Redmond Geotechnical Services

Geotechnical Investigation and Consultation Services

Proposed Middle Housing Residential Development Project

Tax Lot No. 3100

3200 Pheasant Avenue SE

Salem (Marion County), Oregon

for

Montgomery Construction Group

Project No. 2054.001.G
April 9, 2024

REDMOND GEOTECHNICAL SERVICES

April 09, 2024

Mr. Chad Montgomery
Montogomery Construction Group
3755 Amber Street NE
Salem, Oregon 97301

Dear Mr. Montgomery:

Re: Geotechnical Investigation and Consultation Services,
Proposed Middle Housing Residential Development Site, Tax Lot No. 3100,
3200 Pheasant Avenue SE, Salem (Marion County), Oregon

Submitted herewith is our report entitled "Geotechnical Investigation and Consultation Services, Proposed Middle Housing Residential Development Site, Tax Lot No. 3100, 3200 Pheasant Avenue SE, Salem (Marion County), Oregon". The scope of our services was outlined in our formal proposal to Mr. Chad Montgomery of Montgomery Construction Group dated March 4, 2024. Authorization of our services was provided by Mr. Chad Montgomery on March 4, 2024.

During the course of our investigation, we have kept you and/or others advised of our schedule and preliminary findings. We appreciate the opportunity to assist you with this phase of the project. Should you have any questions regarding this report, please do not hesitate to call.

Sincerely,

Daniel M. Redmond, P.E., G.E.
President/Principal Engineer

REGISTERED PROFESSIONAL ENGINEER
12,309
OREGON
July 15, 1983
DANIEL M. REDMOND
Exp. 12-31-24

Cc: Mr. Gene Bolante
Studio 3 Architecture

TABLE OF CONTENTS

	Page No.
INTRODUCTION	1
PROJECT DESCRIPTION	1
SCOPE OF WORK	2
SITE CONDITIONS	3
Site Geology	3
Surface Conditions	3
Subsurface Soil Conditions	4
Groundwater	4
INFILTRATION TESTING	5
LABORATORY TESTING	5
SEISMICITY AND EARTHQUAKE SOURCES	5
Liquefaction	6
Landslides	7
Surface Rupture	7
Tsunami and Seiche	7
Flooding and Erosion	7
CONCLUSIONS AND RECOMMENDATIONS	8
General	8
Site Preparation	9
Foundation Support	10
Shallow Foundations	10
Floor Slab Support	11

Table of Contents (continued)

Retaining/Below Grade Walls 11

Pavements 12

Minor Arterial Street 13

Private Parking and Access Drives 13

Pavement Subgrade, Base Course & Asphalt Materials 14

Wet Weather Grading and Soft Spot Mitigation 14

Shrink-Swell and Frost Heave 15

Excavations/Slopes 15

Surface Drainage/Groundwater 15

Design Infiltration Rates 16

Seismic Design Considerations 16

CONSTRUCTION MONITORING AND TESTING 17

CLOSURE AND LIMITATIONS 17

LEVEL OF CARE 18

REFERENCES 19

ATTACHMENTS

Figure No. 1 - Site Vicinity and Geologic Map
Figure No. 2 - Site Exploration Plan
Figure No. 3 - Typical Perimeter Footing/Retaining Wall Drain Detail
Figure No. 4 - DOGAMI Slido Map
Figure No. 5 - DOGAMI Lidar Map
Figure No. 6 - DOGAMI Landslide Hazard Map

APPENDIX A

Test Pits/Core Hole Logs and Laboratory Data

GEOTECHNICAL INVESTIGATION AND CONSULTATION SERVICES PROPOSED MIDDLE HOUSING RESIDENTIAL DEVELOPMENT SITE TAX LOT NO. 3100, 3200 PHEASANT AVENUE SE SALEM (MARION COUNTY) OREGON

INTRODUCTION

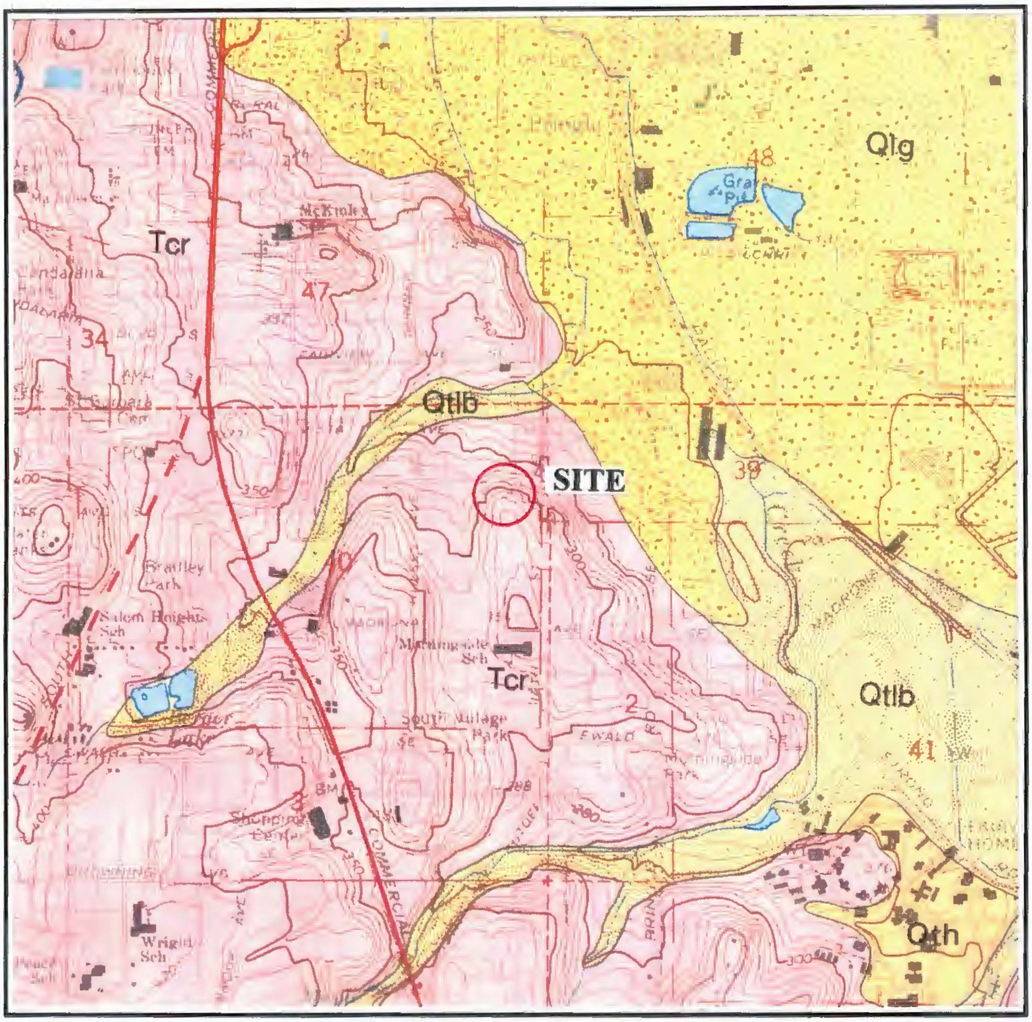
Redmond Geotechnical Services, LLC is please to submit to you the results of our Geotechnical Investigation and Consultation Services at the site of the proposed new Middle Housing residential development site located to the east of Pheasant Avenue SE and south of Ratcliff Drive SE in Salem (Marion County), Oregon. The general location of the subject site is shown on the Site Vicinity and Geologic Map, Figure No. 1. The purpose of our geotechnical investigation and consultation services at this time was to explore the existing subsurface soils and/or groundwater conditions across the subject site and to evaluate any potential concerns with regard to potential slope failure at the site as well as to develop and/or provide appropriate geotechnical design and construction recommendations for the proposed new Middle Housing residential development project.

PROJECT DESCRIPTION

Based on a review of the proposed site development plan, we understand that present plans for the project will consist of the construction of four (4) new residential structures at the site. Reportedly, the new residential structures will be single-family and/or duplex wood-frame structures with a base and/or ground floor footprint of about 946 and 1,892 square feet, respectively. Support for the proposed new single-family and/or duplex structures is anticipated to consist primarily of conventional shallow continuous (strip) footings although some individual (spread) column-type footings are also likely. Structural loading information, although currently unavailable, is expected to result in maximum dead plus live continuous (strip) and individual (spread) column-type footings loads on the order of about 2.0 to 3.0 kips per lineal foot (klf) and 10 to 25 kips, respectively.

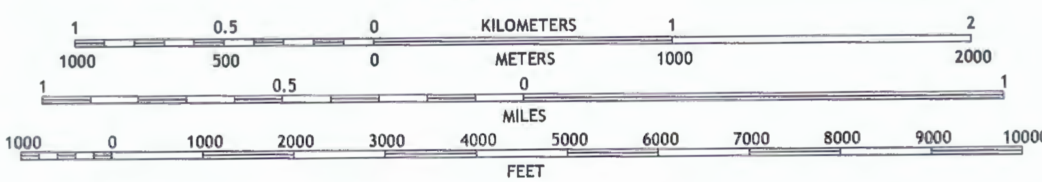
Additionally, we understand that the project will also include new paved surfaces for both automobile parking and drive areas as well as new public street improvements to the easterly north bound lane of Pheasant Avenue SE. Further, we understand that stormwater from hard and/or impervious surfaces (i.e., roofs and pavements), will be collected for on-site treatment and possible disposal in a stormwater facility located near the northwesterly portion of the site.

Earthwork and grading for the project is unknown at this time. However, based on the relatively flat-lying to gently sloping nature of the proposed residential site, we anticipate that relatively minor cuts and/or fills of about two (2) to four (4) feet will be required.



