REDMOND GEOTECHNICAL SERVICES

Geotechnical Investigation and Consultation Services

Proposed Bonaventure of Milwaukie Development Site

Tax Lot No's. 600 and 901

13333 SE Rusk Road

Milwaukie (Clackamas County), Oregon

for

Bonaventure

Project No. 1004.032.G
September 28, 2018

REDMOND GEOTECHNICAL SERVICES

September 28, 2018

Mr. Daniel Dobson
Development Project Manager
Bonaventure
3425 Boone Road SE
Salem, Oregon 97317

Dear Mr. Dobson:

Re: Geotechnical Investigation and Consultation Services, Proposed Bonaventure of Milwaukie Development Site, Tax Lot No's. 600 and 901, 13333 SE Rusk Road, Milwaukie (Clackamas County), Oregon

Submitted herewith is our report entitled "Geotechnical Investigation and Consultation Services, Proposed Bonaventure of Milwaukie Development Site, Tax Lot No's. 600 and 901, 13333 SE Rusk Road, Milwaukie (Clackamas County), Oregon". The scope of our services was outlined in our formal proposal to Mr. Daniel Dobson of Bonaventure dated June 25, 2018. Written authorization of our services was provided by Mr. Daniel Dobson on August 16, 2018.

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. 12-31-18

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MILWAUKIE (CLACKAMAS COUNTY) OREGON

