APPENDIX F: GEOHAZARD/GEOTECHNICAL REPORT

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Report of
Preliminary Geotechnical Investigation, Infiltration Testing & Geologic Assessment
Fairview Subdivision
Pringle Creek Road SE & Battle Creek Road SE
Salem, Oregon

CGT Project Number G1404007

Prepared for

Olsen Design & Development
Attn: Mr. Eric Olsen
170 West Main Street
Monmouth, Oregon 97361

May 23, 2014

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Dear Mr. Olsen:

Carlson Geotechnical (CGT), a division of Carlson Testing, Inc. (CTI), is pleased to submit this report summarizing the results of our preliminary geotechnical investigation, infiltration testing, and geologic assessment for the proposed Fairview Subdivision project. The project site encompasses approximately 50-acres and is located northeast of the intersection of Pringle Creek Road SE and Battle Creek Road SE in Salem, Oregon. We performed our work in general accordance with CGT Proposal GP6205.R2, dated April 17, 2014. Written authorization for our services was provided on April 17, 2014.

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

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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, infiltration testing, and geologic assessment for the proposed Fairview Subdivision project. The project site encompasses approximately 50-acres and is located northeast of the intersection of Pringle Creek Road SE and Battle Creek Road SE in Salem, Oregon, as shown on the attached Site Location, Figure 1.

1.1 Project Information

CGT developed our understanding of the planned project based on our correspondence with the project civil engineer, Westech Engineering (Westech), and review of a preliminary site (boundary) plan provided by Westech. We understand preliminary plans include developing the approximate 50-acre site into a residential subdivision. The locations of residential lots and new site roadways have not been defined at this time. Although no grading plans have been provided, we understand permanent grade changes will likely be relatively minimal, with cuts and fills limited to less than 5 feet in depth.

Storm water collected from new impervious surfaces at the site may be diverted into new storm water infiltration facilities. The type(s), location(s), and depth(s) of the infiltration facilities have not been determined at the time of this report. Design of the storm water facilities will rest with others. As part of preliminary planning, Westech requested twenty infiltration tests be performed at a maximum depth of 5 feet below existing site grades and spread relatively uniformly across the project site.

A geologic assessment is required for the project per provided correspondence with City of Salem.

1.2 Scope of Services

The purpose of our work was to explore subsurface conditions at the site in order to provide preliminary geotechnical engineering recommendations for design and construction of the proposed subdivision. In addition, our work included conducting infiltration tests at the site as requested by the project civil engineer. A geologic assessment was performed as required by the City of Salem. Our services are considered “preliminary” as layout and grading plans for the subdivision have not been developed. Our specific scope of services will include the following:

- Contact the Oregon Utilities Notification Center to mark the locations of public utilities at the site within a 15-foot radius of our planned exploration.
- Explore subsurface conditions at the site by excavating twenty test pits to depths up to about 10 feet below ground surface (bgs).
- Classify the soils encountered in the test pits in general accordance with ASTM D2488 (Visual-Manual Procedure).
- Perform twenty infiltration tests at the site (within the prepared test pits) at maximum depths of about 5 feet bgs in general accordance with the Encased Falling Head test method described in Section 4C.3(d) of the City of Salem Department of Public Works Administrative Rules Manual 109-001 (January 2014).
- Collect representative, disturbed samples of the soils encountered in the test pits in order to confirm our field classifications and perform laboratory testing.

- Perform laboratory testing on the soil samples obtained during site exploration to refine our field classifications.
- Provide preliminary geotechnical recommendations for site preparation and earthwork.
- Provide preliminary geotechnical engineering recommendations for design and construction of shallow spread foundations, floor slabs, and pavements.
- Provide preliminary recommendations for the Seismic Site Class, mapped maximum considered earthquake spectral response accelerations, and site seismic coefficients.
- Conduct a qualitative discussion of seismic hazards at the site, including liquefaction potential, slope instability, and surface rupture.
- Perform a geologic assessment of the project site in accordance with City of Salem Engineering Geology Report guidelines.
- Provide a written report summarizing the results of our investigation, infiltration testing, geologic assessment, and recommendations for the project.

2.0 SITE DESCRIPTION

2.1 Geologic Setting

A discussion of regional and local geology and seismic setting for the site is provided in the attached Appendix A.

2.2 Site Surface Conditions

The irregularly-shaped project site is bounded by Leslie Middle School and residential development to the north, a private roadway to the northeast, a grass field to the southeast, Battle Creek Road SE to the southwest, and Pringle Creek Road SE to the west. At the time of our field investigation, the majority of the project site was vacant of any structures and vegetated with grasses, brush (blackberry), and scattered coniferous trees. The northeast quadrant of the site contained several, abandoned, masonry buildings with appurtenant drive lanes. In terms of topography, the site was generally gently to moderately sloped to the north, with a total vertical relief of approximately 120 feet. Existing site features and topography are shown on the attached Site Plan, Figure 2. Photographs taken at the time of our field investigation are shown on the attached Site Photographs, Figure 3.

3.0 FIELD INVESTIGATION

3.1 Test Pits

CGT excavated twenty test pits (TP-1 through TP-20) at the site between April 29 and 30, 2014, to depths ranging from about 4 to 10 feet bgs. The test pits were excavated using a Bobcat E32, track-mounted excavator equipped with a 24-inch wide toothed bucket provided and operated by CGT. Upon completion of logging, the test pits were loosely backfilled with excavated materials.

The approximate locations of the test pits are shown on the attached Site Plan, Figure 2. The latitude and longitude for each test pit was determined using desktop GIS software and input into a handheld GPS receiver (Garmin GPSmap 60CSx) for use in locating in the field. The locations should be considered approximate within the accuracy of the GPS receiver, on average about 30 feet (+/-) as reported by the manufacturer.

3.2 Soil Classification & Sampling

A member of CGT's staff logged the soils observed within the test pits in general accordance with the Unified Soil Classification System (USCS) and collected representative disturbed (grab) samples of the materials encountered. An explanation of the USCS is presented on the attached Soil Classification Criteria and Terminology, Figure 4. Decomposed rock observed in the test pits was logged in accordance with the Oregon Department of Transportation (ODOT) Soil and Rock Classification Manual[1]. An explanation of rock classification is shown on the attached ODOT Rock Classification Criteria and Terminology, Figure 5. The soil and decomposed rock samples were stored in sealable plastic bags and transported to our laboratory for further examination and testing. Our geotechnical staff visually examined all samples returned to our laboratory in order to refine the field classifications. Logs of the test pits are presented on the attached Test Pit Logs, Figures 6 through 25. Surface elevations indicated on the logs were determined based on the provided topographic survey (reproduced and shown on the attached Figure 2). Elevations shown on the logs should be considered approximate.

3.3 Geologic Reconnaissance

CGT Project Engineering Geologist, Jeff Jones, CEG, performed a geologic reconnaissance of the project site on April 29, 2014. The results of the geologic reconnaissance and geologic assessment of the project site are presented in the attached Appendix A.

3.4 Infiltration Tests

CGT performed eighteen infiltration tests at the site within the prepared test pits on April 30, 2014. The complete results of the infiltration testing are presented in the attached Appendix B.

4.0 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 34 moisture content determinations (ASTM D2216), five percentage passing the U.S. Standard No. 200 Sieve tests (ASTM C117), and five Atterberg limits (plasticity) tests (ASTM D4318). Results of the laboratory tests are shown on the attached Test Pit Logs, Figures 6 through 25.

5.0 SUBSURFACE CONDITIONS

5.1 Soils

The following table presents a "checklist" of the subsurface materials encountered in the test pit explorations. Adjacent to those units, the tabulation presents an indicator (X) whether that subsurface material was encountered within the depth explored in the subject test pit.

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[1] Oregon Department of Transportation, 1987. Soil and Rock Classification Manual.

Table 1: Subsurface Material "Checklist"

Subsurface Material[1]	USCS	Test Pit Exploration																			
		TP-1	TP-2	TP-3	TP-4	TP-5	TP-6	TP-7	TP-8	TP-9	TP-10	TP-11	TP-12	TP-13	TP-14	TP-15	TP-16	TP-17	TP-18	TP-19	TP-20
Clay Topsoil	OL	X			X	X	X	X	X		X	X		X	X	X	X		X		
Undocumented Gravel Fill	GP-GC Fill, GW Fill			X														X			
Undocumented Lean Clay Fill	CL Fill									X											
Lean to Fat Clay	CL-CH	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X
Gravel with clay	GC							X	X												
Decomposed Basalt	RX	X		X	X	X	X									X	X	X			X

[1]Descriptions of each subsurface material are described below.

The following paragraphs provide a summary of the subsurface materials encountered at the site.

<u>Clay Topsoil (OL):</u> Clay topsoil was encountered at the surface of the referenced test pits and extended to depths of about ½- to ¾-foot bgs. The clay topsoil was generally medium stiff, brown, moist, rooted, and exhibited low plasticity.

<u>Gravel Fill (GP-GC Fill, GW Fill):</u> Undocumented gravel fill was encountered at the surface of the referenced explorations and extended to depths of about 1 foot bgs. Undocumented fill refers to materials placed without (available) documentation of subgrade conditions or evaluation of compaction. In TP-3, the gravel fill was generally medium dense, gray, moist, angular, relatively well-graded, and fine-grained. In TP-17, the gravel fill was generally medium dense, brown, moist, round, fine- to coarse-grained, and contained some clay.

<u>Lean Clay Fill (CL Fill):</u> Undocumented lean clay fill was encountered at the surface of TP-9 and extended to a depth of about 4 feet bgs. The lean clay fill was generally stiff, dark brown, moist, exhibited medium plasticity, and contained some angular gravel up to about 3 inches in diameter. The upper ½-foot of this soil contained fine roots.

<u>Lean to Fat Clay (CL-CH):</u> Lean to fat clay was encountered below the clay topsoil and fill materials in the referenced test pits. This soil extended to the full depths explored in TP-2, TP-9 through TP-14, TP-18, and TP-19. This soil extended to depths ranging from about 3 to 6 feet bgs in the remaining test pits. This soil was generally medium stiff to hard, brown, moist to wet, exhibited medium plasticity, and contained no to some weathered rock fragments up to about 2 feet in diameter. In TP-2 and TP-19, the upper 1-foot of this layer contained roots. In-situ moisture content of this soil generally ranged from 23 to 35 percent.

<u>Gravel with clay (GC):</u> Gravel with clay was encountered below the lean to fat clay in the referenced test pits (TP-7 and TP-8) and extended to the full depths explored, about 10 feet bgs. The gravel was generally loose to medium dense, brown and gray, wet, round, and fine- to coarse-grained (up to 3 inches in diameter).

Decomposed Basalt (RX): Decomposed basalt was encountered below the lean to fat clay in the referenced test pits and extended to the full depths explored, up to about 10 feet bgs. In TP-3, TP-4, TP-19, and TP-20, practical refusal of the excavator (Bobcat E32) was encountered on this material due to hard digging conditions or presence of a large (boulder-sized) rock fragment. This material was generally very soft (R1), brown to orange-brown to black, and moist. In-situ moisture content of this material generally ranged from 29 to 47 percent.

5.2 Groundwater

Groundwater seepage was encountered at depths of about 4 to 9½ feet bgs in test pits TP-7 through TP-9. Groundwater was not encountered within the depths explored in the remaining test pits. To determine approximate regional groundwater levels in the area, we researched well logs available at the Oregon Water Resources Department (OWRD)[2] website for wells located within Section 11, Township 8 South, Range 3 West. Our review indicated that groundwater levels were highly variable and ranged from about 40 to 200 feet bgs in the vicinity of the site. It should be noted that 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 test pits 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 groundwater levels at the site will fluctuate due to seasonal and annual variations in precipitation, changes in site utilization, or other factors. Additionally, the lean clay fill (CL Fill), native lean to fat clay (CL-CH), and decomposed basalt (RX) are conducive to formation of perched groundwater.

6.0 SEISMIC CONSIDERATIONS

6.1 Seismic Hazards

6.1.1 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[3] to identify non-liquefiable, fine-grained soils is based on plasticity

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[2] Oregon Water Resources Department, 2014. Water well logs obtained from OWRD website http://www.wrd.state.or.us/

[3] 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.

characteristics of the soils, as follows: (1) liquid limit greater than 47 percent, (2) plasticity index greater than 20 percent, and (3) moisture content less than 85 percent of the liquid limit. Soils identified as susceptible to liquefaction are analyzed using the industry standard "simplified procedure", originally published by Seed and Idriss[4] in 1971 and updated continually since that time. 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).

Based on the lack of saturated conditions and medium plasticity, the soils encountered within test pits TP-1 through TP-6 and TP-10 through TP-20 are considered non-liquefiable within the depths explored. Based on its medium plasticity, the lean to fat clay encountered within test pits TP-7 through TP-9 is considered non-liquefiable within the depths explored. Based on its generally medium dense relative density, and review of hazard mapping (presented in the attached Appendix A), the native gravel with clay (GC) is considered to have a very low to negligible potential for liquefaction for a design-level seismic event.

6.1.2 Slope Instability

Opinions related to seismically-induced slope instability are presented in Section A.4.3.1 of the attached Appendix A.

6.1.3 Surface Rupture

6.1.3.1 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.

6.1.3.2 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. Given the lack of liquefiable soils at the site, the risk of lateral spread is considered negligible.

6.2 Seismic Site Class

Based on the results of the explorations and review of geologic mapping, we have assigned the site as Site Class D for the subsurface conditions encountered in accordance with Table 1613.5.2 of the 2010 Oregon Structural Specialty Code (OSSC), referenced from Section R301.2.2.1.1 of the 2011 Oregon Residential Specialty Code (ORSC). Recommendations for seismic ground motion values at the site are presented in Section 10.3 of this report.

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[4] Seed, H.B., and Idriss, I.M., 1971, Simplified Procedure for Evaluating Soil Liquefaction Potential, Journal of Geotechnical Engineering Division, ASCE, 97(9), 1249-1273.

7.0 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, and pavements can be obtained from the native, medium stiff to better, lean to fat clay (CL-CH), the native, medium dense, gravel with clay (GP-GC), the decomposed basalt (RX), or structural fill that is properly placed and compacted on these materials during construction. These soils were first encountered at depths of about ½ to 1 foot bgs within the test pits.

Where encountered at design subgrade elevations for shallow foundations, floor slabs, pavements, or structural fills, existing fill materials (CL Fill, GP-GC Fill, and GW Fill) should be completely over-excavated and replaced with structural fill. These materials may be re-used as structural fill at the site, provided they are prepared in conformance with Section 8.4.1 of this report.

Due to their fine-grained nature, the near surface lean to fat clay (CL-CH) and decomposed basalt (RX) 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, we recommend measures be implemented to protect the fine-grained subgrade in areas of repeated construction traffic and in foundation bearing areas. Geotechnical recommendations for wet weather construction are presented in Section 8.3 of this report. Re-use of these soils as structural fill during wet times of the year will require special consideration as discussed in Section 8.4.2 of this report.

8.0 RECOMMENDATIONS: SITE PREPARATION & EARTHWORK

The recommendations presented below are provided for general planning purposes and are subject to revision once layout and grading plans for the project are further developed. Our preliminary recommendations are based on the information provided to us, results of the field investigation, 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 and/or location of the proposed development changes, or variations and/or undesirable geotechnical conditions are encountered during site development.

8.1 Site Preparation

8.1.1 Site Stripping

Surface vegetation, clay topsoil (OL), rooted soils, and undocumented fill (if encountered) should be removed from within, and for a 5-foot margin around, the planned structural fill, building pad, and pavement locations. Based on the results of the field explorations, stripping depths at the site are anticipated to range from about ½- to 1-foot bgs. In the area of TP-9, stripping depths are anticipated to be about 4 feet to remove the existing lean clay fill. These materials may be shallow or deeper away from the exploration locations. The geotechnical engineer or his representative should provide recommendations for actual stripping depths based on observations during site stripping. Stripped surface vegetation and rooted soils should be transported off-site for disposal, or stockpiled for later use in landscaped areas.

8.1.2 Grubbing

Grubbing of trees and shrubs (where slated for removal) 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 moderate to large 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 8.4 of this report.

8.1.3 Existing Utilities & Below-Grade Structures

All existing utilities at the site should be identified prior to excavation. Abandoned utility lines beneath new structures, 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 as described in Section 8.4 of this report. No below-grade structures were encountered during our field investigation. If encountered, buried structures (i.e. footings, foundation walls, retaining walls, slabs-on-grade, tanks, etc.) encountered during site preparation should be completely removed and replaced with structural fill in conformance with Section 8.4 of this report.

8.1.4 Test Pits

The test pits conducted at the site were loosely backfilled during our field investigation. Where test pits are located within finalized structural fill, building, or pavement areas, the loose backfill materials should be re-excavated. The resulting excavations should be backfilled with structural fill placed and compacted in general accordance with Section 8.4 of this report.

8.1.5 Subgrade Preparation – Pavement Areas & Residential Lots to Receive Structural Fill

8.1.5.1 Dry Weather Construction

After site preparation as recommended above, but prior to placement of structural fill and/or base rock, the geotechnical engineer or his representative should observe a proof roll test of the exposed subgrade soils in order to identify areas of excessive yielding. Proof rolling of subgrade soils is typically conducted during dry weather conditions using a fully-loaded, 10- to 12-cubic-yard, tire-mounted, dump truck or equivalent weighted water truck. Areas that appear too soft and wet to support proof rolling equipment should be prepared in general accordance with the recommendations for wet weather construction presented in Section 8.3 of this report. If areas of soft soil or excessive yielding are identified, the affected material should be over-excavated to firm, stable subgrade, and replaced with imported granular structural fill in conformance with Section 8.4.3 of this report.

8.1.5.2 Wet Weather Construction

Preparation of residential lot and pavement subgrade soils during wet weather should be in conformance with Section 8.3 of this report. As indicated therein, increased base rock sections and a geotextile separation fabric may be required in wet conditions in order to support construction traffic and protect the subgrade. Cement amendment may also be considered to help stabilize subgrade soils during wet weather.

8.1.6 Erosion Control

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

8.2 Temporary Excavations

8.2.1 Overview

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.

8.2.2 Dewatering

8.2.2.1 Site Areas near Test Pits TP-7 and TP-8

As indicated in Section 5.2 above, we encountered groundwater at depths of about 4 to 5 feet bgs in test pits TP-7 and TP-8, and a depth of about 9½ feet bgs in test pit TP-9. We anticipate dewatering of excavations within this area of the site will be required in order to maintain dry working conditions, particularly in the area of test pits TP-7 and TP-8. At those locations, the soils at depth are primarily gravelly with low fines content and are anticipated to have high rates of transmissivity. Therefore, we would expect moderate to rapid seepage to occur during excavation.

Pumping from sumps may be effective in removing groundwater within shallow or localized excavations in this area of the site. Pumping from multiple well points will likely be required for larger excavations. The sumps or wells should be installed to remove water to a depth of at least 2 feet below the lowest elevation of the excavation, and should be installed and put into operation prior to commencing excavation. The project civil engineer should determine the appropriate size, number, and location of sump pumps or wells, and also evaluate requirements for disposal of the resultant discharge.

In order to refine groundwater levels and estimate flow rates, piezometers or well points could be installed and drawdown tests could be performed prior to, or at the onset of, construction. At a minimum, prior to the start of significant excavations, we recommend test pits be excavated at the site to observe groundwater levels. The geotechnical engineer or his representative should be onsite to observe the excavation of the test pits. Additional recommendations for dewatering plans could be developed following seepage observations and/or drawdown tests.

8.2.2.2 Other Site Areas

Based on the results of the test pits, we do not anticipate that site excavations extending to depths less than 10 feet will require area-wide dewatering during construction. Temporary dewatering of utility trenches and other localized excavations may be required in the event perched groundwater is encountered. We anticipate pumping from sumps should be effective in removing perched groundwater at the site. Disposal locations should be reviewed by the project civil engineer. If groundwater seepage is encountered on temporary cut slopes during construction, provisions may be required to collect and divert the water from the cut slope and reduce the potential of instability. The geotechnical engineer should be consulted in the event groundwater seepage emerges within cut slopes.

8.2.3 Utility Trenches

Temporary trench cuts should stand near vertical to depths of approximately 4 feet in the native soils encountered at the site. If seepage undermines the stability of the trench, or if caving of the sidewalls is observed during excavation, the sidewalls should be flattened or shored. Trench dewatering may be required to maintain dry working conditions, as discussed in Section 8.2.2 above. If groundwater is present at the base of utility excavations, we recommend placing trench stabilization material at the base of the excavations. Trench stabilization material should be in conformance with Section 8.4.5 of this report.

