						4.0 4.0 4.0		
502					4				4.0		
500					6						
			With severely weathered basalt gravels and cobbles up to 10-inches in diameter below 6½ feet bgs (15% in volume).			GRAB 2	100				
498					8						
496					10						
494											

- Test pit terminated at 10 feet bgs.
- No caving or groundwater encountered.
- Test pit loosely backfilled with spoils upon completion.

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FIGURE A8

Test Pit TP-6

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CLIENT Neighborly Development
PROJECT NAME Baxter Road Apartments
PROJECT NUMBER G2406149
PROJECT LOCATION 1709 Baxter Road Southeast - Salem, Oregon
DATE STARTED 6/20/24
GROUND ELEVATION 499 ft
ELEVATION DATUM Provided Topographic Survey
WEATHER Sunny, 80F
SURFACE Grass
LOGGED BY BJG
REVIEWED BY BMW
EXCAVATION CONTRACTOR Doug Shepherd Dirtworks
SEEPAGE ---
EQUIPMENT 350E CAT Mini Excavator
GROUNDWATER DURING DRILLING ---
EXCAVATION METHOD 2-foot toothed bucket
GROUNDWATER AFTER EXCAVATION ---

ELEVATION (ft)	GRAPHIC LOG	GROUP SYMBOL	MATERIAL DESCRIPTION	GROUNDWATER	DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	WDCP N_{60} VALUE	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	▲ WDCP N_{60} VALUE ▲ PL ●MC LL ☐ FINES CONTENT (%) ☐ 0 20 40 60 80 100
		OL	**ORGANIC SOIL:** Brown, dry, low plasticity, with abundant rootlets.		0						
498		MH	**ELASTIC SILT:** Soft, red, moist, medium to high plasticity, trace yellow mottling and roots up to 1-inch in diameter. {Residual soil} Very stiff below 1 foot bgs.		2				0.25 2.5 3.0 3.0		
496			No roots below 3 feet bgs.		4				3.0 3.0 3.0 3.0		
						GRAB 1	100				PL 33, MC 28, LL 53
494			Trace severely weathered basalt gravels up to 2-inches in diameter below 4½ feet bgs (5% by volume).		6						
492			With severely weathered basalt gravels and cobbles up to 5-inches in diameter below 7 feet bgs (15% by volume).		8						
490					10	GRAB 2	100				
488											

- Test pit terminated at 10 feet bgs.
- No caving or groundwater encountered.
- Test pit loosely backfilled with spoils upon completion.

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FIGURE A9

Test Pit TP-7

PAGE 1 OF 1

CLIENT Neighborly Development
PROJECT NAME Baxter Road Apartments
PROJECT NUMBER G2406149
PROJECT LOCATION 1709 Baxter Road Southeast - Salem, Oregon
DATE STARTED 6/20/24
GROUND ELEVATION 507 ft
ELEVATION DATUM Provided Topographic Survey
WEATHER Sunny, 80F
SURFACE Grass
LOGGED BY BJG
REVIEWED BY BMW
EXCAVATION CONTRACTOR Doug Shepherd Dirtworks
SEEPAGE ---
EQUIPMENT 350E CAT Mini Excavator
GROUNDWATER DURING DRILLING ---
EXCAVATION METHOD 2-foot toothed bucket
GROUNDWATER AFTER EXCAVATION ---

ELEVATION (ft)	GRAPHIC LOG	GROUP SYMBOL	MATERIAL DESCRIPTION	GROUNDWATER	DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	WDCP N_{60} VALUE	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	▲ WDCP N_{60} VALUE ▲ PL MC LL ☐ FINES CONTENT (%) ☐ 0 20 40 60 80 100
		OL	**ORGANIC SOIL:** Brown, dry, low plasticity, with abundant rootlets.		0						
		MH	**ELASTIC SILT:** Medium stiff, red with yellow mottling, moist, medium to high plasticity, trace weathered basalt gravel up to 1-inch in diameter (5% by volume). {Residual soil} Very stiff below 1 foot bgs.						0.5		
506									2.5		
									4.0		
					2				4.0		
									4.0		
504									4.0		
						GRAB 1	100		4.0		
					4				4.0		
502											
					6						
500											
			With severely weathered basalt gravel and cobbles up to 6-inches in diameter below 7½ feet bgs (15% by volume).		8						
						GRAB 2	100				
498											
					10						
496											

- Test pit terminated at 10 feet bgs.
- No caving or groundwater encountered.
- Test pit loosely backfilled with spoils upon completion.