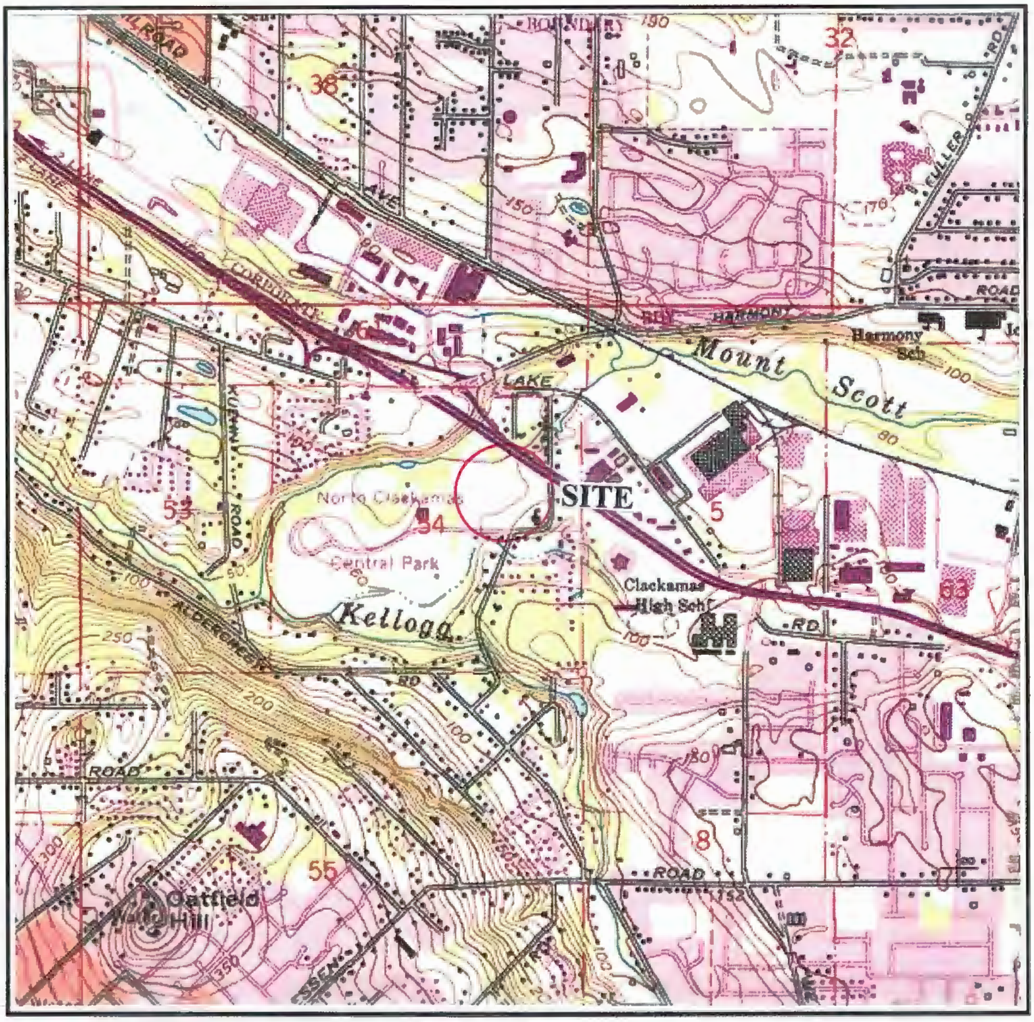
INTRODUCTION

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 Bonaventure of Milwaukie development located to the west of SE Rusk Road and to the north of SE Kellogg Creek Drive in Milwaukie (Clackamas County), Oregon. The general location of the subject site is shown on the Site Vicinity 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 development at the site as well as to develop and/or provide appropriate geotechnical design and construction recommendations for the proposed new Bonaventure of Milwaukie development project.

PROJECT DESCRIPTION

Although the project is still in the preliminary planning stages, we understand that present plans for the project is to develop the subject property into a new senior living and/or care facility. Specifically, we understand that the project will consist of the construction of a new single- and/or four-story senior living building which will be constructed with wood-framing and a concrete slab-on-grade floor system. The new senior care and/or living facility reportedly will total approximately 170,000 square feet and will include a single-story memory care (MC) wing totaling approximately 20 to 30 units, a three- and/or four-story assisted living and memory care (AL/MC) wing totaling approximately 50 to 60 units, and a three- and/or four-story independent living (IL) wing totaling approximately 70 to 80 units. Support of the new senior living and/or care facility structure is anticipated to consist primarily of conventional shallow strip (continuous) footings although some individual (column) footings may also be required. Structural loading information, although unavailable at this time, is anticipated to be fairly typical for this type of single- and/or four-story wood-frame structure and is expected to result in maximum dead plus live continuous (strip) and individual (column) footing loads on the order of about 2.0 to 4.0 kips per lineal foot (klf) and 15 to 125 kips, respectively.

Although a site grading plan is not available at this time, we understand that both cuts and/or fills are presently planned for the project. In general, both cuts and/or fills of less than five (5) feet are generally anticipated across the site.



GLADSTONE QUADRANGLE
OREGON
7.5 MINUTE SERIES (TOPOGRAPHIC)

SCALE 1:24 000

1 ½ 0 1 MILE

1000 0 1000 2000 3000 4000 5000 6000 7000 FEET

1 .5 0 1 KILOMETER

CONTOUR INTERVAL 10 FEET
NATIONAL GEODETIC VERTICAL DATUM OF 1929
DEPTH CURVES AND SOUNDINGS IN FEET—COLUMBIA RIVER DATUM

SITE VICINITY MAP

Project No. 1004.032.G	**BONAVENTURE OF MILWAUKIE** **TL'S 600 & 901/13333 SE RUSK RD**	Figure No. 1

Other associated site improvements for the project will include construction of new paved access drives and parking areas. Additionally, the project will include the construction of new underground utility services and new concrete curbs and sidewalks as well as possible on-site storm water collection and/or disposal systems.

SCOPE OF WORK

The purpose of our geotechnical studies was to evaluate the overall subsurface soil and/or groundwater conditions underlying the subject site with regard to the proposed new senior living and/or care facility development and construction at the site and any associated impacts or concerns with respect to development at the site as well as provide appropriate geotechnical design and construction recommendations for the project. Specifically, our geotechnical investigation 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 including a Geotechnical Engineering Report prepared by GeoPacific Engineering, Inc dated August 8, 2013 and a Geotechnical Evaluation prepared by GEO Consultants Northwest dated October 7, 2016.

2. A detailed field reconnaissance and subsurface exploration program of the soil and ground water conditions underlying the site by means of eleven (11) exploratory test pit excavations. The exploratory test pits were excavated to depths ranging from about five (5) to eight (8) feet beneath existing site grades at the approximate locations as shown on the Site Exploration Plan, Figure No. 2. Additionally, field infiltration testing was also performed within various test pits excavated across 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, maximum dry density and optimum moisture content, gradational characteristics, Atterberg Limits and (remolded) direct shear strength tests as well as consolidation 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 senior living structure. 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 private access drives and parking area improvements.