SALEM WEST QUADRANGLE
OREGON
7.5-MINUTE SERIES

SCALE 1:24 000



CONTOUR INTERVAL 10 FEET
NORTH AMERICAN VERTICAL DATUM OF 1988

SITE VICINITY AND GEOLOGIC MAP

Project No. 2054.001.G	**MIDDLE HOUSING DEVELOPMENT TL 3100, 3200 PHEASANT AVE SE**	Figure No. 1

SCOPE OF WORK

The purpose of our geotechnical and/or geologic studies was to evaluate the overall subsurface soil and/or groundwater conditions underlying the subject site with regard to the proposed new residential development and construction at the site and any associated impacts or concerns with respect to potential slope failure at the site as well as provide appropriate geotechnical design and construction recommendations for the project. Specifically, our geotechnical investigation and landslide hazard study included the following scope of work items:

1. Review of available and relevant geologic and/or geotechnical investigation reports for the subject site and/or area.

2. A detailed field reconnaissance and subsurface exploration program of the soil and groundwater conditions underlying the site by means of two (2) exploratory core holes and four (4) exploratory test pits. The two (2) core holes were cored along the easterly north bound lane of Pheasant Avenue SE and the four (4) exploratory test pit were excavated to depths of between seven (7) and eight (8) feet beneath existing site grades at the approximate location as shown on the Site Exploration Plan, Figure No. 2. Further, field infiltration testing was also performed within one (1) of the test pits (TH-#1) excavated at the subject site.

3. Laboratory testing to evaluate and identify pertinent physical and engineering properties of the subsurface soils encountered relative to the planned site development and construction at the site. The laboratory testing program included tests to help evaluate the natural (field) moisture content and dry density, Atterberg Limits and gradational characteristics as well as direct shear strength and "R"-value tests.

4. A literature review and engineering evaluation and assessment of the regional seismicity to evaluate the potential ground motion hazard(s) at the subject site. The evaluation and assessment included a review of the regional earthquake history and sources such as potential seismic sources, maximum credible earthquakes, and reoccurrence intervals as well as a discussion of the possible ground response to the selected design earthquake(s), fault rupture, landsliding, liquefaction, and tsunami and seiche flooding.

5. Engineering analyses utilizing the field and laboratory data as a basis for furnishing recommendations for foundation support of the proposed new residential structures. Recommendations include maximum design allowable contact bearing pressure(s), depth of footing embedment, estimates of foundation settlement, lateral soil resistance, and foundation subgrade preparation. Additionally, construction and/or permanent subsurface water drainage considerations have also been prepared. Further, our report includes recommendations regarding site preparation, placement and compaction of structural fill materials, suitability of the on-site soils for use as structural fill, criteria for import fill materials, and preparation of foundation, pavement and/or floor slab subgrades.

6. Flexible pavement design and construction recommendations for the proposed new paved private access drives and parking as well as the new public street improvements.

SITE CONDITIONS

Site Geology

According to published mapping (Geologic Map of the Salem West Quadrangle, dated 1981 and geologic mapping for Marion County (NW Geological Services, 1997), the subject site and/or site area is underlain at depth by Columbia River Basalt (Tcr) bedrock deposits of Miocene age.

Logs of nearby wells indicate that the Columbia River basalt deposits extend to depths of 400 to 600 feet. Characteristics include medium-gray to black, fine grained even-textured to slightly porphyritic basalt; un-weathered flows generally dense, fairly crystalline, exhibiting massive columnar jointing near base to diced or hackly jointing in entablature. Unit consists of weathered and un-weathered basaltic lava flows with interflow zones characterized by vesicular flow-top breccia, ash and baked soils. Maximum thickness is greatly modified by erosion and weathering in many places with individual flows ranging from 40 to more than 100-feet.

Weathered flows consist of reddish-brown to grayish-brown, crumbly to medium dense basalt. Weathering is variable and believed related to individual basalt flows; some exposures are altered to red clay (laterite) to depths of 30-feet, and occasionally as deep as 65 to 175-feet, while others are only slightly weathered at the surface.

Surface Conditions

The subject proposed new Middle Housing property consists of one (1) rectangular shaped tax lot (TL 3100) which encompasses a total plan area of approximately 0.26 acres. The proposed new Middle Housing property is roughly located to the east of Pheasant Avenue SE and to the north, south and west by existing and/or developed residential properties.

The subject proposed Middle Housing site is presently unimproved. However, the subject property was previously developed and contained a single-family residential structure. Surface vegetation across the site generally consists of a light to moderate growth of grass, weeds and brush as well as several small to large sized trees.

Topographically, the site are characterized as relatively flat-lying to gently sloping terrain (i.e., 5 to 10 percent) descending downward towards the northwest towards Pheasant Avenue SE. Overall topographic relief across the site is estimated at about twenty-two (22) feet and ranges from a low about Elevation 285 feet in the northwesterly corner of the site to a high of about Elevation 307 feet near the upper southeasterly corner of the site.

Subsurface Soil Conditions

Our understanding of the subsurface soil conditions underlying the site was developed by means of two (2) exploratory core holes in the easterly northbound lane of Pheasant Avenue SE and four (4) exploratory test pits excavated to depths of about seven (7) to eight (8) feet beneath existing site grades on March 6 and March 7, 2024, respectively, with tracked excavating equipment. As well as The location of the exploratory core holes and test pits were located in the field by marking off distances from existing and/or known site features and are shown in relation to the existing site improvements on the Site Exploration Plan, Figure No. 2. Detailed logs of the test pits and core holes, presenting conditions encountered at the locations explored, are presented in the Appendix, Figure No's. A-4 through A-6.

The exploratory test pit excavations and core holes were observed by staff from Redmond Geotechnical Services, LLC who logged the test pit explorations and/or core holes and obtained representative samples of the subsurface soils encountered at the site. Additionally, the elevation of the exploratory test pit excavation was referenced from a topographic survey and/or proposed site development plan and should be considered as approximate. All subsurface soils encountered at the site and/or within the exploratory test pit excavation were logged and classified in general conformance with the Unified Soil Classification System (USCS) which is outlined on Figure No. A-3.

The test pit explorations revealed that the subject site is underlain by native soil deposits comprised of a surficial layer of dark brown, wet, soft, organic, sandy, clayey silt topsoil material to depths of about 10 to 14 inches. These surficial topsoil materials were inturn underlain by medium to reddish-brown, very moist, soft to medium stiff, sandy, clayey silt residual soils to depths of about four (4) to five (5) feet beneath the existing site and/or surface grades. These sandy, clayey silt soil deposits exhibit characteristics of highly weathered bedrock at a depth of about four (4) to five (5) feet and are best characterized by relatively moderate strength and low to moderate compressibility. The upper residual soils were inturn underlain at depth by reddish-to orangish-brown, moist, medium dense to dense, highly to moderately weathered basalt bedrock deposits to the maximum depth explored of about eight (8) feet beneath the existing site and/or surface grades. The highly to moderately weathered Basalt bedrock deposits are best characterized by relatively moderate to high strength and low compressibility.

Groundwater

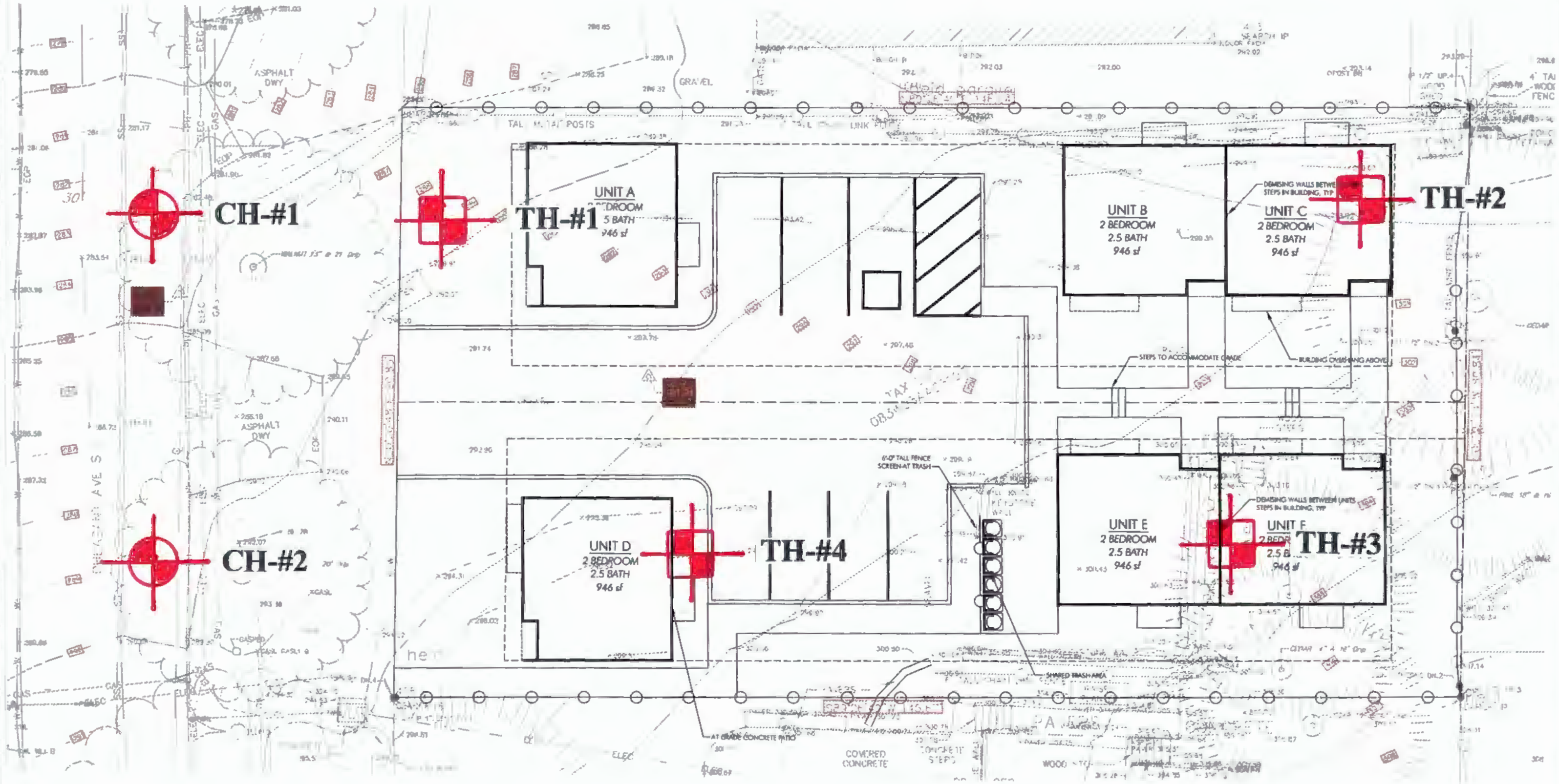
Groundwater was not encountered within any of the exploratory test pit explorations (TH-#1 through TH-#4) at the time of excavation to a depth of at least eight (8) feet beneath existing surface grades except. However, the subject property is underlain by residual soils composed of sandy, clayey silt and at depth by highly to moderately weathered Basalt bedrock deposits. In this regard, although groundwater elevations at the site may fluctuate seasonally in accordance with rainfall conditions as well as changes in site utilization, we are generally of the opinion that the static water levels and/or surface water ponding not observed during our recent field exploration work generally reflect the seasonal groundwater level(s) at and/or beneath the site.