8.2.4 OSHA Soil Type

8.2.4.1 Lean Clay to Fat Clay (CL-CH)

Conventional earthmoving equipment in proper working condition should be capable of making cuts within this soil. For use in the planning and construction of temporary excavations at the site, an OSHA soil type “B” may be used for this soil.

8.2.4.1 Gravel with clay (GP-GC)

Conventional earthmoving equipment in proper working condition should be capable of making cuts within this material. For use in the planning and construction of temporary excavations at the site, an OSHA soil type “C” should be used for this material.

8.2.4.2 Decomposed Basalt (RX)

As indicated earlier, we encountered practical refusal of the excavating equipment (Bobcat E32 excavator with 2-foot-wide toothed bucket) during excavation of test pits TP-3, TP-4, TP-19, and TP-20, due to hard digging conditions or presence of a large (boulder-sized) rock fragment. Based on experience in the area, we anticipate larger excavating equipment in proper working condition should be capable of making cuts in the decomposed basalt. Although not anticipated, hydraulic hammering may be required for excavation and removal of lesser weathered basalt, if encountered in deeper site excavations. An OSHA soil type “A” may be used when considering temporary excavations into the decomposed basalt.

8.2.5 Excavations Near Foundations

Excavations near footings should <u>not</u> extend within a 1H:1V (horizontal:vertical) plane projected out and down from the outside, bottom edge of the footings. In the event that 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.

8.3 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 the middle of September. Notwithstanding the above, soil conditions should be evaluated in the field by the geotechnical engineer or his representative at the initial stage of site preparation to determine whether the recommendations within this section should be incorporated into construction.

8.3.1 Overview

The near-surface, native lean clay to fat clay (CL-CH) and decomposed basalt (RX) are 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 construction that occurs during wet weather, 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. A geotechnical representative from CGT 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, stable subgrade, and replaced with imported granular structural fill.

8.3.2 Geotextile Separation Fabric

We recommend a geotextile separation fabric be placed to serve as a barrier between the prepared fine-grained subgrade and granular fill/base rock in areas of repeated or heavy construction traffic. The geotextile fabric should be in conformance with Section 02320 of the current Oregon Department of Transportation (ODOT) Standard Specification for Construction. In accordance with Table 02320-1 of ODOT specifications, the separation fabric should have minimum puncture strength (ASTM D4833) of 80 pounds and an apparent opening size (ASTM D4751) no larger than the U.S. Standard No. 30 sieve. Examples of products that currently meet these requirements include Propex Geotex 200ST and US Fabrics US200. Other products meeting the requirements presented by ODOT may be considered for separation geotextile fabric.

8.3.3 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 should be sufficient. Additional granular material, geo-grid reinforcement, or cement amendment 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 8.4.3 of this report and have less than 5 percent material passing the U.S. Standard No. 200 Sieve. The prepared subgrade should be covered with geotextile fabric 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.

8.3.4 Footing Subgrade Protection

A minimum of 3 inches of imported granular material is recommended to protect fine-grained footing subgrades from foot traffic during inclement weather. The imported granular material should be in conformance with Section 8.4.3 of this report, have less than 5 percent material passing the U.S. Standard No. 200 Sieve, and have a maximum particle size 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.

8.3.5 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 native lean clay to fat clay (CL-CH) is conducive for cement amendment due to its medium plasticity and experience with similar soils. If cement amendment is considered for the project, the geotechnical engineer should be consulted to provide supplemental recommendations for testing (mix design), cement percentage, and other considerations. We recommend project scheduling allow for a minimum of 2 weeks to conduct the mix design and development of specific recommendations for construction.

8.4 Structural Fill

8.4.1 Overview

On-site or imported materials intended for use as structural fill at the site should be evaluated and accepted by the geotechnical engineer prior to placement. The geotechnical engineer or his 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, deflection (proof roll) tests, or other testing methods accepted by the geotechnical engineer. The following table presents recommended guidelines for frequency of density testing (where practical) of various fill designations.

Table 2: Recommended Guidelines for Frequency of Density Testing

Fill Designation	Recommended Frequency of Density Tests	
	Maximum Depth Interval	Area-Wide
General Structural Fill (Mass Grading)	Test every 2 vertical feet	At least one density test per 2,000 feet^2 of fill area
Utility Trench Backfill$^\alpha$	Test every 2 vertical feet	At least one density test per 50 feet of trench line
Pavement Base Rock$^\alpha$	Test at surface of section	At least one density test per 2,000 feet^2 of base rock area
Floor Slab Base Rock	Test at surface of section	At least one density test per 1,000 feet^2 of base rock area

$^\alpha$Testing frequency within the public right-of-way should be in conformance with the local jurisdiction requirements.

8.4.2 On-Site Soils – General Use

8.4.2.1 Lean to Fat Clay (CL-CH), Lean Clay Fill (CL Fill)

Re-use of these soils as structural fill may be difficult because these soils are sensitive to small changes in moisture content and are difficult, if not impossible, to adequately compact during wet weather. We anticipate the moisture content of these soils 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, these soils should be free of organic matter, debris, and particles larger than 4 inches. When used as structural fill, these soils should be placed in lifts with a maximum 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).

8.4.2.2 Decomposed Basalt

Re-use of this material as structural fill may be difficult because this material often "breaks down" into soils exhibiting relatively high fines content. Soils with high fines content, such as those discussed in Section 8.4.2.1, are sensitive to small changes in moisture content and are difficult, if not impossible, to adequately compact during wet weather. We anticipate the moisture content of this material will be higher than the optimum moisture content for satisfactory compaction. Therefore, moisture conditioning (drying) should be expected in order to achieve adequate compaction.

In cases where moderate to large sized, powered compaction equipment (e.g. sheepsfoot roller, smooth drum roller, vibratory hoe-pack compactor) is intended for compaction of this material, we do not anticipate processing (such as crushing or blending with imported material) will be required to render the material in a condition suitable for use as structural fill. This judgment is based on experience with similar materials in the region and expectation that gravel- and cobble-sized fragments (if present) will "break down" to resemble a silty, clayey, gravelly matrix during normal compaction operations. Although not anticipated, in the event particles in excess of 4 inches in diameter remain within the fill material following application of normal compaction effort, we recommend those particles be processed ("picked free") from the fill material. The geotechnical engineer or his representative should be contacted to observe conditions of each lift of fill material following application of compaction effort.

In cases where limited access and/or other factors preclude the use of moderate to large sized, powered compaction equipment, and cobble-sized (or possibly boulder-sized) fragments exist within the material, we anticipate processing will be required to render the material in a condition suitable for use as structural fill. We recommend the geotechnical engineer be consulted to review the proposed application of the material and intended equipment in order to provide specific recommendations.

When used as structural fill, this soil should be placed in lifts with a maximum thickness of about 12 inches at moisture contents within –1 and +3 percent of optimum, and compacted to not less than 95 percent of the material's maximum dry density as determined in accordance with ASTM D1557 (Standard Proctor). Where the material contains a high concentration of over-sized particles, thereby precluding conventional density testing, evaluation of relative compaction should be performed by deflection (proof roll) testing in accordance with ODOT Test Method TM 158.

8.4.2.1 Gravel Fill (GP-GC Fill, GW Fill), Native Gravel with clay (GP-GC)

Re-use of these materials as structural fill is feasible, provided they can be kept free of debris, deleterious materials, and particles larger than 4 inches in diameter. If used as structural fill, this material should be prepared in conformance with Section 8.4.3 of this report.

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

8.4.3 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. 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 this material.

Compaction of granular fill materials with high percentages of particle sizes in excess of 1½-inches should be evaluated by periodic proof-roll observation or continuous observation by the CGT geotechnical representative during fill placement, since it cannot be tested conventionally using a nuclear densometer. Such materials should be "capped" with a minimum of 12 inches of 1½-inch-minus (or finer) granular fill under all structural elements (footings, concrete slabs, etc.).

8.4.4 Floor Slab 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).

8.4.5 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.

8.4.6 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 3: Utility Trench Backfill Compaction Recommendations

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

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

8.4.7 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, State of Oregon, Standard Specifications for Highway Construction. 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.

8.5 Permanent Slopes

8.5.1 Overview

Permanent cut or fill slopes constructed at the site 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.

8.5.2 Placement of Fill on Slopes

New fill should be placed and compacted against horizontal surfaces. Where slopes exceed 5H:1V (horizontal to vertical), the slopes should be keyed and benched prior to structural fill placement in general accordance with the attached Fill Slope Detail, Figure 26. If subdrains are needed on benches, subject to the review of the geotechnical engineer or his representative, they should be placed as shown on the attached Fill Slope Detail. In order to achieve well compacted slope faces, slopes should be overbuilt by a few feet and then trimmed back to proposed final grades. The geotechnical engineer or his representative should observe the benches, keyways, and associated subdrains, if needed, prior to placement of structural fill.

8.6 Additional Considerations

8.6.1 Drainage

Subsurface drains should be connected to the nearest storm drain, on-site stormwater infiltration facility (designed by others), or other suitable discharge point. Paved surfaces and ground near or adjacent to the residential 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 onto site slopes or into foundation drains, if incorporated.

The relatively low infiltration rates observed during field testing (Appendix B) are anticipated to present challenges for design of stormwater infiltration systems at the site. Design of stormwater management plans will rest with others. If further site characterization and testing is required for design, CGT would be pleased to provide additional geotechnical services upon request.

8.6.2 Expansive Potential

The near surface soils consist of generally medium plasticity, lean to fat clay (CH). Based on experience, 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 this site.

8.6.3 Freezing Weather Considerations

For construction that occurs during extended periods of sub-freezing temperatures, the following special provisions are recommended:

- Structural fill should not be placed over frozen ground.
- Frozen soil should not be placed as structural fill.
- Fine-grained soils should not be placed as structural fill during sub-freezing temperatures.

Identification of frozen soils at the site should be in accordance with ASTM D4083-01 “Standard Practice for Description of Frozen Soils (Visual-Manual Procedure)” or other approved method. The geotechnical engineer can aid the contractor with supplemental recommendations for earthwork that will take place during extended periods of sub-freezing weather, as required.

8.6.4 Below-Grade Tunnel Structures (if encountered)

Although not encountered during our investigation, anecdotally, we understand there may be below-grade tunnel structures near the existing buildings at the northeast portion of the site. Supplemental geotechnical investigation (geophysical surveys) could be performed to help refine the presence (or lack thereof) of tunnel structures at the site. If below-grade tunnel(s) are encountered during site preparation, the geotechnical engineer should be engaged to help develop plans for mitigation on a case-by-case basis. For preliminary planning and consideration, we have presented two options for the owner to consider in the case that tunnel structures are encountered at the site.

8.6.4.1 Option 1 – Remove & Replace with Structural Fill

This option would include demolishing and removing the below-grade tunnel structure in its entirety and backfilling the resulting excavation with structural fill in conformance with Section 8.3 of this report. The geotechnical engineer or his representative should observe finished excavation conditions following removal, prior to placement of structural backfill.

8.6.4.2 Option 2 – Infilling with CLSM

This option would include infilling the below-grade tunnel structure with CLSM in conformance with Section 8.4.7 of this report. If this approach is considered, the geotechnical engineer and civil engineer should be consulted to review the conditions of the tunnel structure and review the suitability of infilling with respect to improvements (e.g. utilities, foundations, etc.) planned in close proximity to the tunnel structure.

9.0 PRELIMINARY RECOMMENDATIONS: PAVEMENTS

9.1 Subgrade Preparation

Subject to review of the pavement designer, we recommend subgrade preparation of pavements be in conformance with Section 8.1.5 of this report. Pavement subgrade surfaces should be crowned (or sloped) for proper drainage in accordance with specifications provided by the project civil engineer.

9.2 Design Sections

Pavement section design was not included in this assignment. At the time this report was prepared, it was our understanding pavement design will rest with others. CGT would be pleased to provide geotechnical recommendations for design and construction of pavements at this project, upon request, for an additional fee.

10.0 PRELIMINARY RECOMMENDATIONS: STRUCTURAL DESIGN

10.1 Shallow Spread Foundations

10.1.1 Subgrade Preparation

Satisfactory subgrade support for shallow foundations associated with the planned residential buildings can be obtained from the native, medium stiff to better, lean to fat clay (CL-CH), the native decomposed basalt (RX), the native, medium dense, gravel with clay (GC), or structural fill that is properly placed and compacted on these materials during construction. The geotechnical engineer or his representative should be contacted to observe subgrade conditions prior to placement of forms, reinforcement steel, or structural fill (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 8.4.3 of this report. 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.

10.1.2 Minimum Footing Width & Embedment

Minimum footing widths should be in conformance with the most recent, Oregon Residential Structural Code (ORSC). As a guideline, CGT recommends individual spread footings have a minimum width of 24 inches. For one-story, light-framed structures, we recommend continuous wall footings have a minimum width of 12 inches. Similarly, for two- and three-story, light-framed structures, we recommend continuous wall footings have a minimum width of 15 and 18 inches, respectively. All footings should be founded at least 18 inches below the lowest, permanent adjacent grade.

10.1.3 Foundation Setback from Descending Slopes

We recommend foundations include a minimum setback of 5 feet be maintained near descending site slopes. 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.

10.1.4 Bearing Pressure & Settlement

Footings founded as recommended above should be proportioned for a maximum allowable soil bearing pressure of 2,000 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.

10.1.5 Lateral Capacity

A maximum passive (equivalent fluid) earth pressure of 150 pounds per cubic foot (pcf) is recommended for design of footings confined by the native soils described above, or 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.

10.1.6 Subsurface Drainage

Recognizing the predominantly fine-grained nature of the site soils, placement of perimeter foundation drains is recommended at the base elevations of continuous wall footings on the outside of footings. Foundation 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 filter fabric in order to provide separation from the surrounding soils. Foundation drains should be positively sloped and should outlet to a suitable discharge point. The geotechnical engineer or his representative should be contacted to observe the drains prior to backfilling. Roof drains should not be tied into foundation drains.

10.2 Floor Slabs

10.2.1 Subgrade Preparation

Satisfactory subgrade support for floor slabs constructed on grade, supporting up to 150 psf area loading, can be obtained from the native, medium stiff to better, lean to fat clay (CL-CH), the native decomposed basalt (RX), the native, medium dense, gravel with clay (GC), or structural fill that is properly placed and compacted on these materials during construction. The geotechnical engineer or his 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 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 8.4.3 of this report.

10.2.2 Crushed Rock Base

Concrete floor slabs should be supported on a minimum 6-inch-thick layer of crushed rock (base rock) in conformance with Section 8.4.4 of this report. For design cases where a vapor barrier or retarder is not placed below the slab, we recommend “choking” the surface of the base rock with fine 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.

10.2.3 Design Considerations

For floor slabs constructed as recommended, a modulus of subgrade reaction of 100 pounds per cubic inch (pci) is recommended for the design of the floor slab. 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.

10.2.4 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.

10.3 Seismic Design

As indicated in Section 6.2 of this report, the site was assigned as Site Class "D". Seismic ground motion values for the site were determined in accordance with Section R301.2.2 of the 2011 ORSC. Earthquake ground motion parameters for the site were obtained based on the United States Geological Survey (USGS) Seismic Design Values for Buildings - Ground Motion Parameter Calculator[5]. The site Latitude 44.896043° North and Longitude 123.021424° West were input as the site location. The following table shows the recommended seismic design parameters for the site.

Table 4: Seismic Ground Motion Values

Parameter		Value
Mapped Acceleration Parameters	Spectral Acceleration, 0.2 second (S_s)	0.856g
	Spectral Acceleration, 1.0 second (S_1)	0.420g
Coefficients (Site Class D)	Site Coefficient, 0.2 sec. (F_A)	1.158
	Site Coefficient, 1.0 sec. (F_V)	1.580
Adjusted MCE Spectral Response Parameters	MCE Spectral Acceleration, 0.2 sec. (S_{MS})	0.991g
	MCE Spectral Acceleration, 1.0 sec. (S_{M1})	0.663g
Design Spectral Response Accelerations	Design Spectral Acceleration, 0.2 seconds (S_{DS})	0.661g
	Design Spectral Acceleration, 1.0 second (S_{D1})	0.442g

11.0 RECOMMENDED ADDITIONAL SERVICES

11.1 Design Review

Geotechnical design review is of paramount importance. CGT recommends that the geotechnical design review take place prior to releasing bid packets to contractors.

11.2 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

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[5] United States Geological Survey, 2014. Seismic Design Parameters determined using:, "U.S. Seismic Design Maps Web Application - Version 3.1.0," from the USGS website http://earthquake.usgs.gov.

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 the geotechnical engineer or his representative attend a pre-construction meeting coordinated by the contractor and/or developer. The project geotechnical engineer or their representative should provide observations and/or testing of at least the following earthwork elements during construction:

- Stripping & Demolition of Existing Structures
- Subgrade Preparation for Structural Fills, Shallow Foundations, and Pavements
- Compaction of Structural Fill
- Compaction of Utility Trench Backfill
- Placement of Foundation Drains and Other Drains
- Compaction of Base Rock for Pavements
- Compaction of Asphaltic Concrete for Pavements.

It is imperative 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.

12.0 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 not intended to be, nor should they be construed as a warranty of subsurface conditions, but are forwarded to assist in the planning and design process.

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.

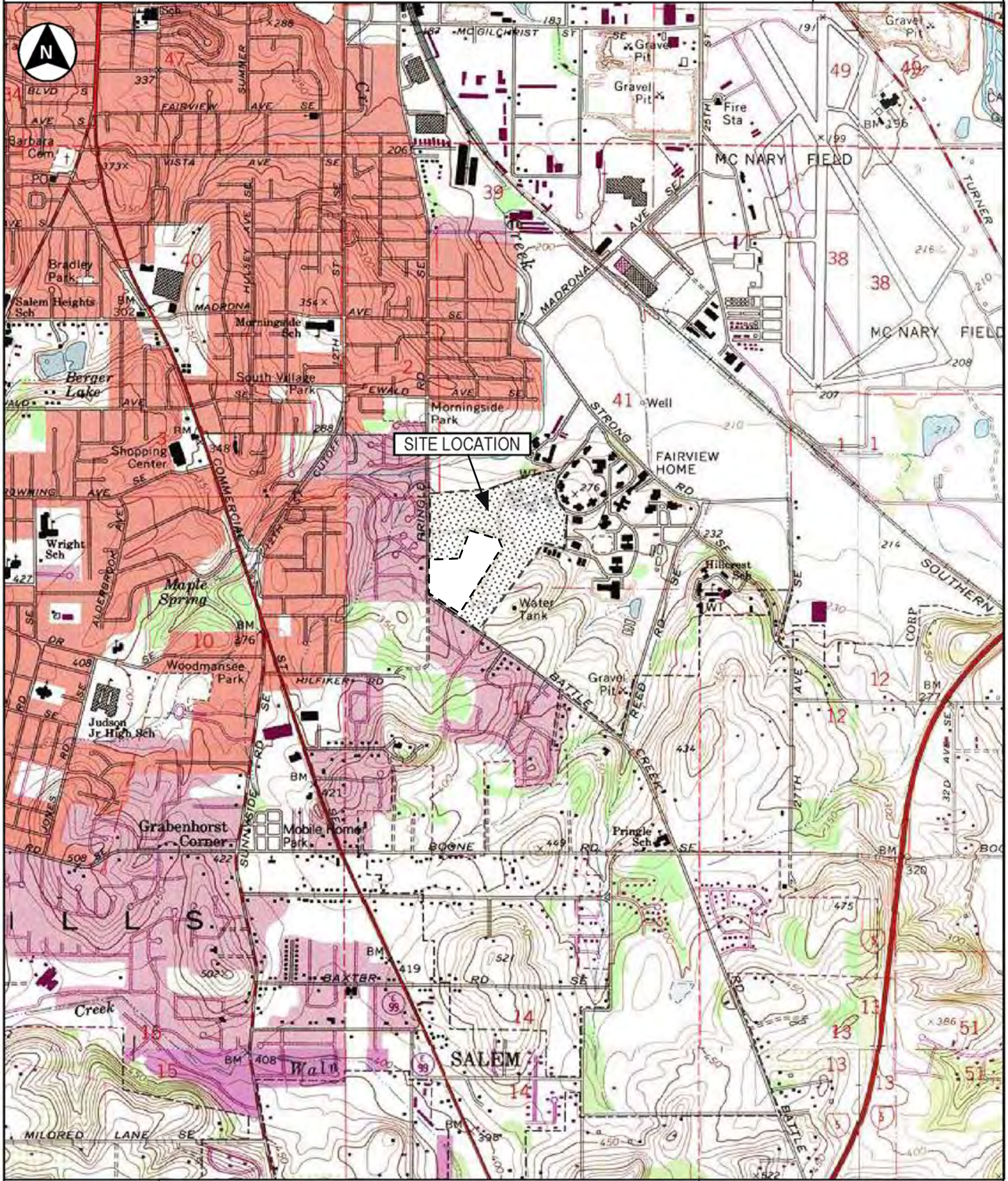
The scope of our services does not include services related to construction safety precautions, and our recommendations are not intended to direct the contractor's methods, techniques, sequences, or procedures, except as specifically described in our report for consideration in design.