SITE CONDITIONS

Site Geology

The site is located within the Columbia River/Puget Sound lowland which is a broad structural depression situated between the Coast Range to the west and the Cascade Range to the east. A series of discontinuous faults subdivide the Columbia River basin into a mosaic of fault-bounded, structural blocks (Yeats et al., 1996). Uplifted structural blocks form bedrock highlands while down-warped structural blocks form sedimentary basins.

Available geologic mapping of the area and/or subject site indicates that the near surface soils consist of fine grained alluvial soil deposits (Qff) comprised of crudely to complexly layered, poorly consolidated medium sand to silt deposited by one or more phases of catastrophic glacial outburst floods from late Pleistocene lake Missoula. Sediments of unit Qff occur along both sides of the Willamette and/or Columbia Rivers and throughout the Tualatin basin. The thickness of unit Qff is typically 30 to 60 feet with a maximum thickness of about 180 feet. However, the site is also underlain at relatively shallow depths by more recent alluvial deposits comprised of silty clay as well as silty and sandy gravel associated with the nearby Mount Scott and Kellogg Creek.

Surface Conditions

The subject proposed new Bonaventure of Milwaukie development property consists of two (2) rectangular and/or irregular shaped tax lots (TL's 600 and 901) which encompass a total plan area of approximately 12 acres. The proposed Bonaventure of Milwaukie development property is roughly located to the west of SE Rusk Road and to the north of SE Kellogg Creek Drive. The subject property is presently unimproved and void of existing structures and/or site improvements.

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 across the northerly and westerly portions of the subject property. Additionally, the northerly and/or northwesterly portions of the subject property are generally low lying and contains an existing seasonal drainage basin and/or wetland.

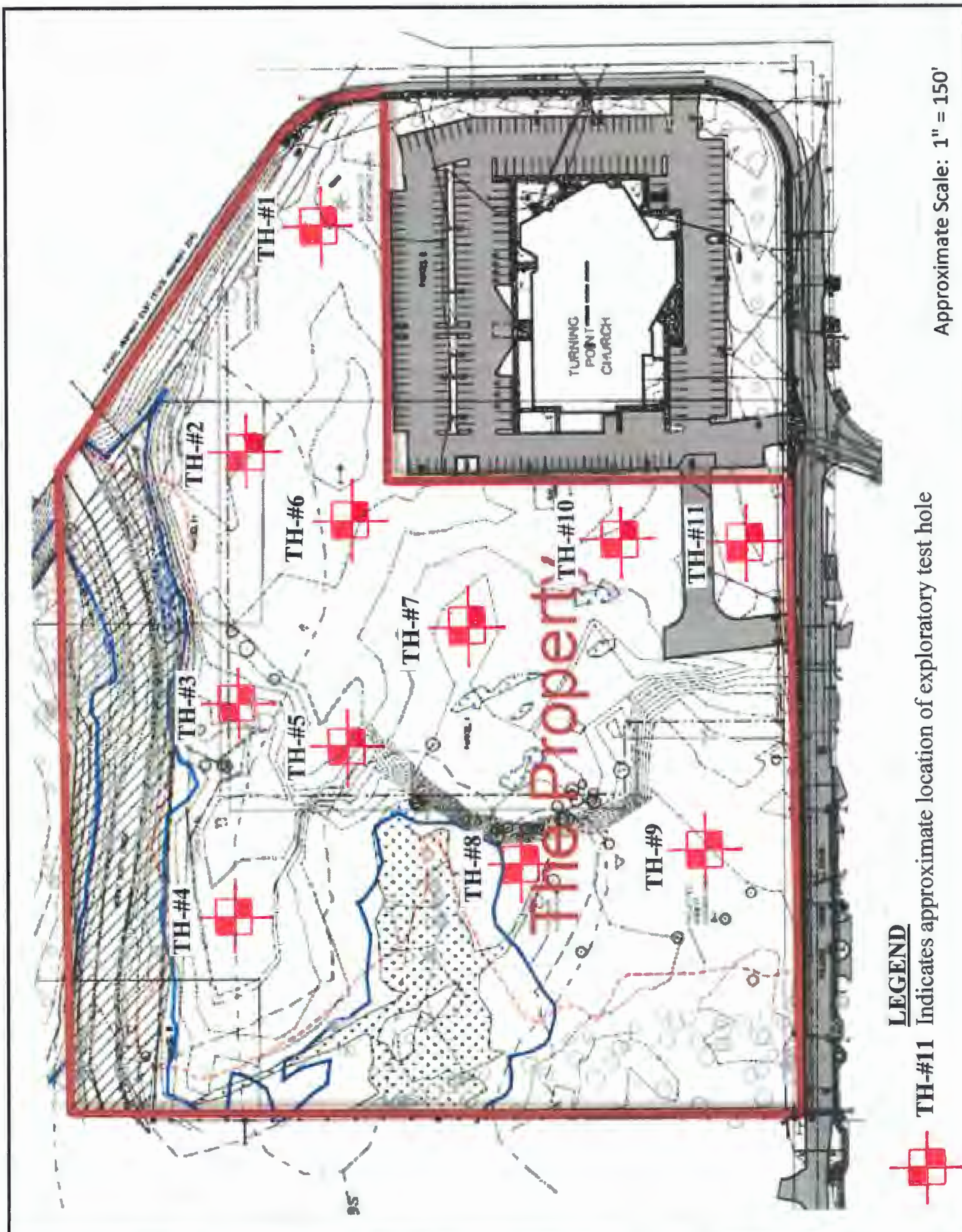
Topographically, the site is characterized as relatively flat-lying to gently to moderately sloping terrain (10 to 20 percent) descending downward towards the north and west with overall topographic relief estimated at about ten (10) to fifteen (15) feet and is estimated to lie at about Elevation 70 feet.