Project No. 2054.001.G

SITE EXPLORATION PLAN

MIDDLE HOUSING DEVELOPMENT
TL 3100, 3200 PHEASANT AVENUE SE

Figure No. 2



LEGEND

CH-#2 Indicates approximate location of exploratory test hole

TH-#4 Indicates approximate location of exploratory test hole

N

Approximate Scale: 1" = 25'

INFILTRATION TESTING

We performed one (1) field infiltration test at the site on March 7, 2024. The infiltration test wasa performed in test hole TH-#1 at a depth of about four (4) feet beneath the existing site and/or surface grades (see Field Infiltration Test Results, Figure No. A-12). The subgrade soils encountered in the infiltration test hole consisted of sandy, clayey silt to highly weathered bedrock deposits.

The infiltration testing was performed in general conformance with current EPA and/or the City of Salem Encased Falling Head test method which consisted of advancing a 6-inch diameter PVC pipe approximately 6 inches into the exposed soil horizon at each test location. Using a steady water flow, water was discharged into the pipe and allowed to penetrate and saturate the subgrade soils. The water level was adjusted over a two (2) hour period and allowed to achieve a saturated subgrade soil condition consistent with the bottom elevation of the surrounding test pit excavation. Following the required saturating period, water was again added into the PVC pipe and the time and/or rate at which the water level dropped was monitored and recorded. Each measurable drop in the water level was recorded until a consistent infiltration rate was observed and/or repeated.

Based on the results of the field infiltration testing at the site, we have found that the native sandy, clayey silt residual subgrade soils and/or highly weathered bedrock deposits possess an ultimate infiltration rate on the order of about 0.1 to 0.2 inches per hour (in/hr).

LABORATORY TESTING

Representative samples of the on-site subsurface soils were collected at selected depths and intervals from the test pit excavation and returned to our laboratory for further examination and testing and/or to aid in the classification of the subsurface soils as well as to help evaluate and identify their engineering strength and compressibility characteristics. The laboratory testing consisted of visual and textural sample inspection, moisture content and dry density determinations, maximum dry density and optimum moisture content, Atterberg Limits and gradational analyses as well as direct shear strength and "R"-value tests. Results of the various laboratory tests are presented in the Appendix, Figure No's. A-7 through A-11.

SEISMICITY AND EARTHQUAKE SOURCES

The seismicity of the southwest Washington and northwest Oregon area, and hence the potential for ground shaking, is controlled by three separate fault mechanisms. These include the Cascadia Subduction Zone (CSZ), the mid-depth intraplate zone, and the relatively shallow crustal zone. Descriptions of these potential earthquake sources are presented below.

The CSZ is located offshore and extends from northern California to British Columbia. Within this zone, the oceanic Juan de Fuca Plate is being subducted beneath the continental North American Plate to the east. The interface between these two plates is located at a depth of approximately 15 to 20 kilometers (km).

The seismicity of the CSZ is subject to several uncertainties, including the maximum earthquake magnitude and the recurrence intervals associated with various magnitude earthquakes. Anecdotal evidence of previous CSZ earthquakes has been observed within coastal marshes along the Washington and Oregon coastlines. Sequences of interlayered peat and sands have been interpreted to be the result of large Subduction zone earthquakes occurring at intervals on the order of 300 to 500 years, with the most recent event taking place approximately 300 years ago. A study by Geomatrix (1995) and/or USGS (2008) suggests that the maximum earthquake associated with the CSZ is moment magnitude (Mw) 8 to 9. This is based on an empirical expression relating moment magnitude to the area of fault rupture derived from earthquakes that have occurred within Subduction zones in other parts of the world. An Mw 9 earthquake would involve a rupture of the entire CSZ. As discussed by Geomatrix (1995) this has not occurred in other subduction zones that have exhibited much higher levels of historical seismicity than the CSZ. However, the 2008 USGS report has assigned a probability of 0.67 for a Mw 9 earthquake and a probability of 0.33 for a Mw 8.3 earthquake. For the purpose of this study an earthquake of Mw 9.0 was assumed to occur within the CSZ.

The intraplate zone encompasses the portion of the subducting Juan de Fuca Plate located at a depth of approximately 30 to 50 km below western Washington and western Oregon. Very low levels of seismicity have been observed within the intraplate zone in western Oregon and western Washington. However, much higher levels of seismicity within this zone have been recorded in Washington and California. Several reasons for this seismic quiescence were suggested in the Geomatrix (1995) study and include changes in the direction of Subduction between Oregon, Washington, and British Columbia as well as the effects of volcanic activity along the Cascade Range. Historical activity associated with the intraplate zone includes the 1949 Olympia magnitude 7.1 and the 1965 Puget Sound magnitude 6.5 earthquakes. Based on the data presented within the Geomatrix (1995) report, an earthquake of magnitude 7.25 has been chosen to represent the seismic potential of the intraplate zone.

The third source of seismicity that can result in ground shaking within the Vancouver and southwest Washington area is near-surface crustal earthquakes occurring within the North American Plate. The historical seismicity of crustal earthquakes in this area is higher than the seismicity associated with the CSZ and the intraplate zone. The 1993 Scotts Mills (magnitude 5.6) and Klamath Falls (magnitude 6.0), Oregon earthquakes were crustal earthquakes.

Liquefaction

Seismic induced soil liquefaction is a phenomenon in which lose, granular soils and some silty soils, located below the water table, develop high pore water pressures and lose strength due to ground vibrations induced by earthquakes. Soil liquefaction can result in lateral flow of material into river channels, ground settlements and increased lateral and uplift pressures on underground structures. Buildings supported on soils that have liquefied often settle and tilt and may displace laterally. Soils located above the ground water table cannot liquefy, but granular soils located above the water table may settle during the earthquake shaking.

Our review of the subsurface soil test pit logs from our exploratory field explorations (TH-#1 through TH-#4) and laboratory test results indicate that the site is generally underlain by medium stiff, sandy, clayey silt residual soils and/or medium dense to dense, highly to moderately weathered Basalt bedrock deposits to a depth of at least eight (8) feet beneath existing site grades. Additionally, groundwater was generally not encountered within any of the exploratory test pit excavations (TH-#1 through TH-#4) at the site during our field exploration work to a depth of at least eight (8) feet.

As such, due to the medium stiff and/or cohesive nature of the sandy, clayey silt residual subgrade soils and medium dense to dense nature of the highly to moderately weathered Basalt bedrock deposits beneath the site, it is our opinion that the native sandy, clayey silt residual subgrade soils and highly to moderately weathered Basalt bedrock deposits beneath the subject site have a very low potential for liquefaction during the design earthquake motions previously described.

Landslides

No ancient and/or active landslides were observed or are known to be present on the subject site. Additionally, development of the subject site into the planned new residential development does not appear to present a potential geologic and/or landslide hazard provided that the site grading and development activities conform with the recommendations presented within this report.

Surface Rupture

Although the site is generally located within a region of the country known for seismic activity, no known faults exist on and/or immediately adjacent to the subject site. As such, the risk of surface rupture due to faulting is considered negligible.

Tsunami and Seiche

A tsunami, or seismic sea wave, is produced when a major fault under the ocean floor moves vertically and shifts the water column above it. A seiche is a periodic oscillation of a body of water resulting in changing water levels, sometimes caused by an earthquake. Tsunami and seiche are not considered a potential hazard at this site because the site is not near to the coast and/or there are no adjacent significant bodies of water.

Flooding and Erosion

Stream flooding is a potential hazard that should be considered in lowland areas of Marion County and Salem. The FEMA (Federal Emergency Management Agency) flood maps should be reviewed as part of the design for the proposed new residential structures and the associated site improvements. Elevations of structures on the site should be designed based upon consultants reports, FEMA (Federal Emergency Management Agency), and Marion County requirements for the 100-year flood levels of any nearby creeks, streams and/or drainage basins.

CONCLUSIONS AND RECOMMENDATIONS

General

Based on the results of our field exploration, laboratory testing, and engineering analyses, it is our professional opinion that the site is presently stable and suitable for the proposed new Middle Housing residential development and the associated site improvements provided that the recommendations contained within this report are properly incorporated into the design and construction of the project. Specifically, the subject site is characterized as relatively flat-lying to gently sloping terrain and, as such, have a relatively low to very low susceptibility to landsliding under any natural geologic circumstance. Additionally, in our experience, the underlying residual soils and/or highly weathered bedrock deposits are only slightly susceptible to slope spreading during strong ground motions from earthquakes. As such, the site does not appear to be at significant risk from the forms of slope instability evaluated by GMS-105 or the DOGAMI landslide susceptibility maps. In our professional opinion, development of the site by the construction of a new single-family and/or duplex residential structures should not create new or exacerbate existing geologic hazards.

The primary features of concern at the site are 1) the presence of highly moisture sensitive clayey and silty subgrade soils across the site, 2) the relatively low infiltration rates anticipated within the near surface clayey and silty subgrade soils and 3) the possible presence of old and/or abandoned foundation remnants and/or site approvements.

With regard to the moisture sensitive clayey and silty subgrade soils, we are generally of the opinion that all site grading and earthwork activities be scheduled for the drier summer months which is typically June through September. In regard to the relatively low infiltration rates anticipated within the clayey and silty subgrade soils beneath the site, we generally do not recommend any storm water infiltration within about ten (10) feet from any existing moderately steep descending slope(s). However, some limited storm water infiltration may be feasible within the relatively flat-lying to gently sloping portions of the site where the existing and/or finish slope gradients are no steeper than about 10 percent. In this regard, we recommend that all proposed storm water detention and/or infiltration systems for the project be reviewed and approved by Redmond Geotechnical Services, LLC. With regard to the possible presence of old and/or abandoned foundation remnants and/or site improvements, stripping and clearing depths greater than 12 inches should be anticipated.