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

FAIRVIEW SUBDIVISION - SALEM, OREGON
Job No. G1404007

FIGURE 1

Site Location



Map created with TOPO!™, © 2006 National Geographic Holdings
USGS 7.5 Minute Topographic Map Series, Salem West, OR Quadrangle.

Township 8 South, Range 3 West, Section 11 Willamette Meridian

CARLSON CGT GEOTECHNICAL 503-601-8250

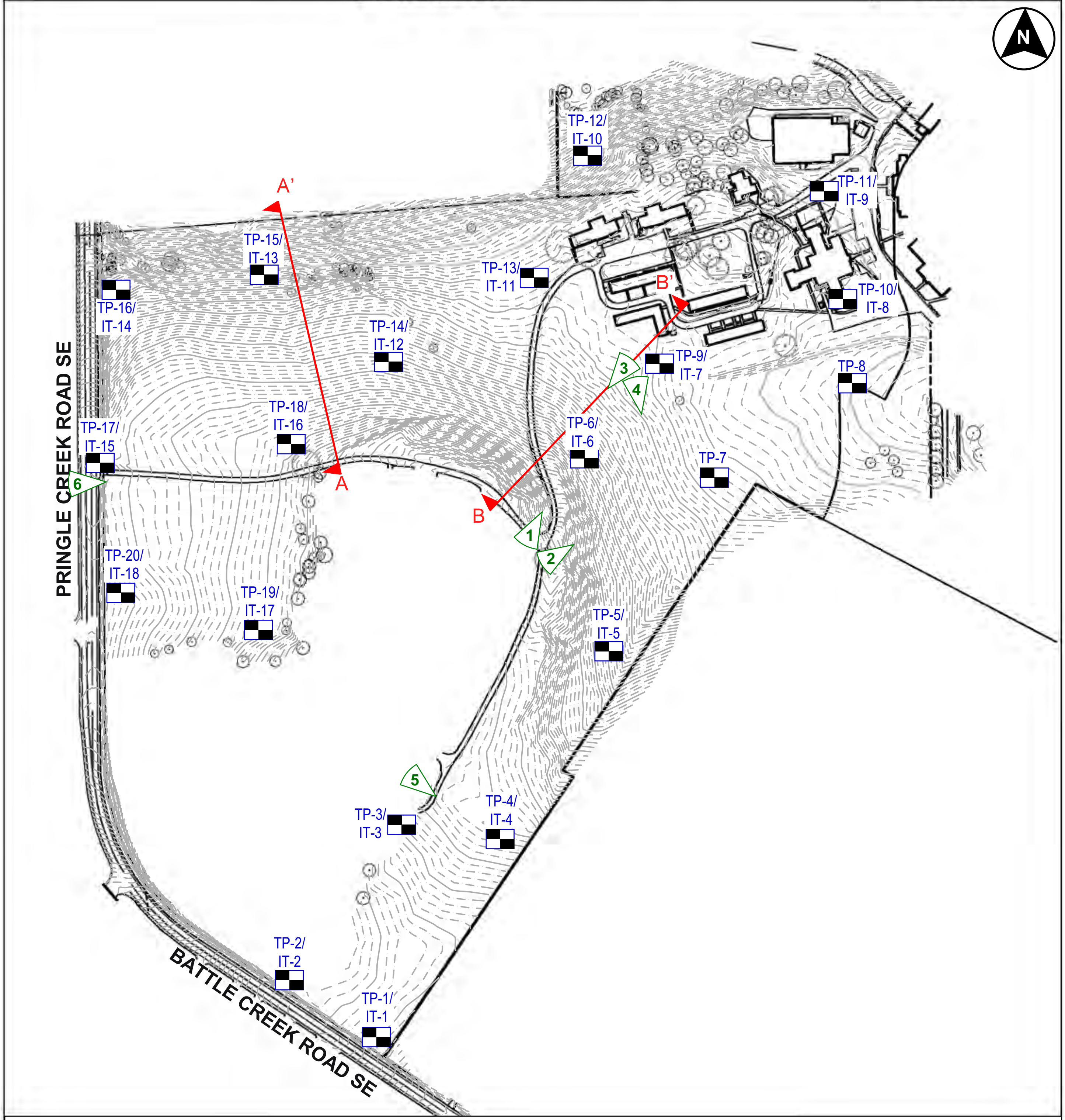
Latitude 44.896043° N
Longitude -123.021424° W

Scale 1 Inch = 2,000 feet

0 2000 4000

FAIRVIEW SUBDIVISION - SALEM, OREGON
Job No. G1404007

FIGURE 2
Site Plan



LEGEND

TP-1/IT-1 ▨ Test pit exploration and infiltration test

△1 Orientation of site photographs shown on Figure 3

Geologic cross section shown on Figure A5 of Appendix A

CARLSON
CGT
GEOTECHNICAL
503-601-8250

NOTES:
Base site plan provided by Westech Engineering. Drawing was reproduced and modified by CGT staff. All exploration and photograph locations should be considered approximate.

Approximate Scale: 1 Inch = 350 Feet
0 350 700

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Job No. G1404007

FIGURE 3

Site Photographs


Photograph 1


Photograph 2


Photograph 3


Photograph 4


Photograph 5


Photograph 6

CARLSON CGT GEOTECHNICAL 503-601-8250

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

FAIRVIEW SUBDIVISION - SALEM, OREGON
Project Number G1404007

FIGURE 4

USCS

Classification of Terms and Content

NAME: MINOR Constituents (12-50%); MAJOR Constituents (>50%); Slightly (5-12%)
Relative Density or Consistency
Color
Moisture Content
Plasticity
Trace Constituents (0-5%)
Other: Grain Shape, Approximate Gradation, Organics, Cement, Structure, Odor...
Geologic Name or Formation: Fill, Willamette Silt, Till, Alluvium, etc.

USCS Grain Size

Fines		<#200 (.075 mm)
Sand	Fine Medium Coarse	#200 - #40 (.425 mm) #40 - #10 (2 mm) #10 - #4 (4.75)
Gravel	Fine Coarse	#4 - 0.75 inch 0.75 inch - 3 inches
Cobbles		3 to 12 inches; scattered <15% est. numerous >15% est.
Boulders		> 12 inches

Relative Density or Consistency

Granular Material		Fine-Grained (cohesive) Materials				
SPT N-Value	Density	SPT N-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
0 - 4	Very Loose	2 - 4	0.13 - 0.25	0.25 - 0.50	Soft	Thumb penetrates about 1 inch
4 - 10	Loose	4 - 8	0.25 - 0.50	0.50 - 1.00	Medium Stiff	Thumb penetrates about ¼ inch
10 - 30	Medium Dense	8 - 15	0.50 - 1.00	1.00 - 2.00	Stiff	Thumb penetrates less than ¼ inch
30 - 50	Dense	15 - 30	1.00 - 2.00	2.00 - 4.00	Very Stiff	Readily indented by thumbnail
>50	Very Dense	>30	>2.00	>4.00	Hard	Difficult to indent by thumbnail

Moisture Content

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

Damp: Some moisture but leaves no moisture on hand

Moist: Leaves moisture on hand

Wet: Visible free water, likely from below water table

	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

Unified Soil Classification Chart (Visual-Manual Procedure) (Similar to ASTM Designation D-2487)

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 silt and organic silty clays of low plasticity
	Silt and Clays High Plasticity Fines		MH	Inorganic silts, clayey silts
			CH	Inorganic clays of high plasticity, fat clays
			OH	Organic clays of medium to high plasticity
Highly Organic Soils			PT	Peat, muck, and other highly organic soils

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Additional References:
ASTM D2487 Standard Practice for Classification of Soils for Engineering Purposes) and
ASTM D2488 Standard Practice for Description and Identification of Soils (Visual-Manual Procedure)

FAIRVIEW SUBDIVISION - SALEM, OREGON
Project Number G1404007

FIGURE 5
ODOT

Table 22: Scale of Relative Rock Weathering

Designation	Field Identification
Fresh	Crystals are bright. Discontinuities may show some minor surface staining. No discoloration in rock fabric.
Slightly Weathered	Rock mass is generally fresh. Discontinuities are stained and may contain clay. Some discoloration in rock fabric. Decomposition extends up to 1-inch into rock.
Moderately Weathered	Rock mass is decomposed 50% or less. Significant portions of rock show discoloration and weathering effects. Crystals are dull and show visible chemical alteration. Discontinuities are stained and may contain secondary mineral deposits.
Predominantly Weathered	Rock mass is more than 50% decomposed 50%. Rock can be excavated with geologist's pick. All discontinuities exhibit secondary mineralization. Complete discoloration of rock fabric. Surface of core is friable and usually pitted due to washing out of highly altered minerals by drilling water.
Decomposed	Rock mass is completely decomposed. Original rock fabric may be evident. May be reduced to soil with hand pressure.

Table 23: Scale of Relative Rock Hardness

Term	Hardness Designation	Field Identification	Approximate Unconfined Compressive Strength
Extremely Soft	R0	Can be indented with difficulty by thumbnail. May be moldable or friable with finger pressure.	<100 psi
Very Soft	R1	Crumbles under firm blows with point of geology pick. Can be peeled by pocket knife. Scratched with finger nail.	100-1000 psi
Soft	R2	Can be peeled by pocket knife with difficulty. Cannot be scratched with finger nail. Shallow indention made by firm blow of geology pick.	1000-4000 psi
Medium Hard	R3	Can be scratched by knife or pick. specimen can be fractured with a single firm blow of hammer/geology pick.	4000-8000 psi
Hard	R4	Can be scratched with knife or pick only with difficulty. Several hard blows required to fracture specimen.	8000-16000 psi
Very Hard	R5	Cannot be scratched by knife or sharp pick. Specimen requires many blows of hammer to fracture or chip. Hammer rebounds after impact.	>16000 psi

Table 24: Stratification Terms

Term	Characteristics
Laminations	Thin beds (<1cm).
Fissle	Tendency to break along laminations.
Parting	Tendency to break parallel to bedding, any scale.
Foliation	Non-depositional, e.g., segregation and layering of minerals in metamorphic rock.

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Tables adapted from the 1987 Soil and Rock Classification Manual, Oregon Department of Transportation.

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FIGURE 6

Test Pit TP-01

CLIENT Olsen Design & Development

PROJECT NAME Fairview Subdivision

PROJECT NUMBER G1404007

PROJECT LOCATION Pringle Creek Rd & Battle Creek Rd - Salem, OR

DATE STARTED 4/29/14

ELEVATION DATUM Provided Topographic Plan (see figure 2)

EXCAVATION CONTRACTOR CGT

GROUND ELEVATION 360 ft

EXCAVATION METHOD Test Pit & Infiltration Test

GROUND WATER LEVELS:

LOGGED BY MDI **CHECKED BY** BMW

SEEPAGE ---

NOTES Bobcat E32 mini-excavator equipped with a 24-inch 'dig' bucket.

AFTER EXCAVATION ---

ELEVATION (ft)	GRAPHIC LOG	U.S.C.S.	MATERIAL DESCRIPTION	GROUNDWATER	DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	▲ SPT N VALUE ▲ PL MC LL ☐ FINES CONTENT (%) ☐ 0 20 40 60 80 100
		OL	**CLAY TOPSOIL:** Medium stiff, brown, moist, exhibited low plasticity, and contained rootlets.		0.0				1.00		
		CL-CH	**LEAN TO FAT CLAY:** Stiff to very stiff, brown, moist, and exhibited medium plasticity. {Residual Soils}		2.5				1.50 2.50 3.00 3.50		
		RX	**DECOMPOSED BASALT:** Very soft (R1), brown with black specks, and moist. Material excavated into 6 inches maximum dimension. {Basalt}			GRAB TP1-1			4.00 4.00 4.00		33
355			Infiltration test (IT-1) performed in test pit at about 5 feet bgs. See text for test results. Orange-brown and black below about 5½ feet bgs.		5.0						
					7.5	GRAB TP1-2					47
350					10.0						

- Test pit terminated at about 10 feet bgs.
- No groundwater or caving observed within depth explored.
- Test pit loosely backfilled with cuttings upon completion.

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FIGURE 7

Test Pit TP-02

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CLIENT Olsen Design & Development
PROJECT NAME Fairview Subdivision
PROJECT NUMBER G1404007
PROJECT LOCATION Pringle Creek Rd & Battle Creek Rd - Salem, OR
DATE STARTED 4/29/14
ELEVATION DATUM Provided Topographic Plan (see figure 2)
EXCAVATION CONTRACTOR CGT
GROUND ELEVATION 366 ft
EXCAVATION METHOD Test Pit & Infiltration Test
GROUND WATER LEVELS:
LOGGED BY MDI **CHECKED BY** BMW
SEEPAGE ---
NOTES Bobcat E32 mini-excavator equipped with a 24-inch 'dig' bucket.
AFTER EXCAVATION ---

ELEVATION (ft)	GRAPHIC LOG	U.S.C.S.	MATERIAL DESCRIPTION	GROUNDWATER	DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	▲ SPT N VALUE ▲ PL MC LL ☐ FINES CONTENT (%) ☐ 0 20 40 60 80 100
365	▨	CL-CH	**LEAN TO FAT CLAY:** Medium stiff to stiff, brown, moist, exhibited medium plasticity, with pieces of weathered rock (up to about 6 inches maximum dimension), isolated small boulders (up to about 18 inches maximum dimension) and roots up to ¼ inch diameter to about 1 foot bgs. {Residual Soils}		0.0				.50 .75 1.50 1.50		
	▨		Stiff to very stiff, red, with no rock or boulders below about 2½ feet bgs.		2.5				1.75 2.50 3.00 3.00		
	▨		Infiltration test (IT-2) performed in test pit at about 5 feet bgs. See text for results.		5.0	GRAB TP2-1					● 68
360	▨		Red and tan below about 6 feet bgs.								
	▨		With pieces of gray, vesicular, decomposed basalt below about 8 feet bgs.		7.5	GRAB TP2-2					● 73
					10.0						

355

- Test pit terminated at about 10 feet bgs.
- No groundwater or caving observed within depth explored.
- Test pit loosely backfilled with cuttings upon completion.

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FIGURE 8

Test Pit TP-03

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CLIENT Olsen Design & Development
PROJECT NAME Fairview Subdivision
PROJECT NUMBER G1404007
PROJECT LOCATION Pringle Creek Rd & Battle Creek Rd - Salem, OR
DATE STARTED 4/29/14
ELEVATION DATUM Provided Topographic Plan (see figure 2)
EXCAVATION CONTRACTOR CGT
GROUND ELEVATION 353 ft
EXCAVATION METHOD Test Pit & Infiltration Test
GROUND WATER LEVELS:
LOGGED BY MDI **CHECKED BY** BMW
SEEPAGE ---
NOTES Bobcat E32 mini-excavator equipped with a 24-inch 'dig' bucket.
AFTER EXCAVATION ---

ELEVATION (ft)	GRAPHIC LOG	U.S.C.S.	MATERIAL DESCRIPTION	GROUNDWATER	DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	▲ SPT N VALUE ▲ PL ● MC LL ☐ FINES CONTENT (%) ☐ 0 20 40 60 80 100
		GW-FILL	**GRAVEL FILL:** Medium dense, gray, moist, angular and relatively well graded. Resembled ¾-inch minus crushed rock. {Man-Made Fill}		0.0						
		CL-CH	**LEAN TO FAT CLAY:** Stiff, brown, moist, medium plasticity, with subangular cobbles and isolated boulders (up to about 2 feet maximum dimension). Roots up to about 3 inches in diameter to about 3 feet bgs. {Residual Soils}		2.5						
350			Hard with black specks and pieces of very decomposed basalt below about 3 feet bgs. Infiltration test (IT-3) performed in test pit at about 3½ feet bgs. See text for results.								PL 27 – LL 62
		RX	**DECOMPOSED BASALT:** Very soft (R1), dark orange-brown with black specks, moist, vesicular. to about 10% volume. {Basalt}			GRAB TP3-1					
					5.0						MC 34
345					7.5	GRAB TP3-2					

- Practical refusal met on decomposed basalt at about 8 feet bgs.
- No groundwater or caving observed within depth explored.
- Test pit loosely backfilled with cuttings upon completion.

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FIGURE 9

Test Pit TP-04

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PROJECT NAME Fairview Subdivision

PROJECT NUMBER G1404007

PROJECT LOCATION Pringle Creek Rd & Battle Creek Rd - Salem, OR

DATE STARTED 4/29/14

ELEVATION DATUM Provided Topographic Plan (see figure 2)

EXCAVATION CONTRACTOR CGT

GROUND ELEVATION 347 ft

EXCAVATION METHOD Test Pit & Infiltration Test

GROUND WATER LEVELS:

LOGGED BY MDI **CHECKED BY** BMW

SEEPAGE ---

NOTES Bobcat E32 mini-excavator equipped with a 24-inch 'dig' bucket.

AFTER EXCAVATION ---

ELEVATION (ft)	GRAPHIC LOG	U.S.C.S.	MATERIAL DESCRIPTION	GROUNDWATER	DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	▲ SPT N VALUE ▲ PL ● MC LL ☐ FINES CONTENT (%) ☐ 0 20 40 60 80 100
		OL	**CLAY TOPSOIL:** Medium stiff, brown, moist, exhibited low plasticity, and contained rootlets.		0.0						
		CL-CH	**LEAN TO FAT CLAY:** Stiff, brown, moist, and exhibited medium plasticity. {Residual Soils}						1.00		
									1.50		
									1.50		
345									2.25		
					2.5				2.25		
									3.00		
									3.25		
		RX	**DECOMPOSED BASALT:** Very soft (R1), dark brown with black rind, moist, breaks into pieces up to 4 inches maximum dimension. {Basalt}						3.25		
			Infiltration test (IT-4) performed in test pit at about 5 feet bgs. See text for results.		5.0	GRAB TP4-1					29
340					7.5	GRAB TP4-2					45

- Practical refusal of excavator met on decomposed basalt at about 8 feet bgs.
- No groundwater or caving observed within depth explored.
- Test pit loosely backfilled with cuttings upon completion.