Subsurface Soil Conditions

Our understanding of the subsurface soil conditions underlying the site was developed by means of eleven (11) exploratory test pits excavated to depths ranging from about five (5) to eight (8) feet beneath existing site grades on September 9, 2018 with a John Deere 200C track-mounted excavator. The location of the exploratory 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 features and/or site improvements on the Site Exploration Plan, Figure No. 2. Detailed logs of the test pit explorations, presenting conditions encountered at each location explored, are presented in the Appendix, Figure No's. A-5 through A-10.

The exploratory test pit excavations were observed by staff from Redmond Geotechnical Services, LLC who logged each of the test pit explorations and obtained representative samples of the subsurface soils encountered across the site. Additionally, the elevation of the exploratory test pit excavations were referenced from the USGS Map of the Gladstone Quadrangle and should be considered as approximate. All subsurface soils encountered at the site and/or within the exploratory test pit excavations were logged and classified in general conformance with the Unified Soil Classification System (USCS) which is outlined on Figure No. A-4.

The test pit explorations revealed that the subject site is underlain by both manmade fill soils and native soil deposits. Specifically, the test pit excavations found that much of the subject property contains fill soils consisting of a highly variable mixture of clay, silt, sand and gravel which also contained various amounts of construction debris (i.e., concrete and asphalt rubble) as well as organics and/or deleterious materials. The fill materials, which are believed to be undocumented, were found to be poorly to moderately compacted and ranged in depth from about 1.5 to at least 6.0 feet below the existing ground surface. However, the existing fill depth across the subject property has been reported at about 12 to 13 feet by others. The upper fill materials were fund to be underlain by native soil deposits consisting of an upper layer of old topsoil remnants consisting of approximately 12 to 18 inches of dark brown to dark gray-brown, moist to very moist, slightly to moderately organic, soft to medium stiff, sandy, clayey silt. The old topsoil zone was inturn underlain by other native alluvial soil deposits consisting of an upper unit of medium to gray-brown, moist to very moist, medium stiff to medium dense, clayey, sandy silt to silty sand to depths of approximately 5.0 to 7.0 feet beneath the existing site and/or surface grades. These underlying clayey, sandy silt to silty sand subgrade soils are best characterized by relatively low to moderate strength and moderate compressibility. All soils were found to be underlain at depth by medium to gray-brown, moist to very moist, medium dense to dense, slightly clayey, silty and sandy gravel with cobbles to the maximum depth explored of 8.0 feet beneath the existing site and/or surface grades.