The following sections of this report provide specific recommendations regarding subgrade preparation and grading as well as foundation and floor slab design and construction for the new Middle Housing residential development project.

Site Preparation

As an initial step in site preparation, we recommend that the proposed new Middle Housing residential development site as well as the associated structural and/or site improvement area(s) be stripped and cleared of all existing improvements, any existing unsuitable fill materials, surface debris, existing vegetation, topsoil materials, and/or any other deleterious materials present at the time of construction. In general, we envision that the site stripping to remove existing vegetation and topsoil materials will generally be about 10 to 14 inches. However, localized areas requiring deeper removals, such as any existing undocumented and/or unsuitable fill materials as well as old and/or abandoned foundation remnants and site improvements, will likely be encountered and should be evaluated at the time of construction by the Geotechnical Engineer. The stripped and cleared materials should be properly disposed of as they are generally considered unsuitable for use/reuse as fill materials.

Following the completion of the site stripping and clearing work and prior to the placement of any required structural fill materials and/or structural improvements, the exposed subgrade soils within the planned structural improvement area(s) should be inspected and approved by the Geotechnical Engineer and possibly proof-rolled with a half and/or fully loaded dump truck. Areas found to be soft or otherwise unsuitable should be over-excavated and removed or scarified and recompacted as structural fill. During wet and/or inclement weather conditions, proof rolling and/or scarification and recompaction as noted above may not be appropriate.

The on-site native sandy, clayey silt subgrade soil materials are generally considered suitable for use/reuse as structural fill materials provided that they are free of organic materials, debris, and rock fragments in excess of about 6 inches in dimension. However, if site grading is performed during wet or inclement weather conditions, the use of some of the on-site native soil materials which contain significant silt and clay sized particles will be difficult at best. In this regard, during wet or inclement weather conditions, we recommend that an import structural fill material be utilized which should consist of a free-draining (clean) granular fill (sand & gravel) containing no more than about 5 percent fines. Representative samples of the materials which are to be used as structural fill materials should be submitted to the Geotechnical Engineer and/or laboratory for approval and determination of the maximum dry density and optimum moisture content for compaction.

In general, all site earthwork and grading activities should be scheduled for the drier summer months (late June through September) if possible. However, if wet weather site preparation and grading is required, it is generally recommended that the stripping of topsoil materials be accomplished with a tracked excavator utilizing a large smooth-toothed bucket working from areas yet to be excavated. Additionally, the loading of strippings into trucks and/or protection of moisture sensitive subgrade soils will also be required during wet weather grading and construction. In this regard, we recommend that areas in which construction equipment will be traveling be protected by covering the exposed subgrade soils with a woven geotextile fabric such as Mirafi FW404 followed by at least 12 inches or more of crushed aggregate base rock.

Further, the geotextile fabric should have a minimum Mullen burst strength of at least 250 pounds per square inch for puncture resistance and an apparent opening size (AOS) between the U.S. Standard No. 70 and No. 100 sieves.

All structural fill materials placed within the new residential structures and/or pavement areas and approved by the Geotechnical Engineer should be moistened or dried as necessary to near (within 3 percent) optimum moisture conditions and compacted by mechanical means to a minimum of 92 percent of the maximum dry density as determined by the ASTM D-1557 (AASHTO T-180) test procedures. Structural fill materials should be placed in lifts (layers) such that when compacted do not exceed about 8 inches. Additionally, all fill materials placed within three (3) lineal feet of the perimeter (limits) of the proposed residential structures and/or pavements should be considered structural fill. All aspects of the site grading, including a review of the proposed site grading plan(s), should be approved and/or monitored by a representative of Redmond Geotechnical Services, LLC.

Foundation Support

Based on the results of our investigation, it is our opinion that the site of the proposed new Middle Housing residential project is suitable for support of the single- and/or two-story residential structures provided that the following foundation design recommendations are followed. The following sections of this report present specific foundation design and construction recommendations for the planned new residential structures.

Shallow Foundations

In general, conventional shallow continuous (strip) footings and individual (spread) column footings may be supported by approved native (untreated) subgrade soil materials and/or silty sand structural fill soils based on an allowable contact bearing pressure of about 2,000 pounds per square foot (psf). This recommended allowable contact bearing pressure is intended for dead loads and sustained live loads and may be increased by one-third for the total of all loads including short-term wind or seismic loads. In general, continuous strip footings should have a minimum width of at least 16 inches and be embedded at least 18 inches below the lowest adjacent finish grade (includes frost protection). Individual column footings (where required) should be embedded at least 18 inches below grade and have a minimum width of at least 24 inches. Additionally, if foundation excavation and construction work is planned to be performed during wet and/or inclement weather conditions, we recommend that a 2- to 4-inch layer of compacted crushed rock be used to help protect the exposed foundation bearing surfaces until the placement of concrete.

Total and differential settlements of foundations constructed as recommended above and supported by approved native subgrade soils or by properly compacted structural fill materials are expected to be well within the tolerable limits for this type of lightly loaded two-story structure and should generally be less than about 1-inch and 1/2-inch, respectively.

Allowable lateral frictional resistance between the base of the footing element and the supporting subgrade bearing soil can be expressed as the applied vertical load multiplied by a coefficient of friction of 0.30 and 0.45 for native silty subgrade soils and/or import gravel fill materials, respectively. In addition, lateral loads may be resisted by passive earth pressures on footings poured "neat" against in-situ (native) subgrade soils or properly backfilled with structural fill materials based on an equivalent fluid density of 300 pounds per cubic foot (pcf). This recommended value includes a factor of safety of approximately 1.5 which is appropriate due to the amount of movement required to develop full passive resistance.

Floor Slab Support

In order to provide uniform subgrade reaction beneath concrete slab-on-grade floors, we recommend that the floor slab area be underlain by a minimum of 6 inches of free-draining (less than 5 percent passing the No. 200 sieve), well-graded, crushed rock. The crushed rock should help provide a capillary break to prevent migration of moisture through the slab. However, additional moisture protection can be provided by using a 10-mil polyolefin geo-membrane sheet such as StegoWrap.

The base course materials should be compacted to at least 95 percent of the maximum dry density as determined by the ASTM D-1557 (AASHTO T-180) test procedures. Where floor slab subgrade materials are undisturbed, firm and stable and where the underslab aggregate base rock section has been prepared and compacted as recommended above, we recommend that a modulus of subgrade reaction of 150 pci be used for design.

Retaining/Below Grade Walls

Retaining and/or below grade walls should be designed to resist lateral earth pressures imposed by native soils or granular backfill materials as well as any adjacent surcharge loads. For walls which are unrestrained at the top and free to rotate about their base, we recommend that active earth pressures be computed on the basis of the following equivalent fluid densities:

Non-Restrained Retaining Wall Pressure Design Recommendations

Slope Backfill (Horizontal/Vertical)	Equivalent Fluid Density/Silt (pcf)	Equivalent Fluid Density/Gravel (pcf)
Level	35	30
3H:1V	60	50
2H:1V	90	80

For walls which are fully restrained at the top and prevented from rotation about their base, we recommend that at-rest earth pressures be computed on the basis of the following equivalent fluid densities:

Restrained Retaining Wall Pressure Design Recommendations

Slope Backfill (Horizontal/Vertical)	Equivalent Fluid Density/Silt (pcf)	Equivalent Fluid Density/Gravel (pcf)
Level	55	50
3H:1V	75	70
2H:1V	95	90

The above recommended values assume that the walls will be adequately drained to prevent the buildup of hydrostatic pressures. Where wall drainage will not be present and/or if adjacent surcharge loading is present, the above recommended values will be significantly higher. For seismic loading, we recommend an additional uniform pressure of about 6H where H is the height of the wall in feet.

Backfill materials behind walls should be compacted to 90 percent of the maximum dry density as determined by the ASTM D-1557 (AASHTO T-180) test procedures. Special care should be taken to avoid over-compaction near the walls which could result in higher lateral earth pressures than those indicated herein. In areas within three (3) to five (5) feet behind walls, we recommend the use of hand-operated compaction equipment.

Pavements

Flexible pavement design for the proposed new street improvements along the east side of Pheasant Avenue SE as well as the on-site private access drivers and parking areas associated with the proposed Middle Housing residential development project was determined in accordance with the City of Salem Department of Public Works Administrative Rules Chapter 109-006 (Street Design Standards) Section 6 dated January 1, 2014 as well as the American Association of State Highway and Transportation Officials (AASHTO) 1993 "Design of Pavement Structures" manual.

Specifically, on March 7, 2024, samples of the subgrade soils from the easterly northbound lane of Pheasant Avenue SE were collected by means of two (2) core holes. The subgrade soils encountered in the core holes located along the easterly northbound lane of the existing pavement grade of Pheasant Avenue SE generally consisted of native and/or residual soils comprised of medium to reddish-brown, medium stiff, sandy, clayey SILT (ML).

The subgrade soil samples collected at the site were tested in the laboratory in accordance with the ASTM Vol. 4.08 Part D-2844-69 (AASHTO T-190-93) test method for the determination of the subgrade soil "R"-value and expansion pressure. The results of the "R"-value testing was then converted to an equivalent Resilient Modulus (M_{RSG}) in accordance with current AASHTO methodology. The results of the laboratory "R"-value tests revealed that the subgrade soils have an apparent "R"-value of 26 (see Figure No. A-13). Using the current AASHTO methodology for converting "R"-value to Resilient Modulus (M_{RSG}), the subgrade soils have a Resilient Modulus (M_{RSG}) of about 5,291 psi which is classified a "Fair" (M_{RSG} = 5,000 psi to 10,000 psi).

In addition to the above, Dynamic Cone Penetration (DCP) tests were performed in the bottom of each of the core holes. The results of the DCP tests found that the underlying native sandy, clayey silt subgrade soils have a DCP value of about 3 blows per 2-inches which correlates to a California Bearing Ratio (CBR) of 12. Using current AASHTO methodology for converting CBR to Resilient Modulus (M_{RSG}), the subgrade soils have a Resilient Modulus (M_{RSG}) of 10,637 psi which is classified as "Good" (M_{RSG} = 10,000 psi to 15,000 psi).