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FIGURE 10

Test Pit TP-05

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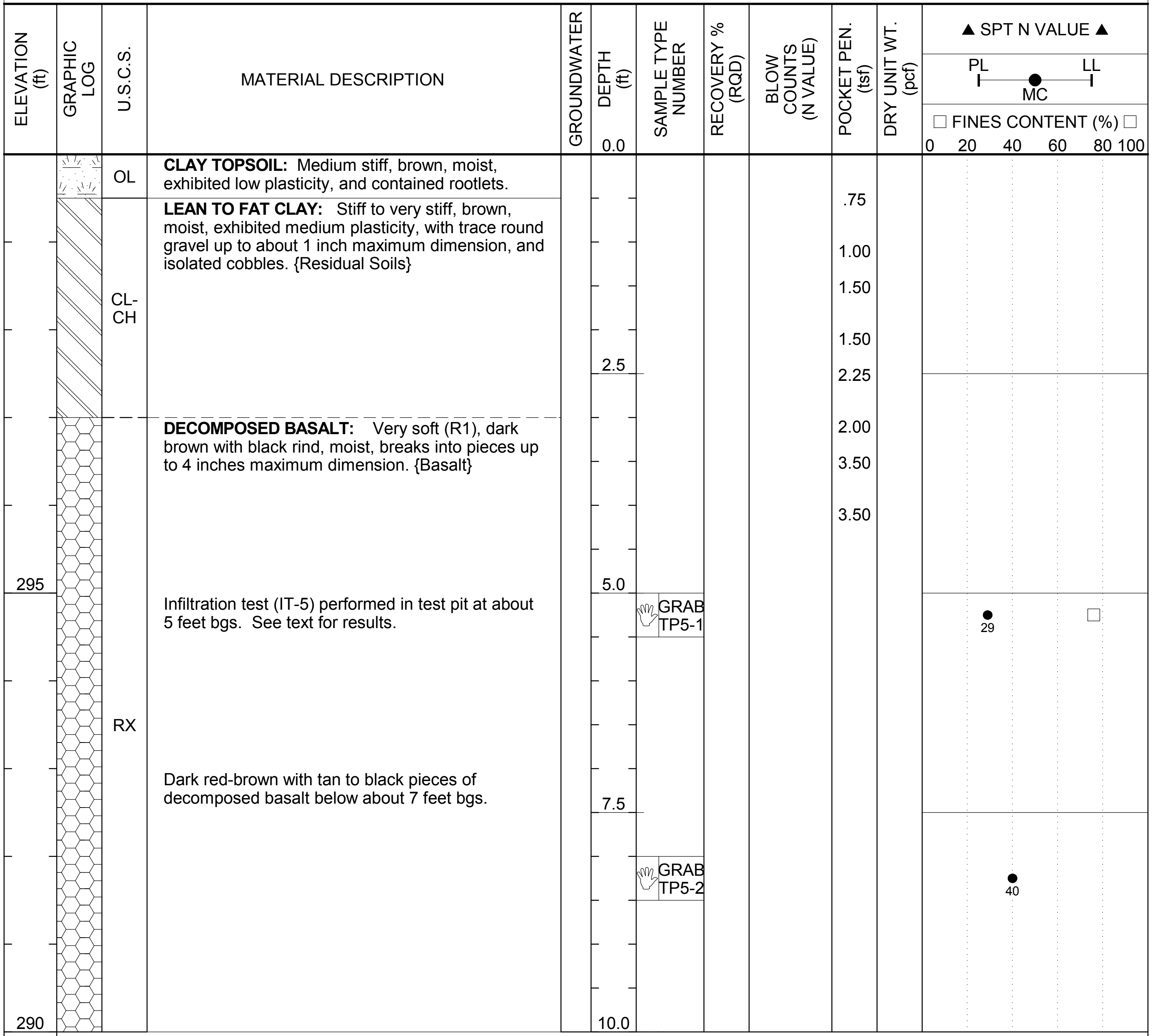
CLIENT Olsen Design & Development
PROJECT NAME Fairview Subdivision
PROJECT NUMBER G1404007
PROJECT LOCATION Pringle Creek Rd & Battle Creek Rd - Salem, OR
DATE STARTED 4/29/14
ELEVATION DATUM Provided Topographic Plan (see figure 2)
EXCAVATION CONTRACTOR CGT
GROUND ELEVATION 300 ft
EXCAVATION METHOD Test Pit & Infiltration Test
GROUND WATER LEVELS:
LOGGED BY MDI **CHECKED BY** BMW
SEEPAGE ---
NOTES Bobcat E32 mini-excavator equipped with a 24-inch 'dig' bucket.
AFTER EXCAVATION ---



ELEVATION (ft)	U.S.C.S.	MATERIAL DESCRIPTION	DEPTH (ft)	SAMPLE TYPE NUMBER	POCKET PEN. (tsf)	MC
	OL	**CLAY TOPSOIL:** Medium stiff, brown, moist, exhibited low plasticity, and contained rootlets.	0.0		.75	
	CL-CH	**LEAN TO FAT CLAY:** Stiff to very stiff, brown, moist, exhibited medium plasticity, with trace round gravel up to about 1 inch maximum dimension, and isolated cobbles. {Residual Soils}	2.5		1.00 1.50 1.50 2.25	
	RX	**DECOMPOSED BASALT:** Very soft (R1), dark brown with black rind, moist, breaks into pieces up to 4 inches maximum dimension. {Basalt}			2.00 3.50 3.50	
295		Infiltration test (IT-5) performed in test pit at about 5 feet bgs. See text for results.	5.0	GRAB TP5-1		29
		Dark red-brown with tan to black pieces of decomposed basalt below about 7 feet bgs.	7.5	GRAB TP5-2		40
290			10.0			

- Test pit terminated at about 10 feet bgs.
- No groundwater or caving observed within depth explored.
- Test pit loosely backfilled with cuttings upon completion.

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FIGURE 11

Test Pit TP-06

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CLIENT Olsen Design & Development
PROJECT NAME Fairview Subdivision
PROJECT NUMBER G1404007
PROJECT LOCATION Pringle Creek Rd & Battle Creek Rd - Salem, OR
DATE STARTED 4/29/14
ELEVATION DATUM Provided Topographic Plan (see figure 2)
EXCAVATION CONTRACTOR CGT
GROUND ELEVATION 290 ft
EXCAVATION METHOD Test Pit & Infiltration Test
GROUND WATER LEVELS:
LOGGED BY MDI **CHECKED BY** BMW
SEEPAGE ---
NOTES Bobcat E32 mini-excavator equipped with a 24-inch 'dig' bucket.
AFTER EXCAVATION ---

ELEVATION (ft)	GRAPHIC LOG	U.S.C.S.	MATERIAL DESCRIPTION	GROUNDWATER	DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	▲ SPT N VALUE ▲ PL MC LL ☐ FINES CONTENT (%) ☐ 0 20 40 60 80 100
		OL	**CLAY TOPSOIL:** Stiff, brown, moist, exhibited low plasticity, and contained rootlets.		0.0						
		CL-CH	**LEAN TO FAT CLAY:** Stiff to very stiff, brown, moist, and exhibited medium plasticity. {Residual Soils}						1.00 1.50 1.50 2.00		
					2.5				2.00		
			With black specks below about 3 feet bgs.						2.75 3.00 3.50		
285			Infiltration test (IT-6) performed in test pit at about 5 feet bgs. See text for results.		5.0	GRAB TP6-1					32
		RX	**DECOMPOSED BASALT:** Very soft (R1), dark brown with black rind, moist, breaks into pieces up to 4 inches maximum dimension. {Basalt}								
					7.5	GRAB TP6-2					30
280					10.0						

- Test pit terminated at about 10 feet bgs.
- No groundwater or caving observed within depth explored.
- Test pit loosely backfilled with cuttings upon completion.

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FIGURE 12

Test Pit TP-07

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PROJECT NAME Fairview Subdivision
PROJECT NUMBER G1404007
PROJECT LOCATION Pringle Creek Rd & Battle Creek Rd - Salem, OR
DATE STARTED 4/29/14
ELEVATION DATUM Provided Topographic Plan (see figure 2)
EXCAVATION CONTRACTOR CGT
GROUND ELEVATION 270 ft
EXCAVATION METHOD Test Pit
GROUND WATER LEVELS:
LOGGED BY MDI **CHECKED BY** BMW
▽ **SEEPAGE** 4.0 ft / Elev 266.0 ft
NOTES Bobcat E32 mini-excavator equipped with a 24-inch 'dig' bucket.
▼ **2hrs AFTER EXCAVATION** 3.0 ft / Elev 267.0 ft

ELEVATION (ft)	GRAPHIC LOG	U.S.C.S.	MATERIAL DESCRIPTION	GROUNDWATER	DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	▲ SPT N VALUE ▲ PL MC LL ☐ FINES CONTENT (%) ☐ 0 20 40 60 80 100
		OL	**CLAY TOPSOIL:** Stiff, brown, moist to very moist, exhibited low plasticity, and contained rootlets.		0.0				.50		
		CL-CH	**LEAN TO FAT CLAY:** Stiff to very stiff, brown, moist, and exhibited medium plasticity. {Alluvium}						.50 1.00		
					2.5	GRAB TP7-1			1.00 1.25		PL 24 LL 41
				▼					1.50		
			Wet below about 4 feet bgs.	▽							
265					5.0						
		GP-GC	**GRAVEL with clay:** Loose to medium dense, brown and gray, wet, round, and up to about 3 inches in diameter. {Alluvium}								
					7.5						
						GRAB TP7-2					MC 47
260					10.0						

- Test pit terminated at about 10 feet bgs.
- Groundwater observed at about 4 feet bgs during excavation of test pit.
- No caving observed within depth explored.
- Test pit loosely backfilled with cuttings upon completion.

CGT BOREHOLE - GRAPHIC LAB G1404007.GPJ GINT US.GDT 5/23/14

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FIGURE 13

Test Pit TP-08

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PROJECT NAME Fairview Subdivision
PROJECT NUMBER G1404007
PROJECT LOCATION Pringle Creek Rd & Battle Creek Rd - Salem, OR
DATE STARTED 4/29/14
ELEVATION DATUM Provided Topographic Plan (see figure 2)
EXCAVATION CONTRACTOR CGT
GROUND ELEVATION 260 ft
EXCAVATION METHOD Test Pit
GROUND WATER LEVELS:
LOGGED BY MDI **CHECKED BY** BMW
▽ **SEEPAGE** 5.0 ft / Elev 255.0 ft
NOTES Bobcat E32 mini-excavator equipped with a 24-inch 'dig' bucket.
▼ **2hrs AFTER EXCAVATION** 3.0 ft / Elev 257.0 ft

ELEVATION (ft)	GRAPHIC LOG	U.S.C.S.	MATERIAL DESCRIPTION	GROUNDWATER	DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	▲ SPT N VALUE ▲ PL MC LL ☐ FINES CONTENT (%) ☐ 0 20 40 60 80 100
		OL	**CLAY TOPSOIL:** Stiff, brown, moist to very moist, exhibited low plasticity, and contained rootlets.		0.0				.50		
		CL-CH	**LEAN TO FAT CLAY:** Stiff to very stiff, brown, moist, and exhibited medium plasticity. {Alluvium}						.75 1.00 1.00		
					2.5				1.50		
				▼		GRAB TP8-1			1.50 2.00 2.00		● 31 ☐
255			Wet below about 5 feet bgs.	▽	5.0						
		GP-GC	**GRAVEL with clay:** Loose to medium dense, brown and gray, wet, round, and up to about 3 inches in diameter. {Alluvium}								
					7.5	GRAB TP8-2					● 40
250					10.0						

- Test pit terminated at about 10 feet bgs.
- Groundwater observed at about 5 feet bgs during excavation of test pit.
- No caving observed within depth explored.
- Test pit loosely backfilled with cuttings upon completion.

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FIGURE 14

Test Pit TP-09

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PROJECT NAME Fairview Subdivision

PROJECT NUMBER G1404007

PROJECT LOCATION Pringle Creek Rd & Battle Creek Rd - Salem, OR

DATE STARTED 4/29/14

ELEVATION DATUM Provided Topographic Plan (see figure 2)

EXCAVATION CONTRACTOR CGT

GROUND ELEVATION 272 ft

EXCAVATION METHOD Test Pit & Infiltration Test

GROUND WATER LEVELS:

LOGGED BY MDI **CHECKED BY** BMW

SEEPAGE ---

NOTES Bobcat E32 mini-excavator equipped with a 24-inch 'dig' bucket.

.25hrs AFTER EXCAVATION ---

ELEVATION (ft)	GRAPHIC LOG	U.S.C.S.	MATERIAL DESCRIPTION	GROUNDWATER	DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	▲ SPT N VALUE ▲ PL MC LL ☐ FINES CONTENT (%) ☐ 0 20 40 60 80 100
270	▨	CL FILL	**LEAN CLAY FILL:** Stiff, dark brown, moist, exhibited medium plasticity, with some angular gravel (up to about 3 inches maximum dimension). Rootlets to about ½ foot bgs. {Man-Made Fill}		0.0 2.5	GRAB TP9-1			2.00 2.50 2.50 2.00 2.25 2.50 2.25		● 26
265 260	▧	CL-CH	**LEAN TO FAT CLAY:** Very stiff, brown with some iron staining, moist, and exhibited medium plasticity. {Residual Soils} Infiltration test (IT-7) performed in test pit at about 5 feet bgs. See text for results. Brown and wet below about 8½ feet bgs.	▼	5.0 7.5 10.0	GRAB TP9-2			2.75		● 32

- Test pit terminated at about 10 feet bgs.
- Groundwater observed within test pit at about 9½ feet bgs.
- No caving observed within depth explored.
- Test pit loosely backfilled with cuttings upon completion.

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FIGURE 15

Test Pit TP-10

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PROJECT NAME Fairview Subdivision

PROJECT NUMBER G1404007

PROJECT LOCATION Pringle Creek Rd & Battle Creek Rd - Salem, OR

DATE STARTED 4/29/14

ELEVATION DATUM Provided Topographic Plan (see figure 2)

EXCAVATION CONTRACTOR CGT

GROUND ELEVATION 270 ft

EXCAVATION METHOD Test Pit & Infiltration Test

GROUND WATER LEVELS:

LOGGED BY MDI

CHECKED BY BMW

SEEPAGE ---

NOTES Bobcat E32 mini-excavator equipped with a 24-inch 'dig' bucket.

AFTER EXCAVATION ---

ELEVATION (ft)	GRAPHIC LOG	U.S.C.S.	MATERIAL DESCRIPTION	GROUNDWATER	DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	▲ SPT N VALUE ▲ PL MC LL ☐ FINES CONTENT (%) ☐ 0 20 40 60 80 100
		OL	**CLAY TOPSOIL:** Stiff, brown, moist, exhibited low plasticity, and contained rootlets.		0.0						
			LEAN TO FAT CLAY: Very stiff, brown, moist to very moist, and exhibited medium plasticity. {Residual Soils}						.75 1.50 1.50 2.00		
					2.5				2.00 2.50 2.50 3.50		
						GRAB TP10-1					29
265		CL-CH	Infiltration test (IT-8) performed in test pit at about 5 feet bgs. See text for results.		5.0						
			Within eastern half of the test pit, lean clay was gray-brown, very moist, and soft below about 6 feet bgs.								
					7.5						
260					10.0	GRAB TP10-2					25

- Test pit terminated at about 10 feet bgs.
- No groundwater or caving observed within depth explored.
- Test pit loosely backfilled with cuttings upon completion.

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FIGURE 16

Test Pit TP-11

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PROJECT NAME Fairview Subdivision
PROJECT NUMBER G1404007
PROJECT LOCATION Pringle Creek Rd & Battle Creek Rd - Salem, OR
DATE STARTED 4/29/14
ELEVATION DATUM Provided Topographic Plan (see figure 2)
EXCAVATION CONTRACTOR CGT
GROUND ELEVATION 266 ft
EXCAVATION METHOD Test Pit & Infiltration Test
GROUND WATER LEVELS:
LOGGED BY MDI **CHECKED BY** BMW
SEEPAGE ---
NOTES Bobcat E32 mini-excavator equipped with a 24-inch 'dig' bucket.
AFTER EXCAVATION ---

ELEVATION (ft)	GRAPHIC LOG	U.S.C.S.	MATERIAL DESCRIPTION	GROUNDWATER	DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	▲ SPT N VALUE ▲ PL ●MC LL ☐ FINES CONTENT (%) ☐ 0 20 40 60 80 100
		OL	**CLAY TOPSOIL:** Stiff, brown, moist, exhibited low plasticity, and contained rootlets.		0.0						
265		CL-CH	**LEAN TO FAT CLAY:** Stiff to very stiff, brown, moist, and exhibited medium plasticity. {Residual Soils}						1.00 1.50 1.50 2.00		
					2.5	GRAB TP11-1			2.50 2.50 3.50 3.50		29 ☐
260			Soft and with trace black specks, below about 5 feet bgs. Infiltration test (IT-9) performed in test pit at about 5 feet bgs. See text for results.		5.0						
					7.5						
						GRAB TP11-2					33
					10.0						
255			• Test pit terminated at about 10 feet bgs. • No groundwater or caving observed within depth explored. • Test pit loosely backfilled with cuttings upon completion.								

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FIGURE 17

Test Pit TP-12

CLIENT Olsen Design & Development
PROJECT NAME Fairview Subdivision
PROJECT NUMBER G1404007
PROJECT LOCATION Pringle Creek Rd & Battle Creek Rd - Salem, OR
DATE STARTED 4/29/14
ELEVATION DATUM Provided Topographic Plan (see figure 2)
EXCAVATION CONTRACTOR CGT
GROUND ELEVATION 250 ft
EXCAVATION METHOD Test Pit & Infiltration Test
GROUND WATER LEVELS:
LOGGED BY MDI **CHECKED BY** BMW
SEEPAGE ---
NOTES Bobcat E32 mini-excavator equipped with a 24-inch 'dig' bucket.
AFTER EXCAVATION ---

ELEVATION (ft)	GRAPHIC LOG	U.S.C.S.	MATERIAL DESCRIPTION	GROUNDWATER	DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	▲ SPT N VALUE ▲ / PL ● MC LL / ☐ FINES CONTENT (%) ☐ 0 20 40 60 80 100
			LEAN TO FAT CLAY: Medium stiff to stiff, brown, moist, exhibited medium plasticity, with roots up to about 3 inches in diameter to about 1 foot bgs. {Residual Soil}		0.0				.50 .75 1.50 1.50		
			Very stiff below about 2½ feet bgs.		2.5				2.25 2.50 3.00		
			Gray-brown below about 4 feet bgs.						3.00		
245		CL-CH	Infiltration test (IT-10) performed in test pit at about 5 feet bgs. See text for results.		5.0	GRAB TP12-1					23 48
					7.5	GRAB TP12-2					● 31
			Hard below about 8 feet bgs.								
240					10.0						

- Test pit terminated at about 10 feet bgs.
- No groundwater or caving observed within depth explored.
- Test pit loosely backfilled with cuttings upon completion.

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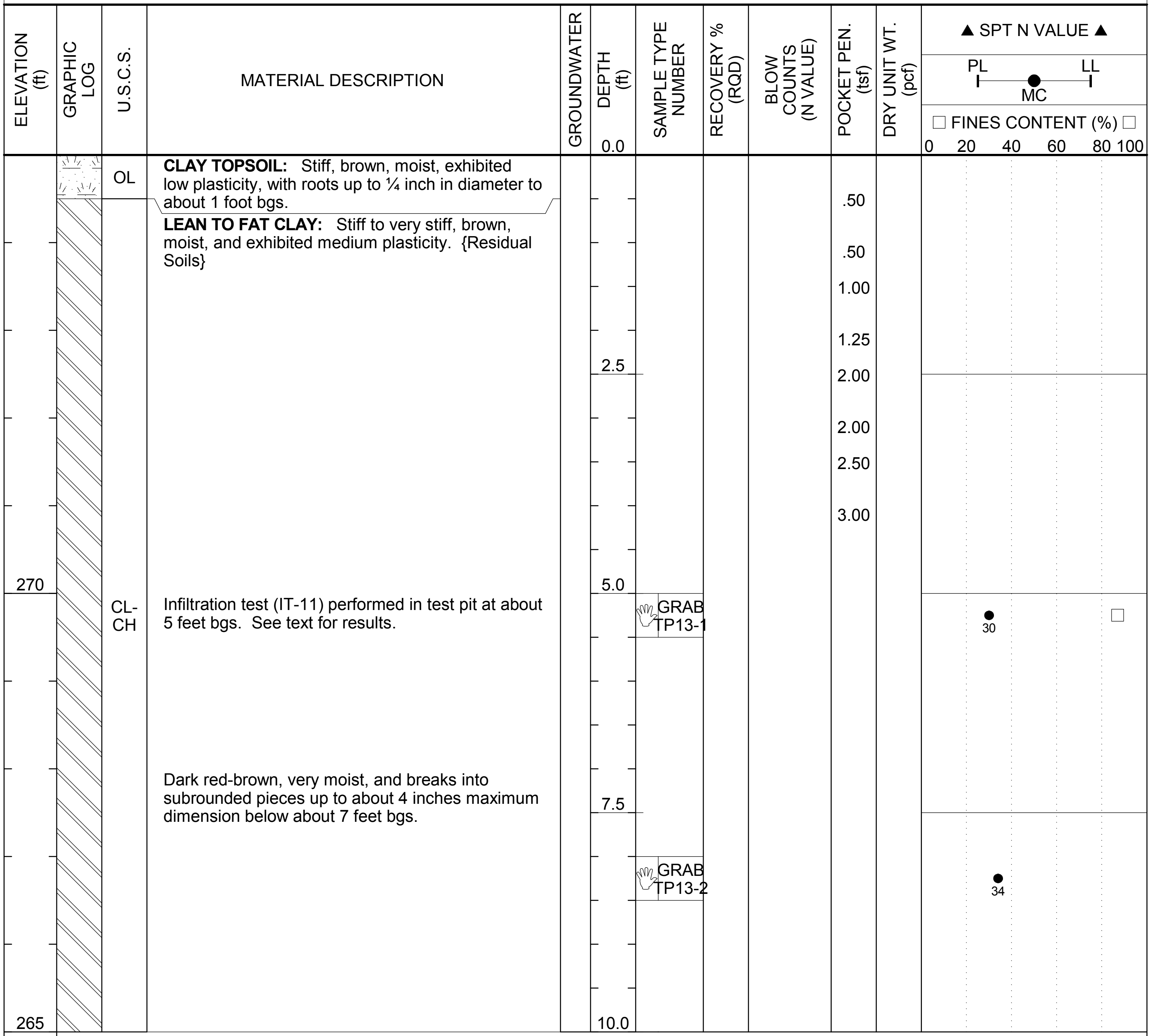
FIGURE 18

Test Pit TP-13

PAGE 1 OF 1

CLIENT Olsen Design & Development
PROJECT NAME Fairview Subdivision
PROJECT NUMBER G1404007
PROJECT LOCATION Pringle Creek Rd & Battle Creek Rd - Salem, OR
DATE STARTED 4/29/14
ELEVATION DATUM Provided Topographic Plan (see figure 2)
EXCAVATION CONTRACTOR CGT
GROUND ELEVATION 275 ft
EXCAVATION METHOD Test Pit & Infiltration Test
GROUND WATER LEVELS:
LOGGED BY MDI **CHECKED BY** BMW
SEEPAGE ---
NOTES Bobcat E32 mini-excavator equipped with a 24-inch 'dig' bucket.
AFTER EXCAVATION ---



CLAY TOPSOIL: Stiff, brown, moist, exhibited low plasticity, with roots up to ¼ inch in diameter to about 1 foot bgs.