SITE EXPLORATION PLAN

Project No. 1004.032.G	**BONAVENTURE OF MILWAUKIE** **TL'S 600 & 901/13333 SE RUSK RD**	Figure No. 2

These underlying medium dense to dense gravel deposits are best characterized by relatively moderate to high strength and low compressibility.

Groundwater

Groundwater was generally not encountered within any of the exploratory test pit explorations excavated across the site at the time of the excavations to depths of up to 8.0 feet beneath existing surface grades except. However, the test pits were excavated near the end of the dry season. Additionally, the northerly and northwesterly portions of the subject property are bounded by and/or contain an existing seasonal drainage basin and/or wetlands. Further, Mount Scott Creek and Kellogg Creek are located to the north and south of the subject property.

In this regard, groundwater elevations at the site may fluctuate seasonally in accordance with rainfall conditions and/or runoff associated with Mount Scott Creek and Kellogg Creek as well as changes in site utilization. Additionally, according to USGS mapping, the regional groundwater elevation in the vicinity of the subject property is at about Elevation 65 to Elevation 70 feet.

INFILTRATION TESTING

We performed three (3) field infiltration tests at the site on September 9, 2018. The infiltration tests were performed in test holes TH-#1 , TH-#4 and TH-#9 at depths of between four (4) to six (6) feet beneath the existing site and/or surface grades. The subgrade soils encountered in the infiltration test hole consisted of clayey, sandy silt to silty sand. The infiltration testing was performed in general conformance with current EPA and/or the City of Milwaukie/Clackamas County 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 underlying native clayey, sandy silt to silty sand subgrade soil deposits posses an ultimate infiltration rate on the order of about 4 to 6 inches per hour (in/hr).

LABORATORY TESTING

Representative samples of the on-site subsurface soils were collected at selected depths and intervals from various test pit excavations 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, gradation analyses and Atterberg Limits as well as direct shear strength, consolidation and "R"-value tests. Results of the various laboratory tests are presented in the Appendix, Figure No's. A-11 through A-16.

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-#11) and laboratory test results indicate that the site is generally underlain at depth by medium dense to dense, slightly clayey, silty and sandy gravel with cobbles deposits to depths of at least 8.0 feet beneath existing site grades. Additionally, groundwater was generally not encountered within any of the exploratory test pit excavations (TH-#1 through TH-#11) at the site during our field exploration work. As such, due to the medium dense to dense nature of the slightly clayey, silty and sandy gravel with cobbles subgrade soil deposits beneath the site, it is our opinion that the native subgrade soil deposits located 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, the subject property does not contain any steep slopes. As such, development of the subject site into the planned senior living and/or care facility 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. The closest known faults to the subject property are the Oatfield Fault and the Portland Hills Fault which are sited approximately 0.2 miles and 1.5 miles to the southwest of the subject site, respectively, and the East Bank Fault which is sited approximately 3.0 miles to the northeast of 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 Lane County and Eugene. The FEMA (Federal Emergency Management Agency) flood maps should be reviewed as part of the design for the proposed new senior living and/or care facility structure and site improvements. Elevations of structures on the site should be designed based upon consultants reports, FEMA (Federal Emergency Management Agency), and Lane 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 explorations, laboratory testing, and engineering analyses, it is our opinion that the site is generally suitable for the proposed new Bonaventure of Milwaukie senior living and/or care facility development and its associated site improvements provided that the recommendations contained within this report are properly incorporated into the design and construction of the project.

The primary feature of concern at the site is the presence of the existing fill soil materials present across the site.

With regard to the existing fill soil materials present across the site, we understand that the existing fill soils were likely placed prior to 1995 during two (2) or more events and are "undocumented". Additionally, the existing fill materials were found to contain various amounts of construction debris (i.e., asphalt and concrete) as well as some organic matter. Further, the results of our field and laboratory work indicates that the existing fill soil materials are generally only moderately compacted. In addition to the above, the existing fill soil materials were found to be placed directly above the old topsoil zone which is characterized as soft to medium stiff and contains some organics. In this regard, due to the variable nature (composition) and/or depth (thickness) of the existing undocumented fill soil materials across the site, it is our professional opinion that construction of the proposed single- and/or four-story wood-frame structure directly on and/or above the existing undocumented fill soil materials would expose the proposed senior living and/or care facility of potential excessive post-construction settlements.