Local Residential Streets

The following documents and/or design input parameters were used to help determine the flexible pavement section design for improvements to Pheasant Avenue SE:

- **Street Classification:** Local Residential Street
- **Design Life:** 25 years
- **Serviceability:** 4.2 initial, 2.5 terminal
- **Traffic Loading Data:** 100,000 18-kip EAL's
- **Reliability Level:** 90%
- **Drainage Coefficient:** 1.0 (asphalt), 0.8 (aggregate)
- **Asphalt Structural Coefficient:** 0.41
- **Aggregate Structural Coefficient:** 0.10

Based on the above design input parameters and using the design procedures contained within the AASHTO 1993 Design of Pavement Structures Manual, a Structural Number (SN) of 3.0 was determined.

In this regard, we recommend the following flexible pavement section for the new improvements to new and/or existing Local Residential Streets:

Material Type	Pavement Section (inches)
Asphaltic Concrete	4.0
Aggregate Base Rock	14.0

Private Parking and Drive Areas

Flexible pavement design for the private parking and drive areas associated with the project was determined on the basis of projected (anticipated) traffic volume and loading conditions relative to laboratory subgrade soil strength ("R"-value) characteristics. Based on a laboratory subgrade "R"-value of 26 (Resilient Modulus = 5,000 to 10,000) and utilizing the Asphalt Institute Flexible Pavement Design Procedures and/or the American Association of State Highway and Transportation Officials (AASHTO) 1993 "Design of Pavement Structures" manual, we recommend that the asphaltic concrete pavement section(s) for the new residential development private parting and drive areas at the site consist of the following:

	Asphaltic Concrete Thickness (inches)	Crushed Base Rock Thickness (inches)
Automobile Parking Areas	3.0	8.0
Automobile Drive Areas	3.0	10.0

Note: Where heavy vehicle traffic is anticipated such as those required for fire and/or garbage trucks, we recommend that the automobile drive area pavement section be increased by adding 1.0 inches of asphaltic concrete and 2.0 inches of aggregate base rock. Additionally, the above recommended flexible pavement section(s) assumes a design life of 20 years.

Pavement Subgrade, Base Course & Asphalt Materials

The above recommended pavement section(s) were based on the design assumptions listed herein and on the assumption that construction of the pavement section(s) will be completed during an extended period of reasonably dry weather. All thicknesses given are intended to be the minimum acceptable. Increased base rock sections and the use of a woven geotextile fabric may be required during wet and/or inclement weather conditions and/or in order to adequately support construction traffic and protect the subgrade during construction. Additionally, the above recommended pavement section(s) assume that the subgrade will be prepared as recommended herein, that the exposed subgrade soils will be properly protected from rain and construction traffic, and that the subgrade is firm and unyielding at the time of paving. Further, it assumes that the subgrade is graded to prevent any ponding of water which may tend to accumulate in the base course.

Pavement base course materials should consist of well-graded 1-1/2 inch and/or 3/4-inch minus crushed base rock having less than 5 percent fine materials passing the No. 200 sieve. The base course and asphaltic concrete materials should conform to the requirements set forth in the latest edition of the Oregon Department of Transportation, Standard Specifications for Highway Construction. The base course materials should be compacted to at least 95 percent of the maximum dry density as determined by the ASTM D-1557 (AASHTO T-180) test procedures. The asphaltic concrete paving materials should be compacted to at least 92 percent of the theoretical maximum density as determined by the ASTM D-2041 (Rice Gravity) test method.

Wet Weather Grading and Soft Spot Mitigation

Construction of the proposed new public street improvements is generally recommended during dry weather. However, during wet weather grading and construction, excavation to subgrade can proceed during periods of light to moderate rainfall provided that the subgrade remains covered with aggregate. A total aggregate thickness of 8-inches may be necessary to protect the subgrade soils from heavy construction traffic. Construction traffic should not be allowed directly on the exposed subgrade but only atop a sufficient compacted base rock thickness to help mitigate subgrade pumping. If the subgrade becomes wet and pumps, no construction traffic shall be allowed on the road alignment. Positive site drainage away from the street shall be maintained if site paving will not occur before the on-set of the wet season.

Depending on the timing for the project, any soft subgrade found during proof-rolling or by visual observations can either be removed and replaced with properly dried and compacted fill soils or removed and replaced with compacted crushed aggregate. However, and where approved by the Geotechnical Engineer, the soft area may be covered with a bi-axial geogrid and covered with compacted crushed aggregate.

Soil Shrink-Swell and Frost Heave

The results of the laboratory "R"-value tests indicate that the native subgrade soils possess a low to moderate expansion potential. As such, the exposed subgrade soils should not be allowed to completely dry and should be moistened to near optimum moisture content (plus or minus 3 percent) at the time of the placement of the crushed aggregate base rock materials. Additionally, exposure of the subgrade soils to freezing weather may result in frost heave and softening of the subgrade. As such, all subgrade soils exposed to freezing weather should be evaluated and approved by the Geotechnical Engineer prior to the placement of the crushed aggregate base rock materials.

Excavation/Slopes

Temporary excavations of up to about four (4) feet in depth may be constructed with near vertical inclinations. Temporary excavations greater than about four (4) feet but less than eight (8) feet should be excavated with inclinations of at least 1 to 1 (horizontal to vertical) or properly braced/shored. Where excavations are planned to exceed about eight (8) feet, this office should be consulted. All shoring systems and/or temporary excavation bracing for the project should be the responsibility of the excavation contractor. Permanent slopes should be constructed no steeper than about 2H to 1V unless approved by the Geotechnical Engineer.

Depending on the time of year in which trench excavations occur, trench dewatering may be required in order to maintain dry working conditions if the invert elevations of the proposed utilities are located at and/or below the groundwater level. If groundwater is encountered during utility excavation work, we recommend placing trench stabilization materials along the base of the excavation. Trench stabilization materials should consist of 1-foot of well-graded gravel, crushed gravel, or crushed rock with a maximum particle size of 4 inches and less than 5 percent fines passing the No. 200 sieve. The material should be free of organic matter and other deleterious material and placed in a single lift and compacted until well keyed.

Surface Drainage/Groundwater

We recommend that positive measures be taken to properly finish grade the site so that drainage waters from the shop building structure and landscaping areas as well as adjacent properties or buildings are directed away from the new residential structure's foundations and/or floor slabs. All roof drainage should be directed into conduits that carry runoff water away from the residential structure(s) to a suitable outfall. Roof downspouts should not be connected to foundation drains. A minimum ground slope of about 2 percent is generally recommended in unpaved areas around the proposed new residential structure(s).

Groundwater was not encountered at the site in any of the exploratory test pits (TH-#1 through TH-#4) at the time of excavation to a depth of at least eight (8) feet beneath existing site grades. However, groundwater elevations in the area and/or across the subject property may fluctuate seasonally and may temporarily pond/perch near the ground surface during periods of prolonged rainfall.

As such, based on our current understanding of the possible site grading required to bring the subject site to finish design grade(s), we are of the opinion that an underslab drainage system is not required for the proposed residential structure(s). However, a perimeter foundation drain is recommended for any perimeter footings and/or below grade retaining walls (see Figure No. 3). Additionally, due to the relatively low infiltration rates of the near surface sandy, clayey silt subgrade soils, we are generally of the opinion that storm water detention and/or disposal systems should not be utilized within ten (10) feet of any existing moderately steep descending slope unless approved by the Geotechnical Engineer.

Design Infiltration Rates

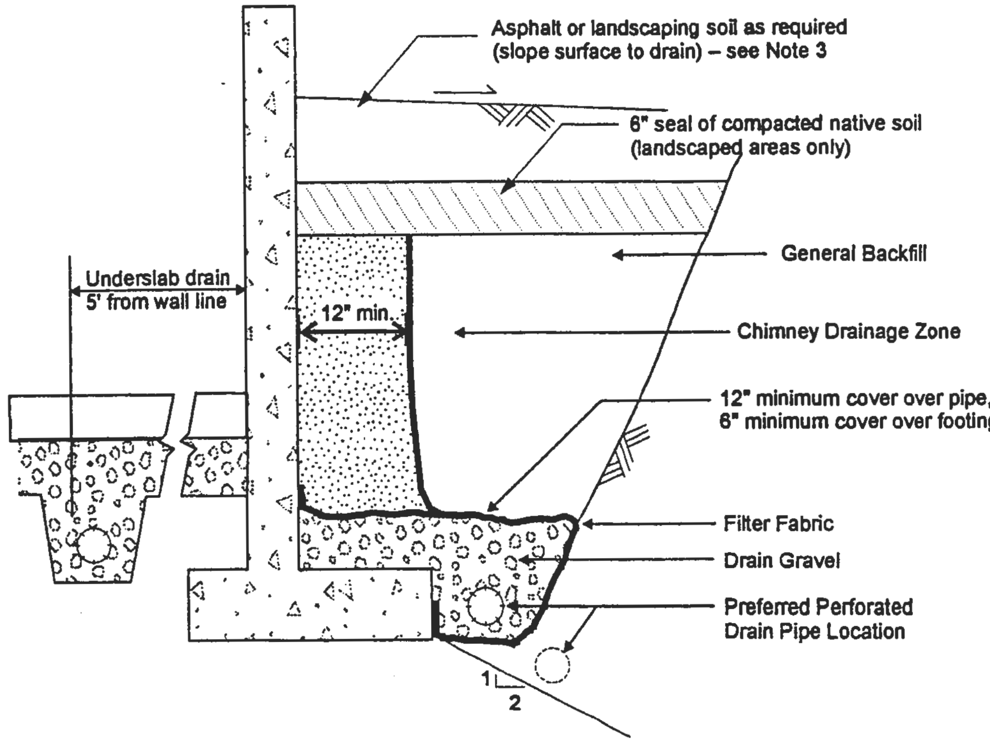
Based on the results of our field infiltration testing, we recommend using the following infiltration rate(s) to design any on-site near surface storm water infiltration systems for the project:

Subgrade Soil Type	Recommended Infiltration Rate
clayey, sandy SILT (ML)	less than 0.1 inches per hour (in/hr)

Note: A safety factor of two (2) was used to calculate the above recommended design infiltration rate. Additionally, given the gradational variability of the on-site sandy, clayey sit subgrade soils beneath the site as well as the anticipation of some site grading for the project, it is generally recommended that field testing be performed during and/or following construction of any on-site storm water infiltration system(s) in order to confirm that the above recommended design infiltration rates are appropriate.