LEAN TO FAT CLAY: Stiff to very stiff, brown, moist, and exhibited medium plasticity. {Residual Soils}

Infiltration test (IT-11) performed in test pit at about 5 feet bgs. See text for results.

Dark red-brown, very moist, and breaks into subrounded pieces up to about 4 inches maximum dimension below about 7 feet bgs.

- Test pit terminated at about 10 feet bgs.
- No groundwater or caving observed within depth explored.
- Test pit loosely backfilled with cuttings upon completion.

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FIGURE 19

Test Pit TP-14

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CLIENT Olsen Design & Development
PROJECT NAME Fairview Subdivision
PROJECT NUMBER G1404007
PROJECT LOCATION Pringle Creek Rd & Battle Creek Rd - Salem, OR
DATE STARTED 4/29/14
ELEVATION DATUM Provided Topographic Plan (see figure 2)
EXCAVATION CONTRACTOR CGT
GROUND ELEVATION 296 ft
EXCAVATION METHOD Test Pit & Infiltration Test
GROUND WATER LEVELS:
LOGGED BY MDI **CHECKED BY** BMW
SEEPAGE ---
NOTES Bobcat E32 mini-excavator equipped with a 24-inch 'dig' bucket.
AFTER EXCAVATION ---

ELEVATION (ft)	GRAPHIC LOG	U.S.C.S.	MATERIAL DESCRIPTION	GROUNDWATER	DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	▲ SPT N VALUE ▲ PL ● MC LL ☐ FINES CONTENT (%) ☐ 0 20 40 60 80 100
		OL	**CLAY TOPSOIL:** Medium stiff, brown, moist, exhibited low plasticity, and contained rootlets.		0.0				.75		
295			**LEAN TO FAT CLAY:** Stiff to very stiff, brown, moist, and exhibited medium plasticity. {Residual Soils}						1.00		
									1.00		
									1.50		
					2.5				1.50		
									2.00		
									3.00		
			With trace black specks and breaks into subangular pieces below about 4 feet bgs.			GRAB TP14-1			4.00		PL 25, LL 59
			Infiltration test (IT-12) performed in test pit at about 4 feet bgs. See text for results.		5.0						
		CL-CH									
290			Isolated boulders (up to about 18 inches maximum dimension) at about 6 feet bgs.								
					7.5	GRAB TP14-2					MC 32
			With pieces of decomposed basalt below about 8 feet bgs.								
					10.0						

285

- Test pit terminated at about 10 feet bgs.
- No groundwater or caving observed within depth explored.
- Test pit loosely backfilled with cuttings upon completion.

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FIGURE 20

Test Pit TP-15

PAGE 1 OF 1

CLIENT Olsen Design & Development
PROJECT NAME Fairview Subdivision
PROJECT NUMBER G1404007
PROJECT LOCATION Pringle Creek Rd & Battle Creek Rd - Salem, OR
DATE STARTED 4/29/14
ELEVATION DATUM Provided Topographic Plan (see figure 2)
EXCAVATION CONTRACTOR CGT
GROUND ELEVATION 257 ft
EXCAVATION METHOD Test Pit & Infiltration Test
GROUND WATER LEVELS:
LOGGED BY MDI **CHECKED BY** BMW
SEEPAGE ---
NOTES Bobcat E32 mini-excavator equipped with a 24-inch 'dig' bucket.
AFTER EXCAVATION ---

ELEVATION (ft)	GRAPHIC LOG	U.S.C.S.	MATERIAL DESCRIPTION	GROUNDWATER	DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	▲ SPT N VALUE ▲ / PL ● MC LL / ☐ FINES CONTENT (%) ☐ (0 20 40 60 80 100)
		OL	**CLAY TOPSOIL:** Medium stiff, brown, moist, exhibited low plasticity, and contained rootlets.		0.0				.75		
255		CL-CH	**LEAN TO FAT CLAY:** Stiff to very stiff, orange-brown, moist, and exhibited medium plasticity. {Residual Soils}		2.5				1.50 1.75 1.75 2.25 2.50 3.00		
250		RX	**DECOMPOSED BASALT:** Very soft (R1), dark brown with black rind, moist, breaks into pieces up to 4 inches maximum dimension. {Basalt} Infiltration test (IT-13) performed in test pit at about 4 feet bgs. See text for results. Red-brown with pieces of decomposed basalt below about 6 feet bgs.		5.0 7.5 10.0	GRAB TP15-1 GRAB TP15-2			4.00		● 30 ● 34
245											

- Test pit terminated at about 10 feet bgs.
- No groundwater or caving observed within depth explored.
- Test pit loosely backfilled with cuttings upon completion.

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FIGURE 21

Test Pit TP-16

PAGE 1 OF 1

CLIENT Olsen Design & Development

PROJECT NAME Fairview Subdivision

PROJECT NUMBER G1404007

PROJECT LOCATION Pringle Creek Rd & Battle Creek Rd - Salem, OR

DATE STARTED 4/29/14

ELEVATION DATUM Provided Topographic Plan (see figure 2)

EXCAVATION CONTRACTOR CGT

GROUND ELEVATION 260 ft

EXCAVATION METHOD Test Pit & Infiltration Test

GROUND WATER LEVELS:

LOGGED BY MDI **CHECKED BY** BMW

SEEPAGE ---

NOTES Bobcat E32 mini-excavator equipped with a 24-inch 'dig' bucket.

AFTER EXCAVATION ---

ELEVATION (ft)	GRAPHIC LOG	U.S.C.S.	MATERIAL DESCRIPTION	GROUNDWATER	DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	▲ SPT N VALUE ▲ PL MC LL ☐ FINES CONTENT (%) ☐ 0 20 40 60 80 100
		OL	**CLAY TOPSOIL:** Medium stiff, brown, moist, exhibited low plasticity, and contained rootlets.		0.0				.50		
255		CL-CH	**LEAN TO FAT CLAY:** Medium stiff, brown, moist, and exhibited medium plasticity. Isolated roots up to ¼ inch in diameter to about 2 feet bgs. {Residual Soils} Stiff to very stiff below about 1½ feet bgs.		2.5 5.0				.50 1.50 1.00 1.50 2.00 2.00 2.50		
		RX	**DECOMPOSED BASALT:** Very soft (R1), dark red brown with black rind, moist, breaks into pieces up to 4 inches maximum dimension. {Basalt} Infiltration test (IT-14) performed in test pit at about 5 feet bgs. See text for results.			GRAB TP16-1					● 33
250					7.5 10.0	GRAB TP16-2					● 34

- Test pit terminated at about 10 feet bgs.
- No groundwater or caving observed within depth explored.
- Test pit loosely backfilled with cuttings upon completion.

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FIGURE 22

Test Pit TP-17

PAGE 1 OF 1

CLIENT Olsen Design & Development
PROJECT NAME Fairview Subdivision
PROJECT NUMBER G1404007
PROJECT LOCATION Pringle Creek Rd & Battle Creek Rd - Salem, OR
DATE STARTED 4/29/14
ELEVATION DATUM Provided Topographic Plan (see figure 2)
EXCAVATION CONTRACTOR CGT
GROUND ELEVATION 304 ft
EXCAVATION METHOD Test Pit & Infiltration Test
GROUND WATER LEVELS:
LOGGED BY MDI **CHECKED BY** BMW
SEEPAGE ---
NOTES Bobcat E32 mini-excavator equipped with a 24-inch 'dig' bucket.
AFTER EXCAVATION ---

ELEVATION (ft)	GRAPHIC LOG	U.S.C.S.	MATERIAL DESCRIPTION	GROUNDWATER	DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	▲ SPT N VALUE ▲ / PL — MC — LL / ☐ FINES CONTENT (%) ☐ (0 20 40 60 80 100)
	▩	GP-GC FILL	**GRAVEL FILL:** Medium dense, brown, moist, large, round, up to about 2 inches maximum dimension, with some clay. {Man-Made Fill}		0.0						
	▧	CL-CH	**LEAN TO FAT CLAY:** Stiff to very stiff, brown, moist, and exhibited medium plasticity. {Residual Soils}						1.50 1.50 2.25		
					2.5				3.00 3.00		
300			Infiltration test (IT-15) performed in test pit at about 3½ feet bgs. See text for results. With trace black specks below about 4 feet bgs.			GRAB TP17-1			4.00		● 35 ☐
	⬡	RX	**DECOMPOSED BASALT:** Very soft (R1), brown and moist, breaks into pieces up to 6 inches maximum dimension. {Basalt}		5.0						
					7.5	GRAB TP17-2					● 38
295					10.0						

- Test pit terminated at about 10 feet bgs.
- No groundwater or caving observed within depth explored.
- Test pit loosely backfilled with cuttings upon completion.

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FIGURE 23

Test Pit TP-18

PAGE 1 OF 1

CLIENT Olsen Design & Development
PROJECT NAME Fairview Subdivision
PROJECT NUMBER G1404007
PROJECT LOCATION Pringle Creek Rd & Battle Creek Rd - Salem, OR
DATE STARTED 4/29/14
ELEVATION DATUM Provided Topographic Plan (see figure 2)
EXCAVATION CONTRACTOR CGT
GROUND ELEVATION 319 ft
EXCAVATION METHOD Test Pit & Infiltration Test
GROUND WATER LEVELS:
LOGGED BY MDI **CHECKED BY** BMW
SEEPAGE ---
NOTES Bobcat E32 mini-excavator equipped with a 24-inch 'dig' bucket.
AFTER EXCAVATION ---

ELEVATION (ft)	GRAPHIC LOG	U.S.C.S.	MATERIAL DESCRIPTION	GROUNDWATER	DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	▲ SPT N VALUE ▲ PL ● MC LL ☐ FINES CONTENT (%) ☐ 0 20 40 60 80 100
		OL	**CLAY TOPSOIL:** Medium stiff, brown, moist, exhibited low plasticity, and contained rootlets.		0.0						
		CL-CH	**LEAN TO FAT CLAY:** Stiff to very stiff, dark red-brown, moist, and exhibited medium plasticity. {Residual Soils}						.75 1.00 1.00		
			With pieces of decomposed basalt, breaks into subangular pieces up to about 3 inches maximum dimension below about 2 feet bgs.		2.5				1.50 3.00 3.75 3.75		
315			Infiltration test (IT-16) performed in test pit at about 4 feet bgs. See text for results.			GRAB TP18-1			4.00		● 28
					5.0						
					7.5	GRAB TP18-2					
310											● 23
					10.0						

- Test pit terminated at about 10 feet bgs.
- No groundwater or caving observed within depth explored.
- Test pit loosely backfilled with cuttings upon completion.

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FIGURE 24

Test Pit TP-19

PAGE 1 OF 1

CLIENT Olsen Design & Development
PROJECT NAME Fairview Subdivision
PROJECT NUMBER G1404007
PROJECT LOCATION Pringle Creek Rd & Battle Creek Rd - Salem, OR
DATE STARTED 4/29/14
ELEVATION DATUM Provided Topographic Plan (see figure 2)
EXCAVATION CONTRACTOR CGT
GROUND ELEVATION 320 ft
EXCAVATION METHOD Test Pit & Infiltration Test
GROUND WATER LEVELS:
LOGGED BY MDI **CHECKED BY** BMW
SEEPAGE ---
NOTES Bobcat E32 mini-excavator equipped with a 24-inch 'dig' bucket.
AFTER EXCAVATION ---

ELEVATION (ft)	GRAPHIC LOG	U.S.C.S.	MATERIAL DESCRIPTION	GROUNDWATER	DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	▲ SPT N VALUE ▲ PL MC LL ☐ FINES CONTENT (%) ☐ 0 20 40 60 80 100
			LEAN TO FAT CLAY: Medium stiff, red-brown, moist, exhibited low plasticity, with cobbles and isolated small boulders (up to about 18 inches maximum dimension). Roots up to ¼ inch diameter to about 1 foot bgs. {Residual Soils}		0				.50		
					1				.75		
			Stiff to very stiff below about 1½ feet bgs.						1.00		
318		CL-CH			2				2.00		
									3.00		
			With black specks below about 3 feet bgs.		3				4.00		
			Infiltration test (IT-17) performed in test pit at about 3 feet bgs. See text for results.			GRAB TP19-1					● 31
									4.00		
316					4						
			• Practical refusal of excavator on large boulder at about 4 feet bgs. • No groundwater or caving observed within depth explored. • Test pit loosely backfilled with cuttings upon completion.						4.00		
314											

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FIGURE 25

Test Pit TP-20

PAGE 1 OF 1

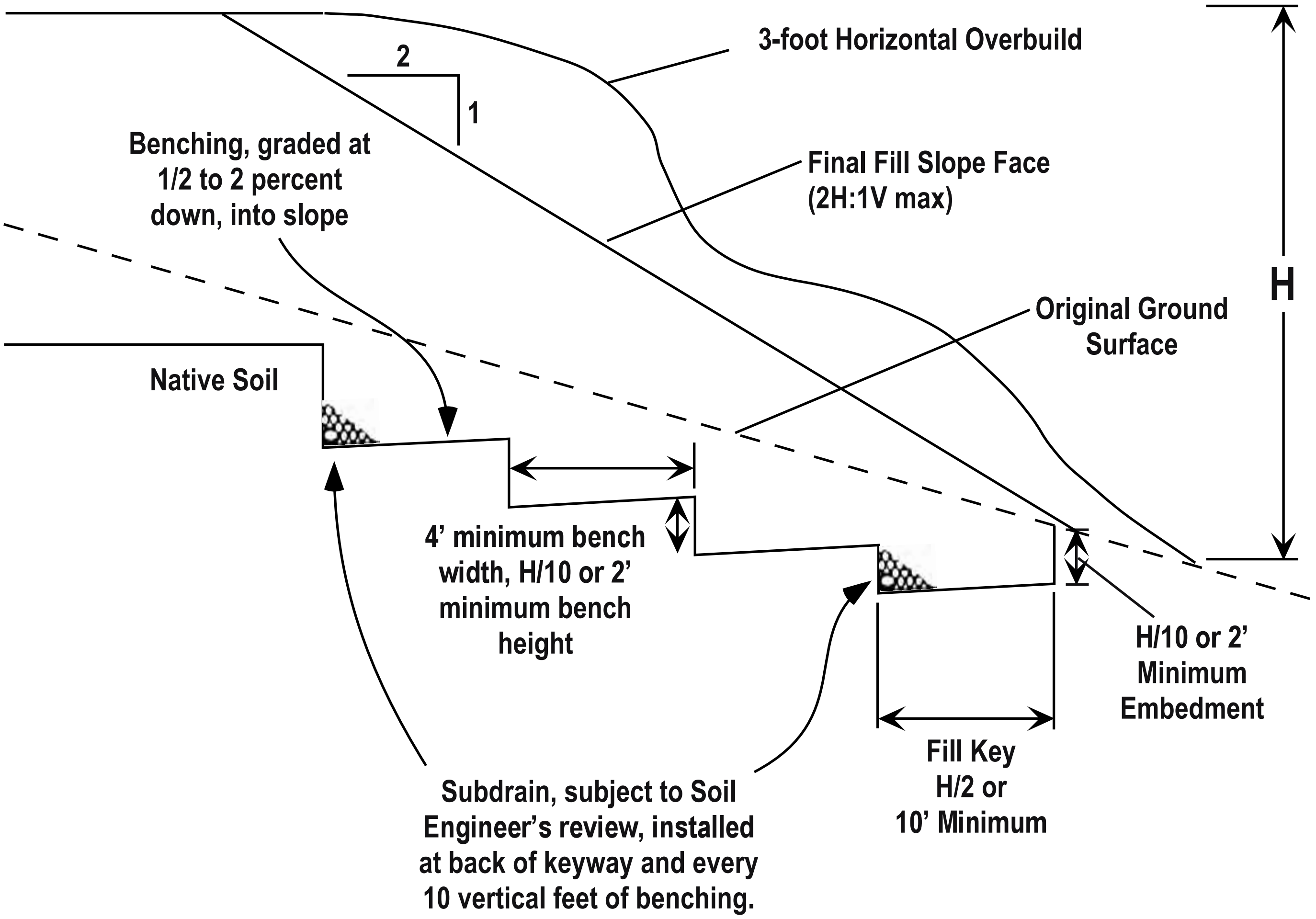
CLIENT Olsen Design & Development
PROJECT NAME Fairview Subdivision
PROJECT NUMBER G1404007
PROJECT LOCATION Pringle Creek Rd & Battle Creek Rd - Salem, OR
DATE STARTED 4/29/14
ELEVATION DATUM Provided Topographic Plan (see figure 2)
EXCAVATION CONTRACTOR CGT
GROUND ELEVATION 302 ft
EXCAVATION METHOD Test Pit & Infiltration Test
GROUND WATER LEVELS:
LOGGED BY MDI **CHECKED BY** BMW
SEEPAGE ---
NOTES Bobcat E32 mini-excavator equipped with a 24-inch 'dig' bucket.
AFTER EXCAVATION ---

ELEVATION (ft)	GRAPHIC LOG	U.S.C.S.	MATERIAL DESCRIPTION	GROUNDWATER	DEPTH (ft)	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	▲ SPT N VALUE ▲ PL ● MC LL ☐ FINES CONTENT (%) ☐ 0 20 40 60 80 100
300	▧	CL-CH	**LEAN TO FAT CLAY:** Stiff to very stiff, brown, very moist, exhibited medium plasticity, with trace large, subrounded gravel (up to about 3 inches maximum dimension), and isolated cobbles. {Residual Soils} Infiltration test (IT-18) performed in test pit at about 2 feet bgs. See text for results.		0.0 2.5	GRAB TP20-1			.50 1.00 1.00 1.50 2.50		PL 24, LL 44
295		RX	**DECOMPOSED BASALT:** Very soft (R1), orange-brown with black rind, moist, breaks into pieces up to 6 inches maximum dimension. {Basalt}		5.0 7.5	GRAB TP20-2			4.00 4.00 4.00		MC 45
290			• Practical refusal of excavator on decomposed basalt at about 8 feet bgs. • No groundwater or caving observed within depth explored. • Test pit loosely backfilled with cuttings upon completion.		10.0						

FAIRVIEW SUBDIVISION - SALEM, OREGON
Project Number G1404007

FIGURE 26

FILL SLOPE DETAIL



NOTE: Surfaces to receive fill with slopes steeper than 5H:1V (horizontal:vertical) should be benched and keyed as shown.