As such, we are of the opinion that the existing fill soil materials as well as the underlying old topsoil zone subgrade soils be removed in their entirety from beneath the proposed senior living and/or care facility down to an approved native subgrade soil following which the area over-excavated may then be filled with properly placed and compacted structural fill materials back to the required design grades and/or elevations.

Secondary features of concern for the project are 1) the moisture sensitive clayey and silty fill and/or native subgrade soils across and/or beneath the site and 2) the anticipated relatively high seasonal groundwater elevations beneath the subject property.

With regard to the moisture sensitive clayey and silty fill and/or native subgrade soils across and/or beneath the site, 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 regards to the anticipated relatively high seasonal groundwater elevations beneath the subject property, w are again of the opinion that all site grading and earthwork associated with removal of the existing undocumented fill soil materials as well as the placement and compaction of any required structural fill soil be performed during the drier summer months.

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 Bonaventure of Milwaukie senior living and/or care facility development project.

Site Preparation

As an initial step in site preparation, we recommend that the proposed new senior living and/or care facility building as well as its 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 16 to 12 inches. However, localized areas requiring deeper removals, such as the existing undocumented and/or unsuitable fill materials as well as the old topsoil remnants located within the proposed senior living and/or care facility building foot print, 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 clayey, sandy silt and/or silty sand 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 (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 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 building and/or pavement areas 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 five (5) lineal feet of the perimeter (limits) of the proposed senior living and/or care facility structure and/or pavements should be considered structural fill. Additionally, due to the sloping site conditions, we recommend that all structural fill materials planned in areas where existing surface and/or slope gradients exceed about 20 percent (1V:5H) be properly benched and/or keyed into the native (natural) slope subgrade soils. In general, a bench width of at least eight (8) feet and a keyway depth of at least one (1) foot is recommended. However, the actual bench width and keyway depth should be determined at the time of construction by the Geotechnical Engineer. Further, all fill slopes should be constructed with a finish slope surface gradient no steeper than about 2H:1V.

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 Bonaventure of Milwaukie senior living and/or care facility development is generally suitable for support of the planned single- and/or four-story wood-frame structure provided that the following foundation design recommendations are followed. As previously noted, the subject site contains existing undocumented fill soil materials which are only moderately compacted and contain various amounts of construction debris as well as organics and/or other deleterious materials. Additionally, the existing fill soil materials are underlain by the old topsoil zone which are also considered to be moderately compressible. In this regard, in order to prevent the potential for excessive port-construction settlements, we are of the opinion that the proposed new senior living and/or care facility not be supported directly by the existing fill soils materials. As such, it is our professional opinion that all of the existing undocumented fill materials as well as the underlying old topsoil remnants be removed in their entirety from beneath the proposed building area down to an approved native subgrade soil following which the area over-excavated can then be filled to the required design grades and/or elevations with properly placed and compacted structural fill materials.

The following sections of this report present specific foundation design and construction recommendations for the planned new senior living and/or care facility structure.

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 by properly placed and compacted structural fill soils based on an allowable contact bearing pressure of about 2,500 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 single- and/or four- wood-frame 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	45	35
3H:1V	65	60
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.

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 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 an average laboratory subgrade "R"-value of 30 (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 senior living and/or care facility development 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	3.5	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 0.5 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 paved site 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- to 12-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 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 and/or existing fill 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 senior living and/or care facility structure and landscaping areas as well as adjacent properties or buildings are directed away from the new structures foundations and/or floor slabs. All roof drainage should be directed into conduits that carry runoff water away from the senior living and/or care facility structure 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 structure.

Groundwater was not encountered at the site within any of the exploratory test pits excavated across the site at the time of excavation to depths of at least 8.0 feet beneath existing site grades. However, the northerly and/or northwesterly portions of the site are bounded by an existing seasonal drainage basin and/or wetland. Additionally, although 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, the depth to the seasonal high groundwater is approximately Elevation 65 to Elevation 70 feet.

As such, based on our current understand 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 generally not required for the proposed senior living and/or care facility structure. However, a perimeter foundation drain is recommended for any perimeter footings and/or below grade retaining walls. A typical recommended perimeter footing/retaining wall drain detail is shown on Figure No. 3.