Seismic Design Considerations

Structures at the site should be designed to resist earthquake loading in accordance with the methodology described in the 2019 and/or latest edition of the State of Oregon Structural Specialty Code (OSSC), ASCE 7-16 and/or Amendments to the 2018 International Building Code (IBC). The maximum considered earthquake ground motion for short period and 1.0 period spectral response may be determined from the Oregon Structural Specialty Code, ASCE 7-16 and/or from the 2015 National Earthquake Hazard Reduction Program (NEHRP) "Recommended Provisions for Seismic Regulations for New Buildings and Other Structures" published by the Building Seismic Safety Council. We recommend Site Class "C" be used for design. Using this information, the structural engineer can select the appropriate site coefficient values (F_a and F_v) from ASCE 7-16 or the 2018 IBC to determine the maximum considered earthquake spectral response acceleration for the project. However, we have assumed the following response spectrum for the project:



SCHEMATIC - NOT TO SCALE

NOTES:

1. Filter Fabric to be non-woven geotextile (Amoco 4545, Mirafi 140N, or equivalent)
2. Lay perforated drain pipe on minimum 0.5% gradient, widening excavation as required. Maintain pipe above 2:1 slope, as shown.
3. All-granular backfill is recommended for support of slabs, pavements, etc. (see text for structural fill).
4. Drain gravel to be clean, washed ¾" to 1½" gravel.
5. General backfill to be on-site gravels, or ¾""-0 or 1½"-0 crushed rock compacted to 92% Modified Proctor (AASHTO T-180).
6. Chimney drainage zone to be 12" wide (minimum) zone of clean washed, medium to coarse sand or drain gravel if protected with filter fabric. Alternatively, prefabricated drainage structures (Miradrain 6000 or similar) may be used.

TYPICAL PERIMETER FOOTING/RETAINING WALL DRAIN DETAIL

Project No. 2054.001.G	MIDDLE HOUSING DEVELOPMENT TL 3100, 3200 PHEASANT AVENUE SE	Figure No. 3

Table 1. ASCE 7-16 Seismic Design Parameters

Site Class	S_S	S_1	F_a	F_v	S_{MS}	S_{M1}	S_{DS}	S_{D1}
C	0.822	0.414	1.200	1.500	0.986	0.621	0.657	0.414

Notes: 1. S_S and S_1 were established based on the USGS 2015 mapped maximum considered earthquake spectral acceleration maps for 2% probability of exceedence in 50 years.

2. Fa and Fv were established based ASCE 7-16 using the selected S_S and S_1 values.

CONSTRUCTION MONITORING AND TESTING

We recommend that **Redmond Geotechnical Services, LLC** be retained to provide construction monitoring and testing services during all earthwork operations for the proposed new Middle Housing project. The purpose of our monitoring services would be to confirm that the site conditions reported herein are as anticipated, provide field recommendations as required based on the actual conditions encountered, document the activities of the grading contractor and assess his/her compliance with the project specifications and recommendations.

It is important that our representative meet with the contractor prior to any site grading to help establish a plan that will minimize costly over-excavation and site preparation work. Of primary importance will be observations made during site preparation and stripping, structural fill placement, footing excavations and construction as well as retaining wall backfill.

CLOSURE AND LIMITATIONS

This report is intended for the exclusive use of the addressee and/or their representative(s) to use to design and construct the proposed new Middle Housing residential structures and the associated site improvements described herein as well as to prepare any related construction documents. The conclusions and recommendations contained in this report are based on site conditions as they presently exist and assume that the explorations are representative of the subsurface conditions between the explorations and/or at other locations across the study area. The data, analyses, and recommendations herein may not be appropriate for other structures and/or purposes. We recommend that parties contemplating other structures and/or purposes contact our office. In the absence of our written approval, we make no representation and assume no responsibility to other parties regarding this report. Additionally, the above recommendations are contingent on Redmond Geotechnical Services, LLC being retained to provide all site inspections and constriction monitoring services for this project. Redmond Geotechnical Services, LLC will not assume any responsibility and/or liability for any engineering judgment, inspection and/or testing services performed by others.

It is the owners/developer's responsibility for ensuring that the project designers and/or contractors involved with this project implement our recommendations into the final design plans, specifications and/or construction activities for the project. Further, in order to avoid delays during construction, we recommend that the final design plans and specifications for the project be reviewed by our office to evaluate as to whether our recommendations have been properly interpreted and incorporated into the project.

If during any future site grading and construction, subsurface conditions different from those encountered in the explorations are observed or appear to be present beneath excavations, we should be advised immediately so that we may review these conditions and evaluate whether modifications of the design criteria are required. We also should be advised if significant modifications of the proposed site development are anticipated so that we may review our conclusions and recommendations.

LEVEL OF CARE

The services performed by the Geotechnical Engineer for this project have been conducted with that level of care and skill ordinarily exercised by members of the profession currently practicing in the area under similar budget and time restraints. No warranty or other conditions, either expressed or implied, is made.

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Appendix "A"

Test Pit Logs and Laboratory Test Data

APPENDIX

FIELD EXPLORATIONS AND LABORATORY TESTING

FIELD EXPLORATION

Subsurface conditions at the site were explored by coring two (2) core holes (CH-#1 and CH-#2) and excavating four (4) exploratory test pits (TH-#1 through TH-#4) on March 6 and March 7, 2024, respectively. The approximate location of the core hole and test pit explorations is shown in relation to the existing site improvements on the Site Exploration Plan, Figure No. 2.

The core holes were drilling with portable coring equipment while the test pits were excavated using tracked excavating equipment in general conformance with ASTM Methods in Vol. 4.08, D-1586-94 and D-1587-83. The test pits were excavated to depths of between seven (7) and eight (8) feet beneath existing site grades. Detailed logs of the core holes and test pits are presented on the Log of Test Pits and Core Holes, Figure No.'s A-4 through A-7. The soils were classified in accordance with the Unified Soil Classification System (USCS), which is outlined on Figure No. A-3.

The exploration program was coordinated by a field engineer who monitored the excavating and exploration activity, obtained representative samples of the subsurface soils encountered, classified the soils by visual and textural examination, and maintained continuous logs of the subsurface conditions. Disturbed and/or undisturbed samples of the subsurface soils were obtained at appropriate depths and/or intervals and placed in plastic bags and/or with a thin walled ring sample.

Groundwater was not encountered in any of the exploratory test pits (TH-#1 through TH-#4) at the time of excavating to a depth of at least eight (8) feet beneath existing surface grades.

LABORATORY TESTING

Pertinent physical and engineering characteristics of the soils encountered during our subsurface investigation were evaluated by a laboratory testing program to be used as a basis for selection of soil design parameters and for correlation purposes. Selected tests were conducted on representative soil samples. The program consisted of tests to evaluate the existing (in-situ) moisture-density, Atterberg Limits and gradational characteristics as well as direct shear strength tests.

Dry Density and Moisture Content Determinations

Density and moisture content determinations were performed on both disturbed and relatively undisturbed samples from the test pit explorations in general conformance with ASTM Vol. 4.08 Part D-216. The results of these tests were used to calculate existing overburden pressures and to correlate strength and compressibility characteristics of the soils. Test results are shown on the test pit logs at the appropriate sample depths.

Maximum Dry Density

One (1) Maximum Dry Density and Optimum Moisture Content test was performed on a representative sample of the on-site sandy, clayey silt subgrade soils in accordance with ASTM Vol. 4.08 Part D-1557. This test was conducted to help establish various engineering properties for use as structural fill. The test results are presented on Figure No. A-7.

Atterberg Limits

One (1) Liquid Limit (LL) and Plastic Limit (PL) test was performed on a representative sample of the clayey, sandy silt subgrade soils in accordance with ASTM Vol. 4.08 Part D-4318-85. These tests were conducted to facilitate classification of the soils and for correlation purposes. The test results appear on Figure No. A-8.

Gradation Analysis

One (1) Gradation analysis was performed on a representative sample of the clayey, sandy silt subsurface soils in accordance with ASTM Vol. 4.08 Part D-422. The test results were used to classify the soil in accordance with the Unified Soil Classification System (USCS). The test results are shown graphically on Figure No. A-9.

Direct Shear Strength Test

One (1) Direct Shear Strength test was performed on an undisturbed and/or remolded sample of the clayey, sandy silt subgrade soil at a continuous rate of shearing deflection (0.02 inches per minute) in accordance with ASTM Vol. 4.08 Part D-3080-79. The test results were used to determine engineering strength properties and are shown graphically on Figure No. A-10.

"R"-Value Tests

One (1) "R"-value test was performed on a remolded sample of the residual sandy, clayey silt subgrade soils in accordance with ASTM Vol. 4.08 Part D-2844. The test results were used to help evaluate the subgrade soils supporting and performance capabilities when subjected to traffic loading. The test results are shown on Figure No. A-11.