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Appendix A: Engineering Geology Report

Fairview Subdivision
Pringle Road SE & Battle Creek Road SE
Salem, Oregon

CGT Project No. G1404007

May 23, 2014

Prepared For:

Mr. Eric Olsen
Olsen Design & Development
170 West Main Street
Monmouth, Oregon 97361

Prepared By:

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GEOLOGY

A.1.1.1 Regional Geology

The site is located in the central portion of the Willamette Valley physiographic province in Salem, Oregon. The Willamette Valley is a broad trough-like lowland defined by uplift and faulting of the Coast and Western Cascade Ranges to the west and east respectively. Approximately 35 million years ago, a large slab of oceanic crust and associated marine sediments accreted onto the margin of North America, which was located in a rough line from southwestern Oregon to the northeastern portion of the state. A portion of this accreted slab became the Willamette Valley, which was still covered by a shallow ocean. Additional accretion, faulting, and folding created the Coast Range to the west. This folding and faulting also raised the Willamette Valley out of the sea. Volcanic activity from the Cascade Range approximately 25 million years ago covered and filled in much of the southern and eastern portions of the early Willamette Valley[1].

Approximately 15 million years ago, Columbia River Basalts flowed down what is now the Columbia River Gorge as far west as the Oregon and Washington coast, and into the Willamette Valley as far south as Salem, Oregon. Uplift and faulting within the Willamette Basin formed the intra-valley highlands such as the Tualatin and Chehalem Mountains and the Amity, Eola, and Salem Hills. Infilling of the Willamette Valley continued from weathering of the adjacent hills and deposition of alluvium by the Willamette River and its tributaries throughout the valley. Catastrophic glacial floods later flowed into the Willamette Valley approximately 12,000 to 15,000 years ago and deposited fine to coarse-grained sedimentary assemblages (Pleistocene flood deposits) mapped on the eastern edge of the site[2,3,4].

A.1.1.2 Site Geology

According to the geologic map for the site and vicinity[5], excerpted on Figure A1, the site is underlain by Tertiary Columbia River Basalt (Tcr). Unit Tcr is characterized as medium-gray to black, fine-grained basalt with a maximum thickness of about 400 to 600 feet. Weathering of Tcr results in reddish-brown to grayish-brown, crumbly to medium dense basalt, with some exposures altered to red to brown clay (residual soil). The residual soil typically has low to moderate expansive potential.

The geologic map shows a narrow band of Quaternary higher terrace deposits (Qth) in the northeastern part of the site. Unit Qth is characterized as semiconsolidated sand, silt, and clay colluvium, slope wash, and alluvial fan deposits. Thickness of this unit is given as about 3 to 15 feet. On the subject site we anticipate that this material is underlain by Tcr described above.

---

[1] Pacific Northwest Ecosystem Research Consortium, 2002. Willamette River Basin: trajectories of environmental and ecological change, Oregon State University Press.

[2] Bela, James L., 1981, Geology of the Rickreall, Salem West, Monmouth, and Sidney 7½' Quadrangles, Marion, Polk, and Linn Counties, Oregon: Oregon Department of Geology and Mineral Industries Map GMS-18, 2 plates.

[3] Orr, Elizabeth L., Orr, William N., and Baldwin, Ewart M., 1992, Geology of Oregon, Fourth Edition: Kendall/Hunt Publishing, pp. 203-222.

[4] O'Connor, Jim E., et al., 2001, Origin, extent, and thickness of quaternary geologic units in the Willamette Valley, Oregon: US Geological Survey, Professional Paper 1620,52p, 1plate.

[5] Beeson et. al., 2000, Geologic Map of the Salem East Quadrangle, Marion County, Oregon. USGS Open-File Report 00-351.

A.2.0 SEISMICITY

A.2.1 Earthquake Sources

A.2.1.1 Cascadia Subduction Zone

The Cascadia Subduction Zone (CSZ) is a 1,000-kilometer-long zone of active tectonic convergence where the oceanic crust of the Juan De Fuca Plate is subducting beneath the North American continental plate at a rate of about 3 to 4 centimeters per year[6]. The fault trace is located off of the Oregon Coast, approximately 180 kilometers west of the site. Two primary sources of seismicity are associated with the CSZ: the interface between the two plates, and faulting within the subducting plate. These sources are detailed below. The location of the CSZ and associated sources of seismicity are shown on the attached Figure A2.

A.2.1.1.1 Plate Interface Source

Very little seismicity has occurred on the plate interface in historic time, and as a result, the seismic potential of the CSZ is a subject of scientific controversy. The lack of seismicity may be interpreted as a period of quiescent stress buildup between large magnitude earthquakes, or characteristic of the long-term behavior of the subduction zone. A growing body of geologic evidence; however, strongly suggests that large prehistoric subduction zone earthquakes have occurred[7,8,9,10]. This evidence includes: (1) buried tidal marshes recording episodic, sudden subsidence along the coast of northern California, Oregon, and Washington; (2) burial of subsided tidal marshes by tsunami wave deposits; (3) paleoliquefaction features; and (4) geodetic uplift patterns on the Oregon Coast. Radiocarbon dates on buried tidal marshes indicate a recurrence interval for major subduction zone earthquakes of 250 to 650 years, with the last major event occurring 300 years ago[11,12,13,14,15]. The inferred seismogenic portion of the plate interface is roughly 10 to 25 kilometers deep, spanning a 75-kilometer wide area roughly centered on the Oregon coastline. The eastern margin of the plate interface seismogenic zone is approximately 80 kilometers west of the site.

A.2.1.1.2 Intra-Slab Source

The subducting Juan De Fuca (oceanic) Plate dips at an angle of 10 to 20 degrees as it descends beneath the North American plate. The curvature of the subducted plate increases as the advancing edge moves east, creating extensional forces within the plate. Normal faulting occurs in response to these extensional forces. This region of maximum curvature and faulting of the slab is where large intra-slab earthquakes are expected to occur, and is located at depths ranging from 30 to 60 kilometers[16]. The site is located within the inferred

---

[6] DeMets, C., Gordon, R.G., Argus, D.F., Stein, S., 1990. Current plate motions: Geophysical Journal International, v. 101, p. 425-478.

[7] Geomatrix Consultants, 1995. *Ibid*.

[8] Atwater, B.F., 1992. Geologic evidence for earthquakes during the past 2,000 years along the Copalis River, southern coastal Washington: Journal of Geophysical Research, v. 97, p. 1901-1919.

[9] Carver, G., 1992. Late Cenozoic tectonics of coastal northern California: American Association of Petroleum Geologists-SEPM Field Trip Guidebook, May, 1992.

[10] Peterson, C.D., Darioenzo, M.E., Burns, S.F., and Burris, W.K., 1993. Field trip guide to Cascadia paleoseismic evidence along the northern California coast: evidence of subduction zone seismicity in the central Cascadia margin: Oregon Geology, v. 55, p. 99 144.

[11] Geomatrix Consultants, 1995. *Ibid*.

[12] Atwater, B.F., 1992. *Ibid*.

[13] Carver, G., 1992. *Ibid*.

[14] Peterson, C.D., Darioenzo, M.E., Burns, S.F., and Burris, W.K., 1993. *Ibid*.

[15] Personius, S.F., and Nelson, A.R., compilers, 2005. Fault number 781, in Quaternary fault and fold database of the United States: U.S. Geological Survey website, http://earthquakes.usgs.gov/regional/qfaults.

[16] Geomatrix Consultants, 1995. *Ibid*.

intra-slab seismogenic zone[17] (see attached Figure A2). Historically, the seismicity rate within the Juan De Fuca Plate beneath Oregon is very low in northern Oregon and southwest Washington, and extremely low along the southern and central Oregon coast[18,19,20].

A.2.1.2 Crustal Sources

Several nearby faults capable of producing damaging earthquakes in this region include the Salem-Eola Hills homocline, the Mill Creek fault, and the Waldo Hills fault. Distances from the site to known active or potentially active faults are summarized in Table 1.

Table 1: Known Active or Potentially Active Crustal Faults in the Vicinity of the Site

USGS Fault No.	Fault Name	Distance and Direction from Site	Activity Level*
872	Waldo Hills fault	3 km SE	Active
871	Mill Creek fault	8 km S-SE	Active
719	Salem-Eola Hills homocline	8 km W	Active
* Activity Level derived from USGS fault database. "Active" indicates that the fault has been active during the Quaternary (the last 1.6 million years).			

<u>Salem-Eola Hills homocline (USGS 719)</u>

The Salem-Eola Hills homocline is a 31-kilometer-long homoclinal fold roughly coincident with the southwestern edge of the Salem and Eola Hills[21]. The homocline deforms Teritary Columbia River Basalts (Tcr), and marks the southwestern margin of the Tcr in this area. The Salem-Eola Hills homocline is likely the result of very slow uplift of the Salem and Eola Hills. No direct evidence has been found for recent (Holocene) deformation, so the fold is typically considered to have a low probability of activity and a long recurrence interval.

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[17] McCrory, Blair, Oppenheimer, and Walter, 2004. Depth to the Juan de Fuca slab beneath the Cascadia subduction margin – A 3-D model for storing earthquakes: U.S. Geological Survey Data Series 91.

[18] Geomatrix Consultants, 1995. *Ibid*.

[19] Geomatrix Consultants, 1993. Seismic margin Earthquake For the Trojan Site: Final Unpublished Report For Portland General Electric Trojan Nuclear Plant, Rainier, Oregon, May 1993.

[20] Personius, S.F., and Nelson, A.R., compilers, 2005. Fault number 781, in Quaternary fault and fold database of the United States: U.S. Geological Survey website, http://earthquakes.usgs.gov/regional/qfaults.

[21] Personius, S.F., compiler, 2002. Fault number 719, Salem-Eola Hills homocline, in Quaternary fault and fold database of the United States: U.S. Geological Survey website, http://earthquakes.usgs.gov/regional/qfaults.

Mill Creek fault (USGS 871)

The Mill Creek fault consists of an 18-kilometer-long, steeply-dipping reverse fault bounding the southeast margin of the Waldo Hills[22]. The Mill Creek fault is recognized in the subsurface by at least 160 feet of vertical separation of the top of the Columbia River Basalt[23]. The Mill Creek fault does not appear to deform Pleistocene or Holocene deposits; however, this fault may have a long recurrence interval and is considered active[24].

Waldo Hills fault (USGS 872)

The Waldo Hills fault is a 12-kilometer-long southeast-dipping reverse fault that is mapped on the northwestern front of the Waldo Hills[25]. The fault is recognized in the subsurface by vertical separation of the top of the Columbia River Basalt[26]. No evidence for middle or late Quaternary displacement on the Waldo Hills fault has been identified; however, Oregon State University geologists suggest that the Waldo Hills fault may have a long recurrence interval and is considered active[27]. Recurrence interval estimates for earthquake activity on the Waldo Hills fault are considered to be on the order of 700,000 years or more. Extensive erosion and degradation of the identified fault scarps supports a long recurrence interval.

A.2.1.3 Historic Seismicity

The Pacific Northwest is a seismically active area. Table 2 lists earthquakes with magnitudes larger than M4.9 that have occurred within 200 kilometers of the site since 1873[28]. These earthquakes are also included on Plate 1: Historical Earthquakes.

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[22] Personius, S.F., compiler, 2002. Fault number 871, Mill Creek fault, in Quaternary fault and fold database of the United States: U.S. Geological Survey website, http://earthquakes.usgs.gov/regional/qfaults.

[23] Yeats, R.S., *et al*., 1996. *Ibid.*

[24] Geomatrix Consultants, 1995. *Ibid*.

[25] Personius, S.F., compiler, 2002. Fault number 872, Waldo Hills fault, in Quaternary fault and fold database of the United States: U.S. Geological Survey website, http://earthquakes.usgs.gov/regional/qfaults.

[26] Yeats, R.S., *et al*., 1996. Tectonics of the Willamette Valley Oregon: in Assessing earthquake hazards and reducing risk in the Pacific Northwest, v. 1: U.S. Geological Survey Professional Paper 1560, p. 183-222, 5 plates, scale 1:100,000.

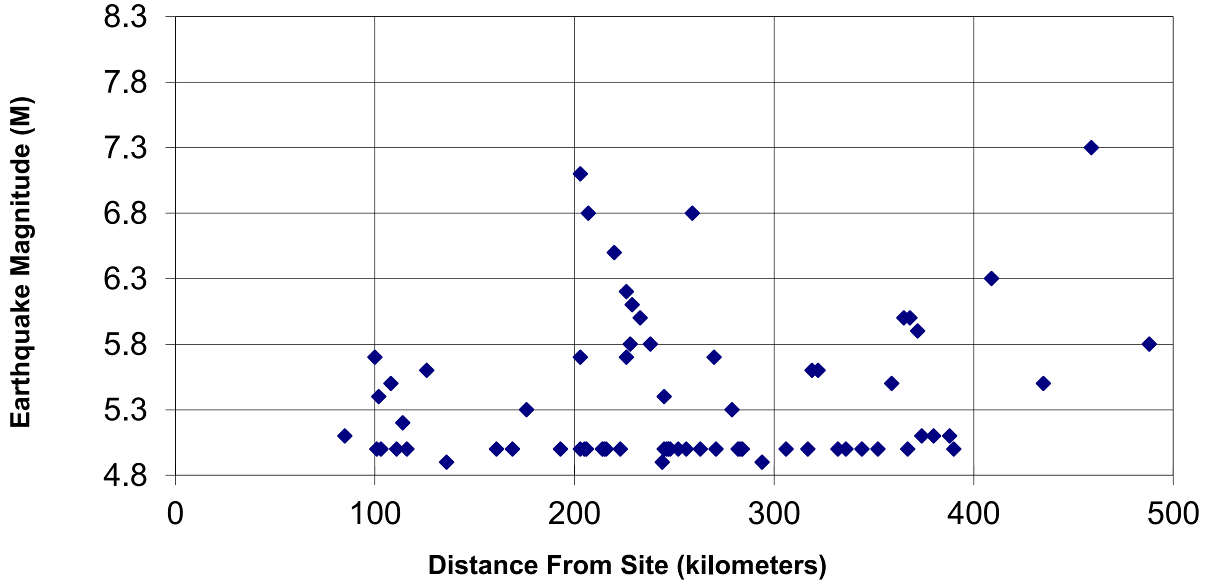
[27] Geomatrix Consultants, 1995. *Ibid*.

[28] Wong et al, 2000. Wong, I. Silva, W. Bott, J., Wright, D., Thomas, P., Gregor, N., Li, S., Mabey, M., Sojourner, A., Wang, Y. IMS-15. Earthquake Scenario and Probabilistic Ground Shaking Maps for the Portland, Oregon, Metropolitan area. Portland Hills Fault M6.8 Earthquake, Peak Horizontal Acceleration at the Ground Surface.

Table 2: Historical Earthquakes since 1873 within 200 kilometers of the site with Magnitudes Greater than M4.9

Date	Magnitude	Distance From Site	Location
July 19, 1930	M5.0	20 km	15 km WNW of Salem, OR
March 25, 1993	M5.6	35 km	23 km ESE of Woodburn, OR (Scotts Mills)
December 16, 1953	M5.0	72 km	7 km WSW of Portland, OR
December 29, 1941	M5.0	77 km	1 km S of Portland, OR
November 17, 1957	M5.0	77 km	18 km S of Tillamook, OR
October 12, 1877	M5.4*	78 km	10 km ESE of Portland, OR
November 06, 1962	M5.5	85 km	8 km NNE of Portland, OR
July 12, 2004	M4.9	133 km	48 km SW of Newport, OR
September 17, 1961	M5.1	143 km	20 km SSE of Mt St Helens, WA
March - May, 1980	M4.9 - M5.2	160 km	27 events at Mt St Helens, WA
May 18, 1980	M5.7	159 km	1 km NNE of Mt St Helens, WA
February 14, 1981	M5.2	172 km	2 km N of Elk Lake, WA
November 08, 1960	M5.0	188 km	115 km WNW of Newport, OR

Plate 1. Historical Earthquakes



A.3.0 LOCAL TOPOGRAPHY

Topography in the vicinity of the site was obtained from the USGS 7.5 minute topographic map for the Salem West quadrangle, and is shown on Figure 1 attached to the geotechnical report. The site is located on the northeast margin of the Salem Hills. The subject property wraps around a localized high, with onsite slopes descending to the west, north, and east. Slope gradients on and in the immediate vicinity of the site are generally 5H:1V (Horizontal:Vertical) or flatter. Site topography is discussed in detail in Section A.5.1 below.

A.4.0 HAZARDS

A.4.1 Slope Stability

Landsliding is a common hazard in the Pacific Northwest that can be initiated on marginally stable slopes by human disturbances such as grading and deforestation, and by natural processes including earthquake shaking, volcanism, heavy rainfalls, and rapid snow melt. Recent studies indicate that the most common causes for slope failures are intense rainfall and human alteration, including the placement of building loads on slopes, excavating or over-steepening slopes, and the infiltration or diversion of storm water run off[29]. For example, excavation into the base of marginally stable slopes may reduce forces resisting failure on those slopes, thus causing movement. Adding fill and/or a structure to the top or mid portion of a slope increases the driving forces on a slope and may contribute to failure. Redirecting water onto or into slopes may exploit existing planes of weakness within those slopes, causing failure.

Review of map IMS-22[30] indicates that the site is not located within a mapped “Potential Landslide Hazard Zone.” Review of the Statewide Landslide Information Database for Oregon (SLIDO-2)[31], indicates that no mapped landslides are located on or immediately adjacent to the site. Susceptibility to earthquake-induced landslides is discussed in Section A.4.3.1 of this report.

A.4.2 Flooding

The Federal Emergency Management Agency (FEMA) publishes the Flood Insurance Rate Maps (FIRM) for flood insurance purposes[32]. The FIRM map shows that that the entire site is located within Zone X, outside the mapped 100-year floodplain for the West and Middle forks of Pringle Creek.

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[29] Hofmeister, R., Madin, I., Wang, Y., and Hasenberg, C. 2003, Earthquake and Landslide Hazards Maps and Future Earthquake Damage Estimates, Clackamas County, Oregon: Oregon Department of Geology and Mineral Industries, Open File Report OFR 0-03-10.

[30] Hofmeister, R., Miller, D., Mills, K., Hinkle, J., and Beier, A.,2002. GIS overview map of potential rapidly moving landslide hazards in western Oregon, Oregon Department of Geology and Mineral Industries, Interpretive Map Series 22.

[31] http://www.oregongeology.org/slido/

[32] FEMA, 2003. Flood Insurance Rate Map, Marion County, Oregon and Incorporated Areas. Map Number 41047C0344 H.

A.4.3 Seismic Hazards

Review of Map GMS-105[33] indicates the site is located in seismic hazard Zones B through D, with a low to high relative earthquake hazard. In general, the localized ridges located in the southwestern portion of the site are mapped as Zone C (low to intermediate hazard), the central portion of the site as Zone D (lowest hazard), and the northeastern margin of the site as Zone B (intermediate to high hazard). This map is excerpted on Figure A3. The relative earthquake hazard is based on the evaluation of susceptibility to earthquake-induced landsliding, potential for soil liquefaction, and amplification of ground shaking during a seismic event. The hazard zone indicates which areas have the greatest tendency to experience any one of, or a combination of, these individual hazards. The relative hazard was dominated by earthquake-induced landslide and amplification hazards. The individual hazards are discussed in more detail below.

A.4.3.1 Slope Instability

The landslide susceptibility map from GMS-105, excerpted on Figure A4, indicates the potential for seismically induced landslides, with categories ranging from 0 to 5. Categories 0 (<6 degrees of slope angle) through 4 (>22 degrees of slope angle) are based solely on slope gradient. Category 5 is designated for areas where existing (previous) landslides are identified. Review of the landslide susceptibility map indicates the site is mapped in Categories 1 and 2 (low hazard), reflecting the variable topography across the site. The northeastern margin of the site is located in an area of “High susceptibility to landsliding in areas with existing landslides” (Category 5).

A.4.3.2 Liquefaction

A wide variety of slope and ground failures can occur in response to intense seismic shaking during large magnitude earthquakes. These failures are often related to the phenomenon of liquefaction, the process by which water-saturated sediment changes from a solid to a liquid state. Since liquefied sediment may not support the overlying ground, or any structure built thereon, a variety of failures may occur, including lateral spreading, landslides, ground settlement and cracking, sand boils, oscillation lurching, etc. The conditions necessary for liquefaction to occur are: (1) the presence of poorly consolidated, generally cohesionless sediment; (2) saturation of the sediment by groundwater; and (3) an earthquake that produces intense seismic shaking (generally a moment magnitude greater than M5.0). In general, older, more consolidated sediment, and sediment above the water table will not liquefy[34]. Field performance data and laboratory tests indicate that liquefaction occurs predominantly in well-sorted, loose to medium dense sand or silty sand, but can also occur in lean clays and silts[35].

Review of the liquefaction susceptibility map from GMS-105 indicates no hazard (Category 0) associated with liquefaction at the site.