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FIGURE A10

Test Pit TP-8

PAGE 1 OF 1

CLIENT Neighborly Development

PROJECT NAME Baxter Road Apartments

PROJECT NUMBER G2406149

PROJECT LOCATION 1709 Baxter Road Southeast - Salem, Oregon

DATE STARTED 6/20/24

GROUND ELEVATION 506 ft

ELEVATION DATUM Provided Topographic Survey

WEATHER Sunny, 80F

SURFACE Grass

LOGGED BY BJG

REVIEWED BY BMW

EXCAVATION CONTRACTOR Doug Shepherd Dirtworks

SEEPAGE ---

EQUIPMENT 350E CAT Mini Excavator

GROUNDWATER DURING DRILLING ---

EXCAVATION METHOD 2-foot toothed bucket

GROUNDWATER AFTER EXCAVATION ---

ELEVATION (ft)	GRAPHIC LOG	GROUP SYMBOL	MATERIAL DESCRIPTION	GROUNDWATER	DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	WDCP N_{60} VALUE	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	▲ WDCP N_{60} VALUE ▲ PL MC LL ☐ FINES CONTENT (%) ☐ 0 20 40 60 80 100
506		OL	**ORGANIC SOIL:** Brown, dry, low plasticity, with abundant rootlets.		0						
		MH	**ELASTIC SILT:** Medium stiff, red with yellow mottling, moist, medium to high plasticity, trace weathered basalt gravel up to ½-inch in diameter (5% by volume). {Residual soil} Stiff to very stiff below 1 foot bgs.						0.5 1.5 3.5		
504			Very stiff below 2 feet bgs.		2	GRAB 1	100		4.0 4.0 4.0 4.0		
502					4				4.0		
500			With severely weathered basalt gravel and cobbles up to 5-inches in diameter below 6 feet bgs (15% by volume).		6	GRAB 2	100				● 30
498					8						
496					10						
494											

- Test pit terminated at 10 feet bgs.
- No caving or groundwater encountered.
- Test pit loosely backfilled with spoils upon completion.

CGT EXPLORATION WITH WDCP DRAFT LOGS.GPJ 7/11/24 DRAFTED BY: AET

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FIGURE A11

Test Pit TP-9

PAGE 1 OF 1

CLIENT Neighborly Development
PROJECT NAME Baxter Road Apartments
PROJECT NUMBER G2406149
PROJECT LOCATION 1709 Baxter Road Southeast - Salem, Oregon
DATE STARTED 6/20/24
GROUND ELEVATION 510 ft
ELEVATION DATUM Provided Topographic Survey
WEATHER Sunny, 80F
SURFACE Grass
LOGGED BY BJG
REVIEWED BY BMW
EXCAVATION CONTRACTOR Doug Shepherd Dirtworks
SEEPAGE ---
EQUIPMENT 350E CAT Mini Excavator
GROUNDWATER DURING DRILLING ---
EXCAVATION METHOD 2-foot toothed bucket
GROUNDWATER AFTER EXCAVATION ---

ELEVATION (ft)	GRAPHIC LOG	GROUP SYMBOL	MATERIAL DESCRIPTION	GROUNDWATER	DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	WDCP N_{60} VALUE	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	▲ WDCP N_{60} VALUE ▲ PL ● MC LL ☐ FINES CONTENT (%) ☐ 0 20 40 60 80 100
		OL	**ORGANIC SOIL:** Brown, dry, low plasticity, with abundant rootlets.		0						
		MH	**ELASTIC SILT:** Medium stiff, red, moist, medium to high plasticity, trace yellow mottling and weathered basalt gravel up to ¼-inch in diameter (5% by volume). {Residual soil} Very stiff below 1 foot bgs.						0.5		
									2.5		
									4.0		
508					2				4.0		
									4.0		
						GRAB 1	100		4.0		● 25
									4.0		
506					4				4.0		
504					6						
						GRAB 2	100				
502			Some severely weathered basalt gravel up to 2-inches in diameter below 8 feet bgs (10% by volume).		8						
500					10						
498											

- Test pit terminated at 10 feet bgs.
- No caving or groundwater encountered.
- Test pit loosely backfilled with spoils upon completion.