The following figures are attached and complete the Appendix:

Figure No. A-3	Key to Exploratory Test Pit Logs
Figure No's. A-4 through A-6	Log of Test Pits/Core Holes
Figure No. A-7	Maximum Dry Density Test Results
Figure No. A-8	Atterberg Limits Test Results
Figure No. A-9	Gradation Test Results
Figure No. A-10	Direct Shear Strength Test Results
Figure No. A-11	"R"-value Test Results
Figure No. A-12	Field Infiltration Test Results

PRIMARY DIVISIONS			GROUP SYMBOL	SECONDARY DIVISIONS
COARSE GRAINED SOILS — MORE THAN HALF OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVELS — MORE THAN HALF OF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE	CLEAN GRAVELS (LESS THAN 5% FINES)	GW	Well graded gravels, gravel-sand mixtures, little or no fines.
			GP	Poorly graded gravels or gravel-sand mixtures, little or no fines.
		GRAVEL WITH FINES	GM	Silty gravels, gravel-sand-silt mixtures, non-plastic fines.
			GC	Clayey gravels, gravel-sand-clay mixtures, plastic fines.
	SANDS — MORE THAN HALF OF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE	CLEAN SANDS (LESS THAN 5% FINES)	SW	Well graded sands, gravelly sands, little or no fines.
			SP	Poorly graded sands or gravelly sands, little or no fines.
		SANDS WITH FINES	SM	Silty sands, sand-silt mixtures, non-plastic fines.
			SC	Clayey sands, sand-clay mixtures, plastic fines.
FINE GRAINED SOILS — MORE THAN HALF OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS — LIQUID LIMIT IS LESS THAN 50%		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
			OL	Organic silts and organic silty clays of low plasticity.
	SILTS AND CLAYS — LIQUID LIMIT IS GREATER THAN 50%		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
			CH	Inorganic clays of high plasticity, fat clays.
			OH	Organic clays of medium to high plasticity, organic silts.
HIGHLY ORGANIC SOILS			Pt	Peat and other highly organic soils.

DEFINITION OF TERMS

U.S. STANDARD SERIES SIEVE: 200, 40, 10, 4 — CLEAR SQUARE SIEVE OPENINGS: 3/4", 3", 12"

SILTS AND CLAYS	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

GRAIN SIZES

SANDS, GRAVELS AND NON-PLASTIC SILTS	BLOWS/FOOT†
VERY LOOSE	0 - 4
LOOSE	4 - 10
MEDIUM DENSE	10 - 30
DENSE	30 - 50
VERY DENSE	OVER 50

RELATIVE DENSITY

CLAYS AND PLASTIC SILTS	STRENGTH‡	BLOWS/FOOT†
VERY SOFT	0 - 1/4	0 - 2
SOFT	1/4 - 1/2	2 - 4
FIRM	1/2 - 1	4 - 8
STIFF	1 - 2	8 - 16
VERY STIFF	2 - 4	16 - 32
HARD	OVER 4	OVER 32

CONSISTENCY

†Number of blows of 140 pound hammer falling 30 inches to drive a 2 inch O.D. (1-3/8 inch I.D.) split spoon (ASTM D-1586).

‡Unconfined compressive strength in tons/sq. ft. as determined by laboratory testing or approximated by the standard penetration test (ASTM D-1586), pocket penetrometer, torvane, or visual observation.

REDMOND GEOTECHNICAL SERVICES
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KEY TO EXPLORATORY TEST PIT LOGS
Unified Soil Classification System (ASTM D-2487)

MIDDLE HOUSING DEVELOPMENT
TAX LOT 3100, 3200 PHEASANT AVENUE SE

PROJECT NO.	DATE	Figure A-3
2054.001.G	4/09/24	

BACKHOE COMPANY: MCG | BUCKET SIZE: 24 inches | DATE: 3/07/24

SOIL DESCRIPTION

TEST PIT NO. TH_#1 ELEVATION 289'±

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION
0					ML	Dark brown, wet, soft, organic, sandy, clayey SILT (Topsoil)
					ML	Medium to reddish-brown, very moist, medium stiff, sandy, clayey SILT (Residual Soil)
5					SM/ RK	Reddish- to orangish-brown, moist to very moist, medium dense to dense, highly to moderately weathered Basalt bedrock
10						Total Depth = 8.0 feet No groundwater encountered at time of exploration
15						

TEST PIT NO. TH-#2 ELEVATION 300'±

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION
0					ML	Dark brown, wet, soft, organic, sandy, clayey SILT (Topsoil)
	X			28.8	ML	Medium to reddish-brown, very moist, medium stiff, sandy, clayey SILT (Residual Soil)
5	X			23.5	SM/ RK	Reddish- to orangish-brown, moist to very moist, medium dense to dense, highly to moderately weathered Basalt bedrock
10						Total Depth = 7.0 feet No groundwater encountered at time of exploration
15						

LOG OF TEST PITS

PROJECT NO. 2054.001.G | MIDDLE HOUSING DEVELOPMENT | FIGURE NO. A-4

BACKHOE COMPANY: MCG | BUCKET SIZE: 24 inches | DATE: 3/07/24

SOIL DESCRIPTION

TEST PIT NO. TH-#3 ELEVATION 303'±

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION
0					ML	Reddish-brown, very moist, medium stiff, sandy, clayey SILT (Residual Soil)
5					SM/ RK	Reddish- to orangish-brown, moist to very moist, medium dense to dense, highly to moderately weathered Basalt bedrock
10						Total Depth = 7.0 feet No groundwater encountered at time of exploration
15						

TEST PIT NO. TH-#4 ELEVATION 297'±

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION
0					ML	Dark brown, wet, soft, organic, sandy, clayey SILT (Topsoil)
	X			29.1	ML	Medium to reddish-brown, very moist, medium stiff, sandy, clayey SILT (Residual Soil)
5	X			22.7	SM/ RK	Reddish- to orangish-brown, moist to very moist, medium dense to dense, highly to moderately weathered Basalt bedrock
10						Total Depth = 7.0 feet No groundwater encountered at time of exploration
15						

LOG OF TEST PITS

PROJECT NO. 2054.001.G | MIDDLE HOUSING DEVELOPMENT | FIGURE NO. A-5

LOG OF CORE HOLES

CH-#1:	2.00 inches of Asphaltic Concrete (AC) over 7.0 inches of Aggregate Base Rock (ABR) over Medium to reddish-brown, sandy, clayey SILT (ML)
CH-#2:	2.25 inches of Asphaltic Concrete (AC) over 6.00 inches of Aggregate Base Rock (ABR) over Medium to reddish-brown, sandy, clayey SILT (ML)

Figure No. A-6

MAXIMUM DENSITY TEST RESULTS

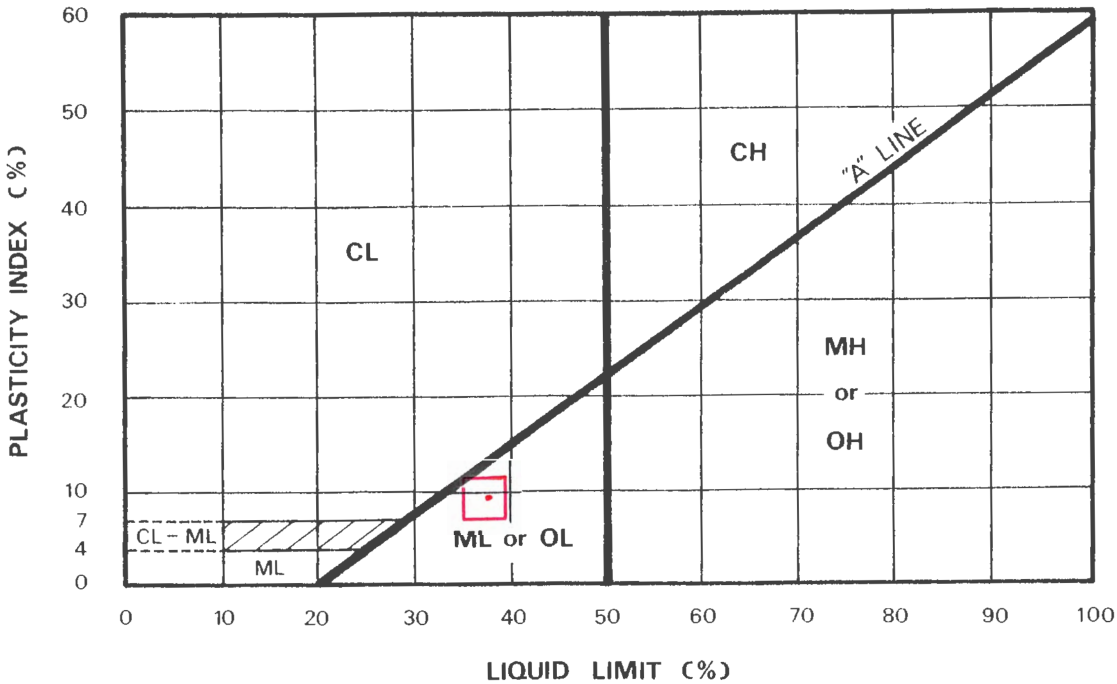
SAMPLE LOCATION	SOIL DESCRIPTION	MAXIMUM DRY DENSITY (pcf)	OPTIMUM MOISTURE CONTENT (%)
TH-#4 @ 2.5'	Medium to reddish-brown, sandy, clayey SILT (ML)	104.0	26.0

EXPANSION INDEX TEST RESULTS

SAMPLE LOCATION	INITIAL MOISTURE (%)	COMPACTED DRY DENSITY (pcf)	FINAL MOISTURE (%)	VOLUMETRIC SWELL (%)	EXPANSION INDEX	EXPANSIVE CLASS.