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[33] Y. Wang and W.J. Leonard, 1996. Relative Earthquake Hazard Maps of the Salem East and Salem West Quadrangles, Marion & Polk Counties, Oregon, Oregon Department of Geology and Mineral Industries GMS-105.

[34] Youd, T.L. and Hoose, S.N. 1978. Historic ground failures in Northern California triggered by earthquakes: U.S. Geological Survey Professional Paper 993, p.117

[35] Seed, R.B., et al. 2003. Recent Advances In Soil Liquefaction Engineering: A Unified And Consistent Framework. Earthquake Engineering Research Center College Of Engineering University Of California, Berkeley.

A.4.3.3 Amplification of Ground Shaking

Thick sequences of unconsolidated, soft sediments typically amplify the shaking of long period ground motions such as those associated with major earthquakes[36]. Areas underlain by shallow soil profiles are not likely to amplify seismic waves. The amplification hazard map from GMS-105 indicates the potential for amplification varies from no susceptibility (Category 0) to possible high susceptibility to amplification in areas of abrupt topographic changes (Category 5). The site is mapped in Categories 0 through 2, with no susceptibility to the southwest and low to moderate susceptibility to the northeast.

A.4.3.4 Estimated Ground Motion

The peak horizontal ground acceleration (PGA) was determined for the site based on data from the United States Geologic Survey (USGS) National Seismic Hazard Mapping Program[37]. PGAs are expressed as a fraction of the acceleration of gravity, and are based on empirical attenuation relationships of seismic wave energy with distance from the causative source. A ground surface (Site Class D) PGA of 0.26g was calculated for the site, with a frequency of occurrence of once in 2,475 years (2% chance of occurrence in any 50 year period). A PGA of 0.26g can potentially cause slight to moderate damage in ordinary structures with considerable damage in poorly built structures.

A.5.0 SITE RECONNAISSANCE

CGT Project Engineering Geologist Jeff Jones, RG, CEG, performed a reconnaissance of the site on April 29, 2014.

A.5.1 Site Surface Conditions

The irregularly-shaped project site is bounded by Leslie Middle School and residential development to the north, a private roadway to the northeast, a grass field to the southeast, Battle Creek Road SE to the southwest, and Pringle Creek Road SE to the west. With the exception of its northeast quadrant, the project site is vacant of any structures and vegetated with grasses, brush (blackberry), and scattered coniferous trees. The northeast quadrant of the site contains several abandoned buildings with appurtenant drive lanes. Site layout, topography, and surface conditions are shown on the Site Plan (Figure 2) and Site Photographs (Figure 3) attached to the geotechnical report. Topographic and generalized geologic profiles of the site are shown on the attached Figure A5.

In terms of topography, the site survey shown on Figure 2 attached to the geotechnical report indicates the site is generally located between about elevation 275 and 340 feet. The site wraps around a localized high, with slopes descending to the west, north, and east. Slope gradients are generally 5H:1V or flatter, with localized areas as steep as 3H:1V. The slopes are generally convex to concave, and exhibit fairly uniform morphology. We did not observe arcuate slopes, uneven topography, tilted tress, disturbed soils, or other obvious signs of previous or on-going slope instability during our reconnaissance.

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[36] Hofmeister, R., Madin, I., Wang, Y., and Hasenberg, C. 2003, *Ibid.*

[37] U.S. Geologic Survey, 2002. National Seismic Hazard Maps, http://earthquake.usgs.gov/research/hazmaps/

A.5.2 Site Subsurface Conditions

Site subsurface conditions are described in Section 5.0 of the geotechnical report. In summary, the site is generally mantled in residual soil (lean to fat clay) resulting from in-place weathering of the underlying basalt. The residual soil was approximately 3 to 10 feet thick, transitioning to decomposed basalt that extended to the full depths explored, approximately 10 feet below ground surface (bgs). Isolated fills were encountered, which were approximately 1 to 4 feet thick.

In the lower, northeastern portion of the site, clay with rounded gravel was encountered in some of the test pits. This material is interpreted as the terrace deposits described above, and extended to the full depths explored where encountered, about 10 feet bgs.

As described in the geotechnical report, perched groundwater was encountered at depths of about 4 to 9 feet bgs within the test pits excavated in the northeastern (lower) portion of the site. Groundwater was not encountered elsewhere on the site.

A.6.0 FINDINGS

It is our opinion that the site is geologically suitable for the proposed development, as described in Section 1.1 of the geotechnical report. The primary geologic hazards identified are associated with slope instability and amplification of seismic shaking. With the use of generally accepted construction techniques and by strictly following the recommendations contained in this report and the attached geotechnical report, we anticipate that the project will have a minimal impact on existing geologic hazards or adjacent properties.

A.6.1 Slope Instability

As indicated in Section A.4.1, the referenced mapping does not indicate a high hazard due to landslides at the site and no known landslides are mapped at or near the site, and we did not observe obvious signs of previous or on-going slope instability during our reconnaissance. As indicated in Section A.4.3.1, the hazard due to seismically-induced landslides is generally low to moderate for the majority of the site.

The northeastern margin of the site is located in an area of "High susceptibility to landsliding in areas with existing landslides" (Category 5). Based on review of the referenced topographic maps and lidar imagery available online[38], the area mapped as Category 5 appears to coincide roughly with the area mapped as terrace deposits referenced in Section A.1.1.2. The terrace deposits include colluvium, slope wash, and alluvial fan deposits. This area appears to be located between two primary drainages, with several interceding, subordinate drainages and the somewhat lobate geomorphology may reflect alluvial fan and slope wash deposits, rather than landslide deposits. It is therefore our opinion that the risk of landslides, seismically-induced or otherwise, in this portion of the site is low to moderate.

As described in the geotechnical report, grading proposed at the site generally consists of cuts and fills limited to less than 5 feet in depth. Site grading should be performed in accordance with the recommendations presented in the geotechnical report. If grading plans change significantly from those described herein, CGT should be consulted to review and, if warranted, revise our conclusions.

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[38] http://www.oregongeology.org/slido/

Control and proper disposal of stormwater runoff from impervious areas is critical in ensuring that the proposed development does not increase the risk of instability of the site slopes. Recommendations regarding drainage are presented in Section 8.8.1 of the geotechnical report.

Provided the recommendations contained in the attached geotechnical report regarding grading and drainage are incorporated into design and construction, the proposed development should not increase the hazard posed by slope instability.

Notwithstanding the above, any construction within hillside areas inherently bears greater risk of slope instability. This risk increases in seismically active areas, such as the Pacific Northwest. The existing site slopes may be susceptible to slope instability resulting from factors beyond the owner’s control, such as a major earthquake, heavy precipitation, or off-site human activities. The owners must recognize and accept the risk of potential slope instability from causes beyond their control or as yet unrecognized.

A.6.2 Seismic Shaking

The proposed development will have no impact on this hazard. To minimize the risk that this hazard will adversely impact the proposed development, the structures should be designed and constructed in accordance with current building codes (2011 Oregon Residential Specialty Code as of the date of this report).

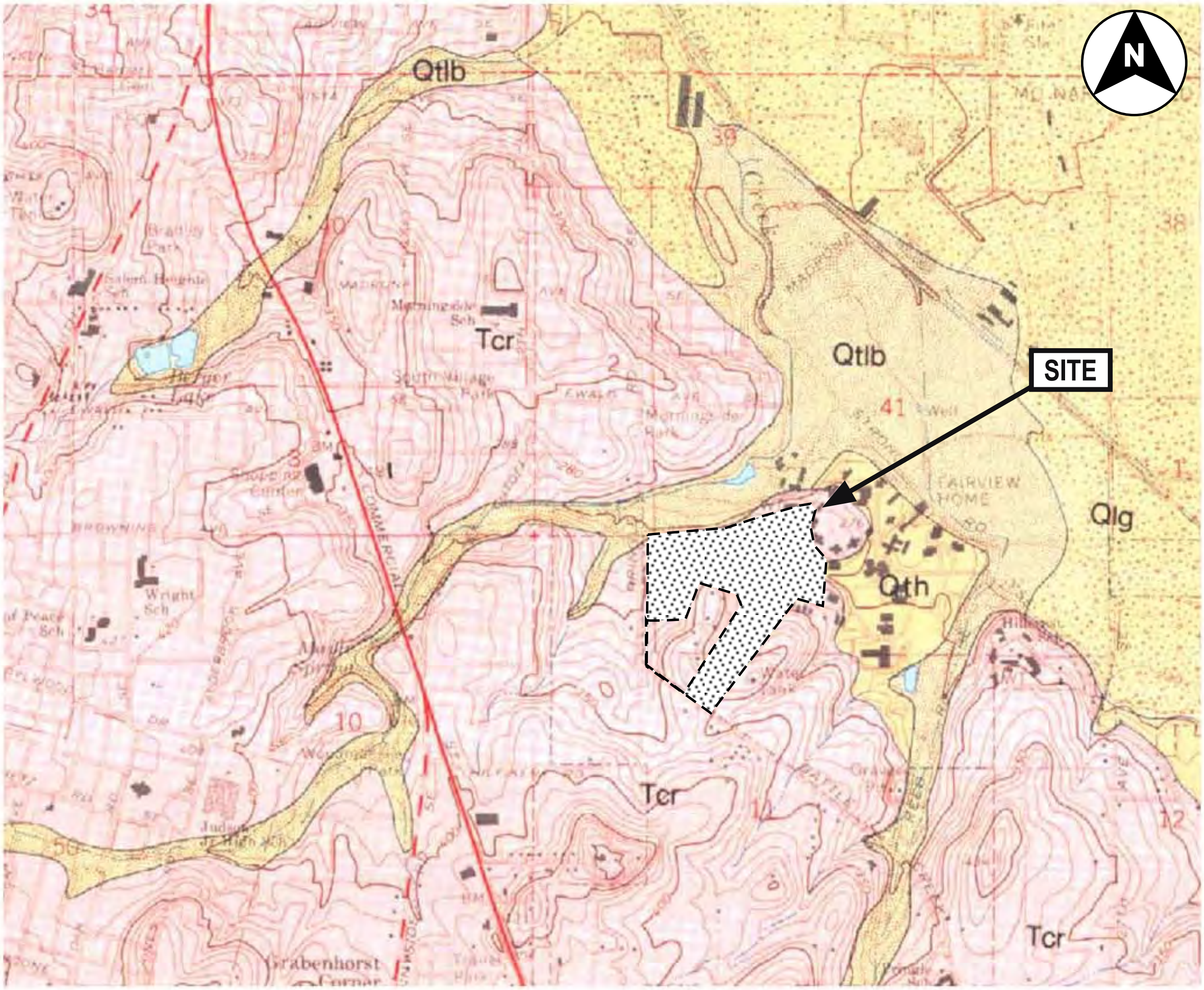
A.6.3 Other Hazards

Other geologic hazards identified in the Oregon State Board of Geologist Examiners Guidelines for Preparing Engineering Geologic Reports in Oregon include:

- Flooding/Inundation
- Subsidence
- Fault Rupture

Based on our research, field reconnaissance, and previous experience in the area, none of these hazards are present at the site.

FAIRVIEW SUBDIVISION - SALEM, OREGON
GEOLOGIC MAP



Qal	Recent river alluvium
Qtlw	Lower Willamette River terrace deposits
Qtlt	Lower terrace deposits of tributary rivers
Qtlb	Lower terrace deposits of alluvial bottomlands
Qtm	Middle terrace deposits
Qlg	Linn gravel
Qth	Higher terrace deposits
Tcr	Columbia River Basalt Group
Toe	Eocene-Oligocene sedimentary rock
Ts	Upper Eocene sandstone
Ty	Yamhill Formation
Tsr	Siletz River Volcanics
bc	Basaltic Colluvium and/or landslide debris

Map adapted from Bela, 1981, Geology of the Rickreal, Salem West, Monmouth, and Sidney NE 7.5 Minute Quadrangles, Oregon Department of Geology and Mineral Industries, GMS-18.

Scale 1 Inch = 2,000 feet

0 2000 4000

Township 8 South, Range 3 West, Section 11 Willamette Meridian

CARLSON CGT GEOTECHNICAL 503-601-8250

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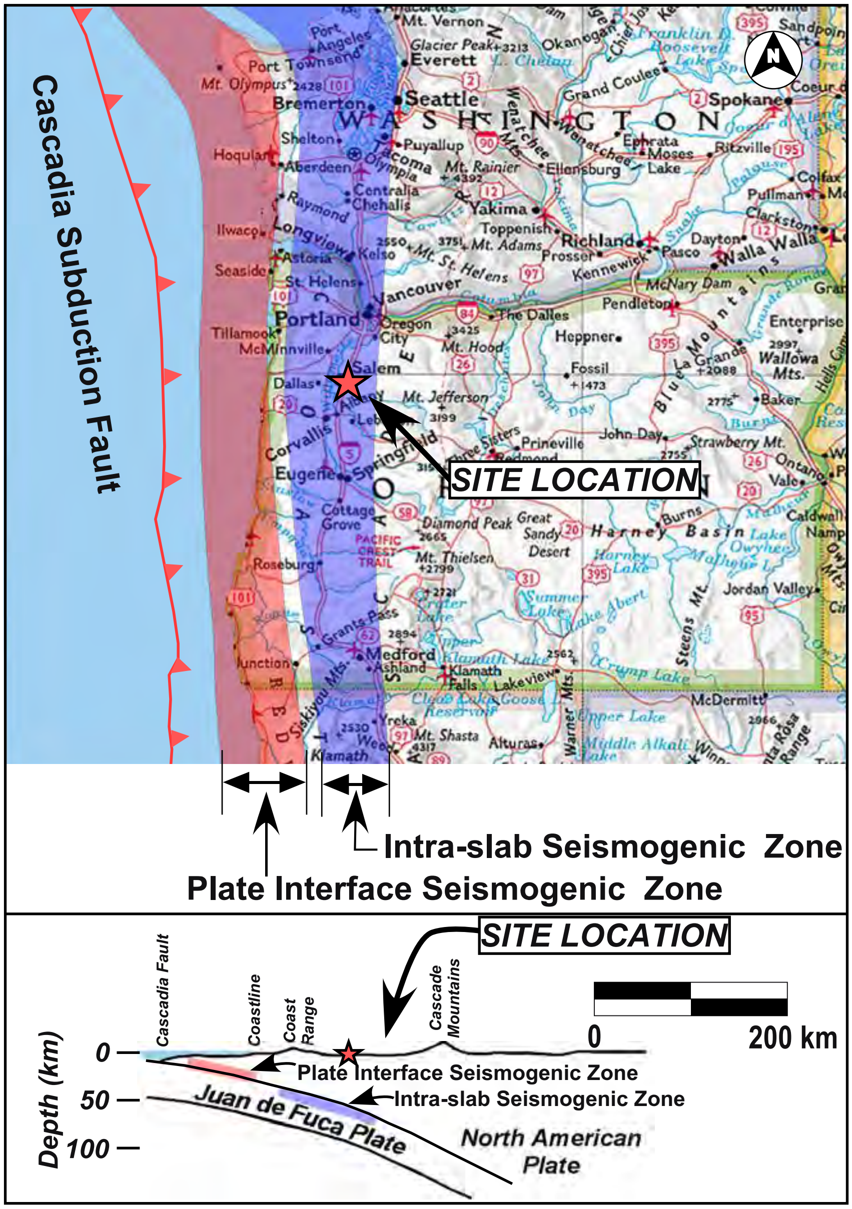
Tigard, Oregon 97281

CGT Job No. G1404007

FIGURE A1

FAIRVIEW SUBDIVISION - SALEM, OREGON

CASCADIA SUBDUCTION ZONE



McCrory, Blair, Oppenheimer, and Walter, 2004. Depth to the Juan de Fuca slab beneath the Cascadia subduction margin - A 3-D model for storing earthquakes: U.S. Geological Survey Data Series 91.

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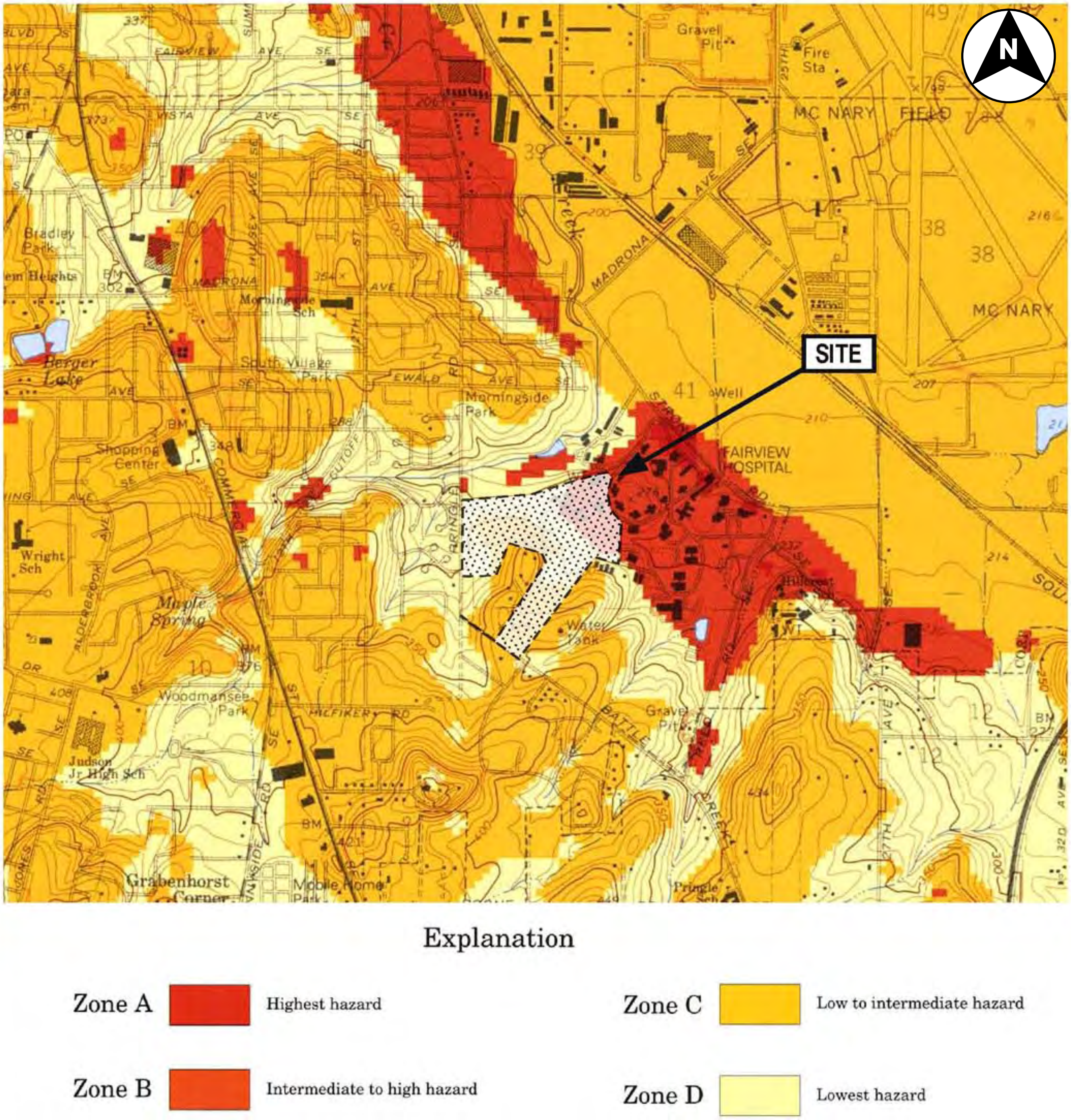
FIGURE A2

FAIRVIEW SUBDIVISION - SALEM, OREGON

RELATIVE EARTHQUAKE HAZARD MAP



Map adapted from Wang, Y., and Leonard, W., 1996, Relative Earthquake Hazard
Map of the Salem East and Salem West Quadrangles, Marion and Polk Counties, Oregon.
Oregon Department of Geology and Mineral Industries GMS-105.