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FIGURE A12

Test Pit TP-10

CLIENT Neighborly Development
PROJECT NAME Baxter Road Apartments
PROJECT NUMBER G2406149
PROJECT LOCATION 1709 Baxter Road Southeast - Salem, Oregon
DATE STARTED 6/20/24
GROUND ELEVATION 512 ft
ELEVATION DATUM Provided Topographic Survey
WEATHER Sunny, 80F
SURFACE Grass
LOGGED BY BJG
REVIEWED BY BMW
EXCAVATION CONTRACTOR Doug Shepherd Dirtworks
SEEPAGE ---
EQUIPMENT 350E CAT Mini Excavator
GROUNDWATER DURING DRILLING ---
EXCAVATION METHOD 2-foot toothed bucket
GROUNDWATER AFTER EXCAVATION ---

ELEVATION (ft)	GRAPHIC LOG	GROUP SYMBOL	MATERIAL DESCRIPTION	GROUNDWATER	DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	WDCP N_{60} VALUE	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	▲ WDCP N_{60} VALUE ▲ PL MC LL ☐ FINES CONTENT (%) ☐ 0 20 40 60 80 100
		OL	**ORGANIC SOIL:** Brown, dry, low plasticity, with abundant rootlets.		0						
			ELASTIC SILT: Medium stiff, red, moist, medium to high plasticity, some weathered basalt gravel up to 1-inch in diameter (10% by volume), trace yellow mottling. {Residual soil} Very stiff below 1 foot bgs.						0.5 3.5 4.0		
510					2				4.0 4.0 4.0 4.0		
508					4				4.0		
		MH				GRAB 1	100				● 29
506					6						
			With severely weathered basalt gravel up to 3-inches in diameter below 6½ feet bgs (15% by volume).								
504					8						
502					10	GRAB 2	100				
500											

- Test pit terminated at 10 feet bgs.
- No caving or groundwater encountered.
- Test pit loosely backfilled with spoils upon completion.

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FIGURE A13

Test Pit TP-11

PAGE 1 OF 1

CLIENT Neighborly Development
PROJECT NAME Baxter Road Apartments
PROJECT NUMBER G2406149
PROJECT LOCATION 1709 Baxter Road Southeast - Salem, Oregon
DATE STARTED 6/20/24
GROUND ELEVATION 513 ft
ELEVATION DATUM Provided Topographic Survey
WEATHER Sunny, 80F
SURFACE Grass
LOGGED BY BJG
REVIEWED BY BMW
EXCAVATION CONTRACTOR Doug Shepherd Dirtworks
SEEPAGE ---
EQUIPMENT 350E CAT Mini Excavator
GROUNDWATER DURING DRILLING ---
EXCAVATION METHOD 2-foot toothed bucket
GROUNDWATER AFTER EXCAVATION ---

ELEVATION (ft)	GRAPHIC LOG	GROUP SYMBOL	MATERIAL DESCRIPTION	GROUNDWATER	DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	WDCP N_{60} VALUE	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	▲ WDCP N_{60} VALUE ▲ PL MC LL ☐ FINES CONTENT (%) ☐ 0 20 40 60 80 100
		OL	**ORGANIC SOIL:** Brown, dry, low plasticity, with abundant rootlets.		0						
512		MH	**ELASTIC SILT:** Medium stiff, red, moist, medium to high plasticity, trace weathered basalt gravel up to ½-inch in diameter (5% by volume). {Residual soil} Very stiff below 1 foot bgs.						0.5 2.5 3.0		
					2				3.5		
510									4.0 4.0		
					4	GRAB 1	100		4.0 4.0		49 59 46
508											
					6						
506						GRAB 2	100				
					8						
			With severely weathered basalt gravel up to 3-inches in diameter below 8 feet bgs (15% by volume).								
504											
					10						

502

- Test pit terminated at 10 feet bgs.
- No caving or groundwater encountered.
- Test pit loosely backfilled with spoils upon completion.