MAXIMUM DENSITY & EXPANSION INDEX TEST RESULTS

PROJECT NO.: 2054.001.G	MIDDLE HOUSING DEVELOPMENT	FIGURE NO.: A-7



KEY SYMBOL	BORING NO.	SAMPLE DEPTH (feet)	NATURAL WATER CONTENT %	LIQUID LIMIT %	PLASTICITY INDEX %	PASSING NO. 200 SIEVE %	LIQUIDITY INDEX	UNIFIED SOIL CLASSIFICATION SYMBOL
⊡	TH-#2	2.0	28.8	38.2	9.9	83.8		ML

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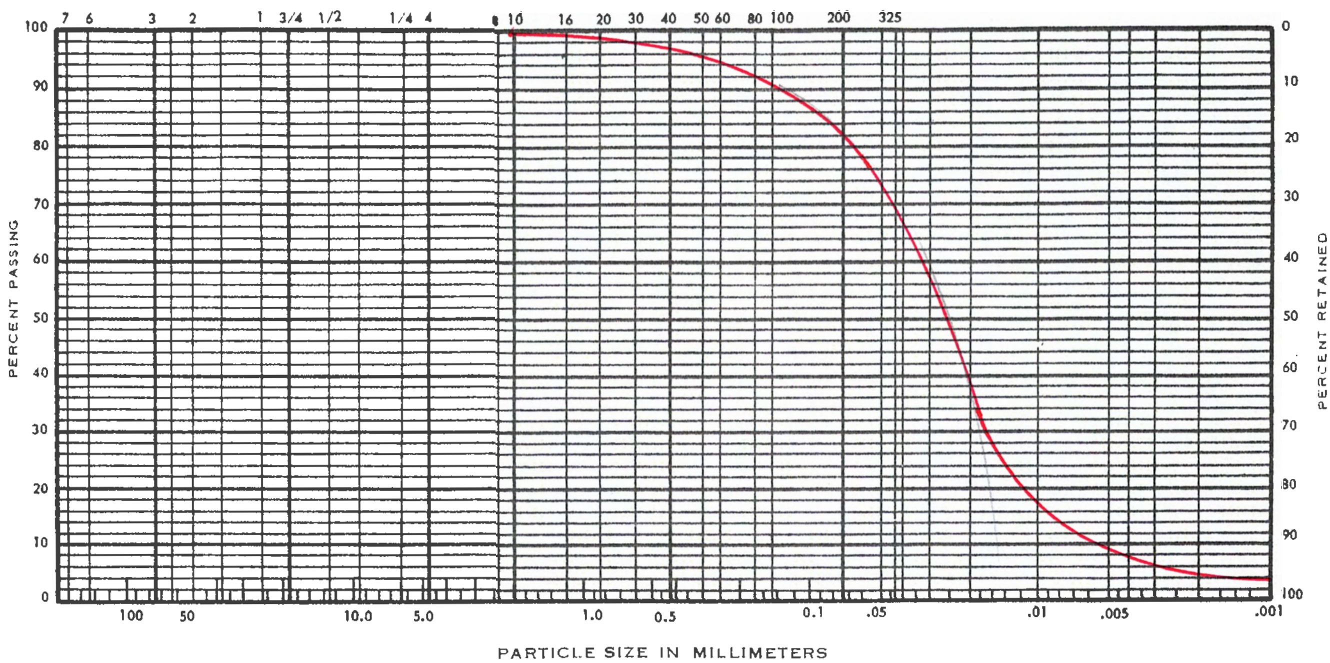
PLASTICITY CHART AND DATA

MIDDLE HOUSING DEVELOPMENT
TAX LOT 3100, 3200 PHEASANT AVENUE SE

PROJECT NO.	DATE	Figure A-8
2054.001.G	4/09/24	

UNIFIED SOIL CLASSIFICATION SYSTEM

(ASTM D 422-72)



COBBLES	GRAVEL		SAND			SILT AND CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

KEY SYMBOL	BORING NO.	SAMPLE DEPTH (feet)	ELEV. (feet)	UNIFIED SOIL CLASSIFICATION SYMBOL	SAMPLE DESCRIPTION
——	TH-#2	2.0		ML	Medium to reddish-brown, sandy, clayey SILT

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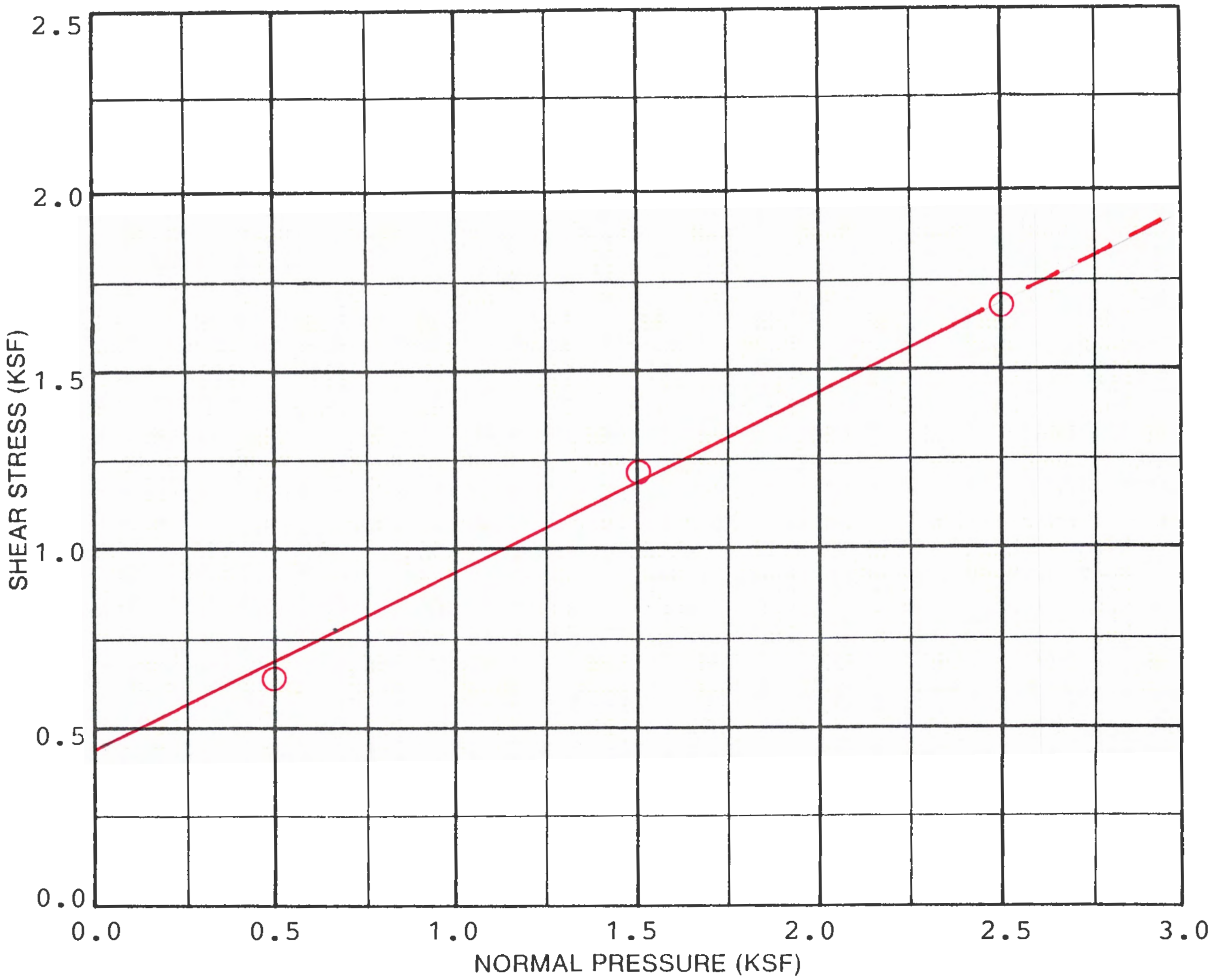
GRADATION TEST DATA		
MIDDLE HOUSING DEVELOPMENT TAX LOT 3100, 3200 PHEASANT AVENUE SE		
PROJECT NO. 2054.001.G	DATE 4/p9/24	FIGURE A-9



SAMPLE DATA	
DESCRIPTION: Medium to reddish-brown, sandy, clayey SILT (ML) (Remolded)	
BORING NO.: TH-#4	
DEPTH (ft.): 2.5	ELEVATION (ft):
TEST RESULTS	
APPARENT COHESION (C): 450 psf	
APPARENT ANGLE OF INTERNAL FRICTION (ϕ): 26°	

TEST DATA				
TEST NUMBER	1	2	3	4
NORMAL PRESSURE (KSF)	0.5	1.5	2.5	
SHEAR STRENGTH (KSF)	0.6	1.2	1.7	
INITIAL H_2O CONTENT (%)	26.0	26.0	26.0	
FINAL H_2O CONTENT (%)	26.8	22.3	17.8	
INITIAL DRY DENSITY (PCF)	92.0	92.0	92.0	
FINAL DRY DENSITY (PCF)	92.8	96.6	103.7	
STRAIN RATE: 0.02 inches per minute				

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DIRECT SHEAR TEST DATA		
MIDDLE HOUSING DEVELOPMENT TAX LOT 3100, 3200 PHEASANT AVENUE SE		
PROJECT NO. 2054.001.G	DATE 4/09/24	Figure A-10

RESULTS OF R (RESISTANCE) VALUE TESTS

SAMPLE LOCATION: TH-#4

SAMPLE DEPTH: 2.5 feet bgs

Specimen	A	B	C
Exudation Pressure (psi)	219	322	431
Expansion Dial (o.ooo1")	0	1	3
Expansion Pressure (psf)	0	3	9
Moisture Content (%)	29.6	25.4	21.1
Dry Density (pcf)	100.7	103.4	106.5
Resistance Value, "R"	15	27	38
"R"-Value at 300 psi Exudation Pressure = 26			

SAMPLE LOCATION:

SAMPLE DEPTH:

Specimen	A	B	C
Exudation Pressure (psi)			
Expansion Dial (0.0001")			
Expansion Pressure (psf)			
Moisture Content (%)			
Dry Density (pcf)			
Resistance Value "R"			
"R"-Value at 300 psi Exudation Pressure =			

Figure No. A-11

Field Infiltration Test Results

Location: TL 3100, 3200 Pheasant Ave SE	**Date:** March 7, 2024	**Test Hole:** TH-#1
Depth to Bottom of Hole: 4.0 feet	**Hole Diameter:** 6 inches	**Test Method:** Encased Falling Head
Tester's Name: Daniel M. Redmond, P.E., G.E. **Tester's Company:** Redmond Geotechnical Services, LLC	**Tester's Contact Number:** 503-285-0598	

Depth (feet)	Soil Characteristics
0.0-1.0	Dark brown, sandy, clayey SILT (TOPSOIL)
1.0-4.0	Medium to reddish-brown, sandy, clayey SILT (ML)

Time	Time Interval (Minutes)	Measurement (inches)	Drop in Water (inches)	Infiltration Rate (inches/hour)	Remarks
10:00	0	36.00	----		Filled w/12" water
10:20	20	36.15	0.15	0.45	
10:40	20	36.26	0.11	0.33	
11:00	20	36.35	0.09	0.27	
11:20	20	36.43	0.08	0.24	
11:40	20	36.51	0.08	0.24	
12:00	20	36.58	0.07	0.21	
12:20	20	36.65	0.07	0.21	
12:40	20	36.72	0.07	0.21	

Infiltration Test Data Table

Figure No. A-12