Scale 1 Inch = 2,000 feet

0 2000 4000

Township 8 South, Range 3 West, Section 11 Willamette Meridian

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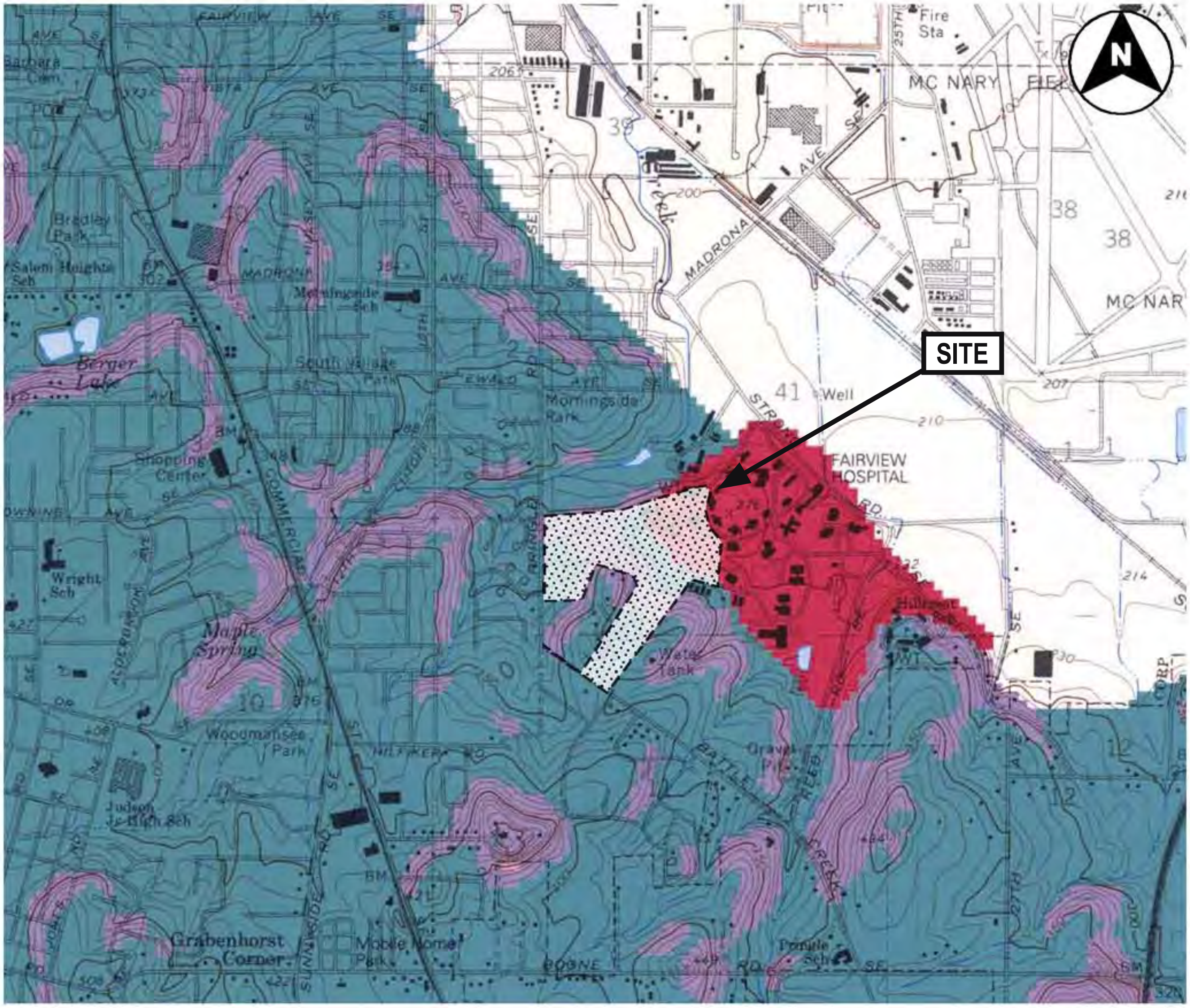
FIGURE A3

FAIRVIEW SUBDIVISION - SALEM, OREGON

LANDSLIDE SUSCEPTIBILITY MAP



Explanation

Category 5	🟥	High susceptibility to landsliding in areas with existing landslides	Category 2	🟪	≥6-14 degrees of slope angle
Category 4	🟥	>22 degrees of slope angle	Category 1	🟦	<6 degrees of slope angle in hills
Category 3	🟥	≥14-22 degrees of slope angle	Category 0	⬜	<6 degrees of slope angle in valley

Map adapted from Wang, Y., and Leonard, W., 1996, Landslide Susceptibility Map of the Salem East and Salem West Quadrangles, Marion and Polk Counties, Oregon. Oregon Department of Geology and Mineral Industries GMS-105.

Scale 1 Inch = 2,000 feet

0 2000 4000

Township 8 South, Range 3 West, Section 11 Willamette Meridian

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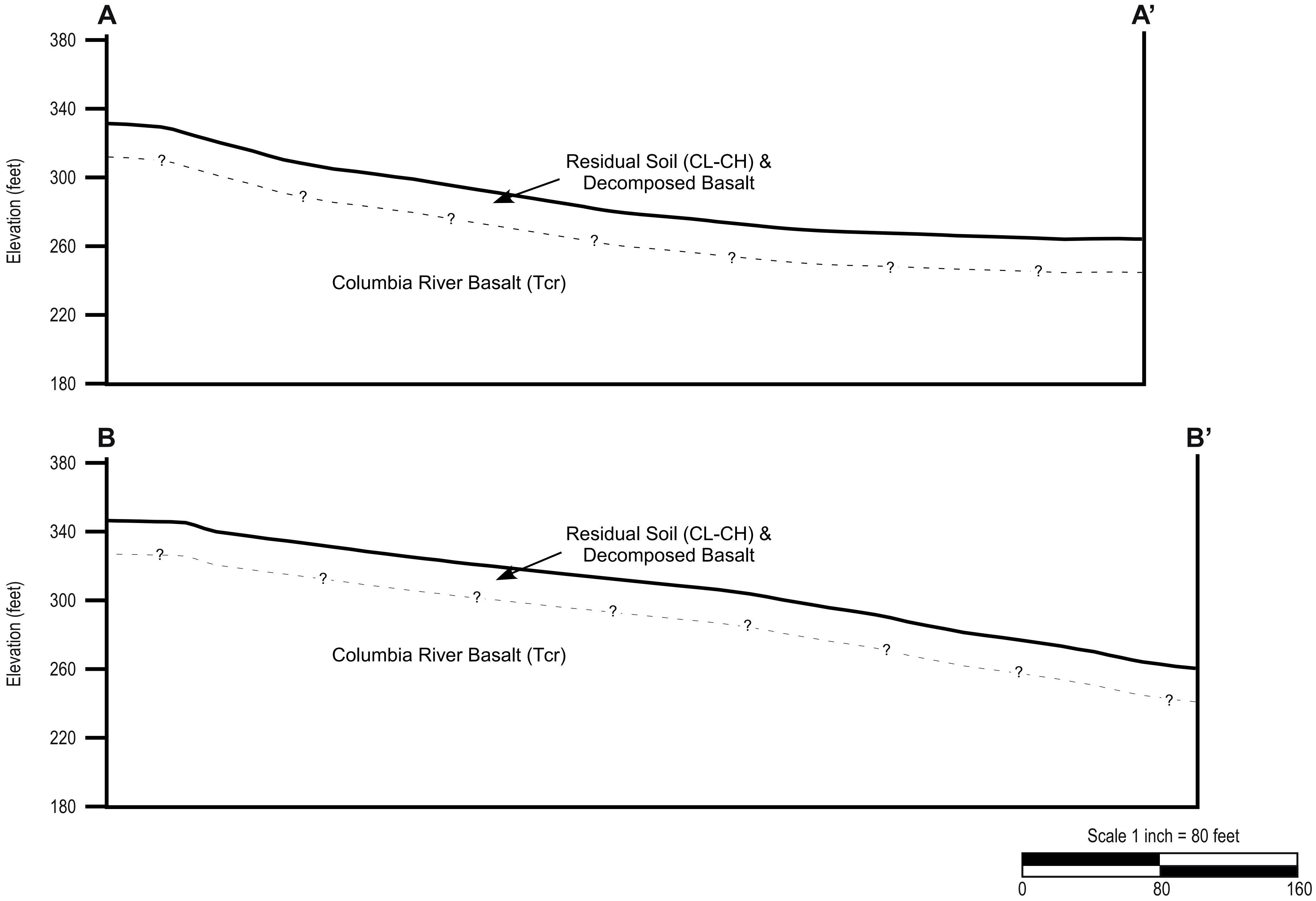
FIGURE A4



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FAIRVIEW SUBDIVISION - SALEM, OREGON

GEOLOGIC CROSS SECTIONS

CGT Job No. G1404007

FIGURE A5

Carlson Geotechnical
A Division of Carlson Testing, Inc.
Phone: (503) 601-8250
Fax: (503) 601-8254

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

Appendix B: Results of Infiltration Testing

Fairview Subdivision
Pringle Road SE & Battle Creek Road SE
Salem, Oregon

CGT Project No. G1404007

May 23, 2014

Prepared For:

Mr. Eric Olsen
Olsen Design & Development
170 West Main Street
Monmouth, Oregon 97361

Prepared By:

Carlson Geotechnical

B.1.0 CORRESPONDENCE WITH DESIGN TEAM

The project civil engineer, Westech Engineering (Westech), requested twenty infiltration tests be performed at the site at maximum depths of 5 feet below existing site grades. Westech has requested the tests be performed relatively uniformly across the project site. The approximate locations of the infiltration tests (designated as IT-1 through IT-18) are shown on the Site Plan, which is attached to the report as Figure 2. Testing was omitted at test pits TP-7 and TP-8, as relatively shallow groundwater was encountered at those exploration locations.

B.2.0 TEST PROCEDURE

Eighteen infiltration tests were performed within prepared test pits at the site on April 30, 2014, in general accordance with the Encased Falling Head test method described in Section 4C.3(d) of the City of Salem Department of Public Works Administrative Rules Manual 109-001 (January 2014). The following table presents the depth of the tests and the subsurface material encountered at the test depths.

Table B1. Infiltration Test Depths & Materials

Infiltration Test	Test Pit	Test Depth[1] (feet bgs)	Test Elevation[2] (feet)	Subsurface Material at the Test Depth
IT-1	TP-1	5	355	Decomposed Basalt (RX)
IT-2	TP-2	5	361	Lean to Fat Clay (CL-CH)
IT-3	TP-3	3½	349½	Lean to Fat Clay (CL-CH)
IT-4	TP-4	5	342	Decomposed Basalt (RX)
IT-5	TP-5	5	295	Decomposed Basalt (RX)
IT-6	TP-6	5	285	Lean to Fat Clay (CL-CH)
IT-7	TP-9	5	267	Lean to Fat Clay (CL-CH)
IT-8	TP-10	5	265	Lean to Fat Clay (CL-CH)
IT-9	TP-11	5	261	Lean to Fat Clay (CL-CH)
IT-10	TP-12	5	245	Lean to Fat Clay (CL-CH)
IT-11	TP-13	5	270	Lean to Fat Clay (CL-CH)
IT-12	TP-14	5	291	Lean to Fat Clay (CL-CH)
IT-13	TP-15	4	253	Decomposed Basalt (RX)
IT-14	TP-16	5	255	Lean to Fat Clay (CL-CH)
IT-15	TP-17	3½	300½	Lean to Fat Clay (CL-CH)
IT-16	TP-18	4	315	Lean to Fat Clay (CL-CH)
IT-17	TP-19	3	317	Lean to Fat Clay (CL-CH)
IT-18	TP-20	2	300	Lean to Fat Clay (CL-CH)

[1] Relative to existing site grades. bgs = below ground surface.

[2] Determined from provided site topographic survey prepared by Barker Surveying. Elevation should be considered approximate.

In each case, the test pit was excavated to the respective test depth and a 6-inch inner-diameter, PVC pipe was hydraulically pushed (using the excavator bucket) into the soil horizon approximately 6 inches. An approximate 2-inch thick layer of clean gravel was placed within the base of the pipes. The subsurface soils at the base of the pipes were "soaked" in accordance with the referenced test method by pouring about 12 inches of water (measured vertically) into the test pipes. After allowing the soils to soak overnight, testing was initiated by recording the drop in water level of an approximate 6-inch column of water at 30-minute intervals. A minimum of three trials was administered for each infiltration test.

B.3.0 TEST RESULTS

The following table presents the raw data and calculated rates of infiltration that we observed from these infiltration tests. Please note the calculated infiltration rates do <u>not</u> include any safety or correction factors.

Table B2. Results of Infiltration Test IT-1

Infiltration Test	Trial	Time Interval (minutes)	Drop in Water Level[1] (inches)	Raw Infiltration Rate[2] (inches per hour)
IT-1	1	30	⅜	¾ (see note 2)
	2	30	$^{5}/_{16}$	⅝ (see note 2)
	3	30	$1^{5}/_{16}$	2⅝ (see note 2)
	4	30	$1^{5}/_{16}$	2⅝ (see note 2)

[1] Variability in test results due to improper seal of PVC pipe in test pit (due to high concentration of rock fragments).

[2] Rates shown are **not** representative of actual conditions. Supplemental testing (using the test pit method) is recommended to refine actual infiltration rate of weathered rock material. Quantity of water required for test pit method not available at time of field testing.

Table B3. Results of Infiltration Test IT-2

Infiltration Test	Trial	Time Interval (minutes)	Drop in Water Level (inches)	Raw Infiltration Rate (inches per hour)
IT-2	1	30	⅛	¼
	2	30	¼	½
	3	30	¼	½
	4	30	¼	½

Table B4. Results of Infiltration Test IT-3

Infiltration Test	Trial	Time Interval (minutes)	Drop in Water Level (inches)	Raw Infiltration Rate (inches per hour)
IT-3	1	30	$^{7}/_{16}$	$^{7}/_{8}$
	2	30	$^{7}/_{16}$	$^{7}/_{8}$
	3	30	$^{7}/_{16}$	$^{7}/_{8}$
	4	30	⅜	¾
	5	30	$^{1}/_{16}$	⅛

Table B5. Results of Infiltration Test IT-4

Infiltration Test	Trial	Time Interval (minutes)	Drop in Water Level[1] (inches)	Raw Infiltration Rate[2] (inches per hour)
IT-4	1	30	$7^{7}/_{16}$	$14^{7}/_{8}$ (see note 2)
	2	30	---	Could not be determined
	3	30	---	Could not be determined
	4	30	2½	5 (see note 2)
	5	30	1⅛	2¼ (see note 2)

[1] Variability in test results due to improper seal of PVC pipe in test pit (due to high concentration of rock fragments).

[2] Rates shown are **not** representative of actual conditions. Supplemental testing (using the test pit method) is recommended to refine actual infiltration rate of weathered rock material. Quantity of water required for test pit method not available at time of field testing.

Table B6. Results of Infiltration Test IT-5

Infiltration Test	Trial	Time Interval (minutes)	Drop in Water Level (inches)	Raw Infiltration Rate (inches per hour)
IT-5	1	30	⅛	¼
	2	30	⅛	¼
	3	30	⅛	¼
	4	30	1/16	⅛

Table B7. Results of Infiltration Test IT-6

Infiltration Test	Trial	Time Interval (minutes)	Drop in Water Level (inches)	Raw Infiltration Rate (inches per hour)
IT-6	1	30	5/8	1¼
	2	30	9/16	1⅛
	3	30	¾	1½
	4	30	5/8	1¼

Table B8. Results of Infiltration Test IT-7

Infiltration Test	Trial	Time Interval (minutes)	Drop in Water Level (inches)	Raw Infiltration Rate (inches per hour)
IT-7	1	30	5/8	1¼
	2	30	5/8	1¼
	3	30	¼	½
	4	30	5/8	1¼

Table B9. Results of Infiltration Test IT-8

Infiltration Test	Trial	Time Interval (minutes)	Drop in Water Level (inches)	Raw Infiltration Rate (inches per hour)
IT-8	1	30	¼	½
	2	30	¼	½
	3	30	¼	½
	4	30	¼	½

Table B10. Results of Infiltration Test IT-9

Infiltration Test	Trial	Time Interval (minutes)	Drop in Water Level (inches)	Raw Infiltration Rate (inches per hour)
IT-9	1	30	⅛	¼
	2	30	⅛	¼
	3	30	3/8	¾
	4	30	¼	½

Table B11. Results of Infiltration Test IT-10

Infiltration Test	Trial	Time Interval (minutes)	Drop in Water Level (inches)	Raw Infiltration Rate (inches per hour)
IT-10	1	30	1/16	⅛
	2	30	1/16	⅛
	3	30	0	0
	4	30	⅛	¼

Table B12. Results of Infiltration Test IT-11

Infiltration Test	Trial	Time Interval (minutes)	Drop in Water Level[1] (inches)	Raw Infiltration Rate[2] (inches per hour)
IT-11	1	30	3⅛	6¼ (see note 2)
	2	30	3⅜	6¾ (see note 2)
	3	30	2¾	5½ (see note 2)
	4	30	2	4 (see note 2)

[1] Variability in test results due to improper seal of PVC pipe in test pit (due to high concentration of rock fragments).

[2] Rates shown are **not** representative of actual conditions. Supplemental testing (using the test pit method) is recommended to refine actual infiltration rate of weathered rock material. Quantity of water required for test pit method not available at time of field testing.

Table B13. Results of Infiltration Test IT-12

Infiltration Test	Trial	Time Interval (minutes)	Drop in Water Level (inches)	Raw Infiltration Rate (inches per hour)
IT-12	1	30	½	1
	2	30	⅜	¾
	3	30	½	1
	4	30	⅜	¾

Table B14. Results of Infiltration Test IT-13

Infiltration Test	Trial	Time Interval (minutes)	Drop in Water Level (inches)	Raw Infiltration Rate (inches per hour)
IT-13	1	30	⅝	1¼
	2	30	⅝	1¼
	3	30	⅝	1¼
	4	30	⅜	¾

Table B15. Results of Infiltration Test IT-14

Infiltration Test	Trial	Time Interval (minutes)	Drop in Water Level (inches)	Raw Infiltration Rate (inches per hour)
IT-14	1	30	⅝	1¼
	2	30	⅝	1¼
	3	30	½	1
	4	30	½	1

Table B16. Results of Infiltration Test IT-15

Infiltration Test	Trial	Time Interval (minutes)	Drop in Water Level (inches)	Raw Infiltration Rate (inches per hour)
IT-15	1	30	¾	1½
	2	30	⅝	1¼
	3	30	¾	1½
	4	30	¾	1½

Table B17. Results of Infiltration Test IT-16

Infiltration Test	Trial	Time Interval (minutes)	Drop in Water Level (inches)	Raw Infiltration Rate (inches per hour)
IT-16	1	30	¼	½
	2	30	¼	½
	3	30	¼	½
	4	30	¼	½

Table B18. Results of Infiltration Test IT-17

Infiltration Test	Trial	Time Interval (minutes)	Drop in Water Level[1] (inches)	Raw Infiltration Rate[2] (inches per hour)
IT-17	1	30	1½	3 (see note 2)
	2	30	1½	3 (see note 2)
	3	30	1	2 (see note 2)
	4	30	1¼	2½ (see note 2)

[1] Variability in test results due to marginal seal of PVC pipe in test pit (due to moderate concentration of rock fragments).

[2] Rates shown are **not** representative of actual conditions. Supplemental testing (using the test pit method) is recommended to refine actual infiltration rate of weathered rock material. Quantity of water required for test pit method not available at time of field testing.

Table B19. Results of Infiltration Test IT-18

Infiltration Test	Trial	Time Interval (minutes)	Drop in Water Level (inches)	Raw Infiltration Rate (inches per hour)
IT-18	1	30	½	1
	2	30	⁵/₈	1¼
	3	30	½	1
	4	30	½	1

B.4.0 DISCUSSION

As indicated in the preceding section, we calculated raw infiltration rates ranging from about ¼ to 1¼ inches per hour in the test pits. These infiltration rates do <u>not</u> include any safety or correction factors. It is recommended the infiltration system designer consult the appropriate design manual in order to assign appropriate safety/correction factors to calculate the design infiltration rate for the infiltration system. Once the design is completed, we recommend the infiltration system design (provided by others) and location be reviewed by CGT. If the location and/or depth of the system changes from what was indicated at the time of our fieldwork, additional testing may be recommended.

As indicated above, a proper seal of the PVC pipes could not be achieved at infiltration test locations IT-1, IT-4, IT-11, and IT-17 due to the presence of rock fragments at the test depths. The infiltration rates calculated at those locations are <u>not</u> anticipated to be representative of actual site conditions. Supplemental testing using an alternative test method (such as the test pit method) is recommended to refine actual infiltration rates of the decomposed basalt (RX) at those locations, if desired. CGT would be pleased to perform supplemental infiltration testing at the site, upon request, for an additional fee.