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FIGURE A14

Test Pit TP-12

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CLIENT Neighborly Development
PROJECT NAME Baxter Road Apartments
PROJECT NUMBER G2406149
PROJECT LOCATION 1709 Baxter Road Southeast - Salem, Oregon
DATE STARTED 6/20/24
GROUND ELEVATION 514 ft
ELEVATION DATUM Provided Topographic Survey
WEATHER Sunny, 80F
SURFACE Grass
LOGGED BY BJG
REVIEWED BY BMW
EXCAVATION CONTRACTOR Doug Shepherd Dirtworks
SEEPAGE ---
EQUIPMENT 350E CAT Mini Excavator
GROUNDWATER DURING DRILLING ---
EXCAVATION METHOD 2-foot toothed bucket
GROUNDWATER AFTER EXCAVATION ---

ELEVATION (ft)	GRAPHIC LOG	GROUP SYMBOL	MATERIAL DESCRIPTION	GROUNDWATER	DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	WDCP N_{60} VALUE	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	▲ WDCP N_{60} VALUE ▲ PL — MC — LL ☐ FINES CONTENT (%) ☐ 0 20 40 60 80 100
		OL	**ORGANIC SOIL:** Brown, dry, low plasticity, with abundant rootlets.		0						
512			**ELASTIC SILT:** Medium stiff, red, moist, medium to high plasticity, some weathered basalt gravel up to 1-inch in diameter (10% by volume). {Residual soil} Very stiff below 1 foot bgs.		2				1.0 4.0 4.0 4.0		
510					4				4.0 4.0 4.0 4.0		
508		MH	With severely weathered basalt gravel and cobbles up to 6-inches in diameter below 5 feet bgs (15% by volume).		6	GRAB 1	100				27
506					8						
504					10	GRAB 2	100				

- Test pit terminated at 10 feet bgs.
- No caving or groundwater encountered.
- Test pit loosely backfilled with spoils upon completion.

502

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FIGURE A15

Test Pit TP-13

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CLIENT Neighborly Development
PROJECT NAME Baxter Road Apartments
PROJECT NUMBER G2406149
PROJECT LOCATION 1709 Baxter Road Southeast - Salem, Oregon
DATE STARTED 6/20/24
GROUND ELEVATION 512 ft
ELEVATION DATUM Provided Topographic Survey
WEATHER Sunny, 80F
SURFACE Grass
LOGGED BY BJG
REVIEWED BY BMW
EXCAVATION CONTRACTOR Doug Shepherd Dirtworks
SEEPAGE ---
EQUIPMENT 350E CAT Mini Excavator
GROUNDWATER DURING DRILLING ---
EXCAVATION METHOD 2-foot toothed bucket
GROUNDWATER AFTER EXCAVATION ---

ELEVATION (ft)	GRAPHIC LOG	GROUP SYMBOL	MATERIAL DESCRIPTION	GROUNDWATER	DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	WDCP N_{60} VALUE	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	▲ WDCP N_{60} VALUE ▲ PL MC LL ☐ FINES CONTENT (%) ☐ 0 20 40 60 80 100
		OL	**ORGANIC SOIL:** Brown, dry, low plasticity, with abundant rootlets.		0						
		MH	**ELASTIC SILT:** Stiff, red with yellow mottling, moist, medium to high plasticity. {Residual soil} Very stiff below 1 foot bgs.						1.0 2.5 3.5		
510					2				3.5 4.0 4.0 4.0		
508					4	GRAB 1	100		4.0		27
			Some severely weathered basalt gravel up to 3-inches in diameter below 4½ feet bgs (10% by volume).								
506					6						
504					8	GRAB 2	100				
502											
500											

- Test pit terminated at 8 feet bgs due to practical refusal on very hard soil.
- No caving or groundwater encountered.
- Test pit loosely backfilled with spoils upon completion.