Further, due to our anticipation that various surface infiltration ditches and/or swales may be utilized for the project as well as the relatively low infiltration rates of the anticipated new structural fill soil materials within and/or near to the foundation bearing level of the proposed senior living and/or care facility structure, we are generally of the opinion that storm water detention and/or disposal systems should not be utilized around and/or up-gradient of the proposed senior living and/or care facility structure unless approved by the Geotechnical Engineer.

Design Infiltration Rates

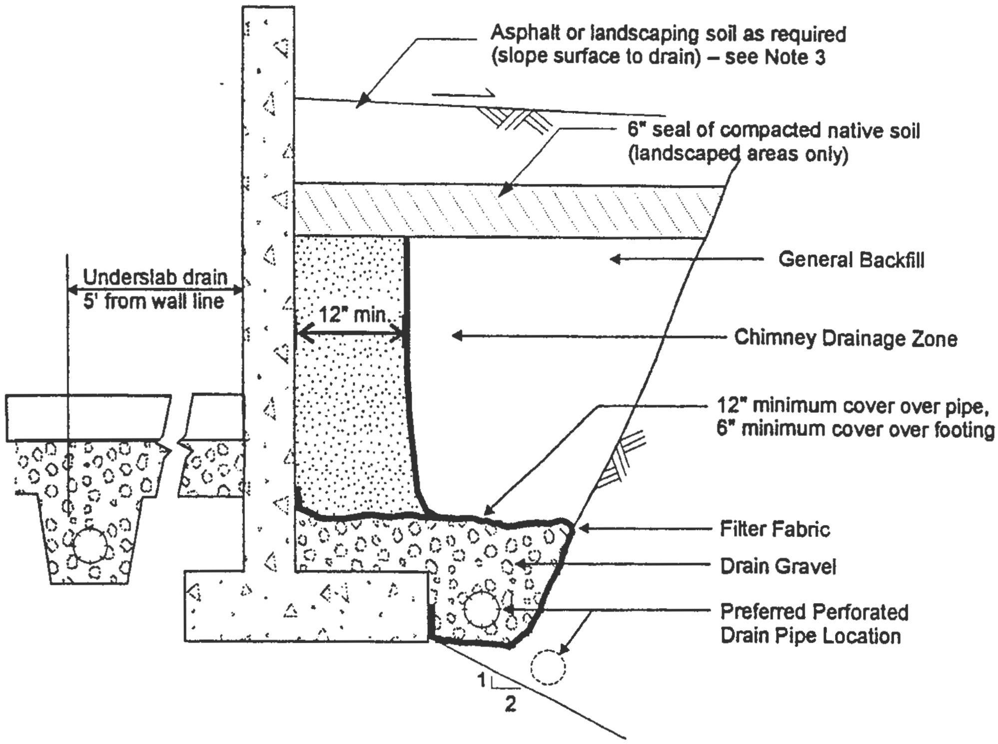
Based on the results of our field infiltration testing, we recommend using the following infiltration rate to design any on-site subsurface storm water infiltration and/or disposal systems for the project:

Subgrade Soil Type	**Recommended Infiltration Rate**
clayey, sandy SILT/silty SAND (ML/SM)	2 to 3 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 native clayey, sandy silt to silty sand 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 2014 and/or latest edition of the State of Oregon Structural Specialty Code (OSSC) and/or Amendments to the 2015 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 and/or from the 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 "D" be used for design. Using this information, the structural engineer can select the appropriate site coefficient values (F_a and F_v) from the 2015 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.

PERIMETER FOOTING/RETAINING WALL DRAIN DETAIL		
Project No. 1004.032.G	BONAVENTURE OF MILWAUKIE TL'S 600 & 901/13333 SE RUSK RD	Figure No. 3

Table 1. Recommended Seismic Design Parameters

Site Class	S_S	S_1	Fa	Fv	S_{MS}	S_{M1}	S_{DS}	S_{D1}
D	0.965	0.412	1.114	1.588	1.075	0.654	0.717	0.436

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 on IBC 2015 tables 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 Bonaventure of Milwaukie senior living and/or care facility development. 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 Bonaventure of Milwaukie senior living and/or care facility structure and its 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/developers responsibility for insuring 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 excavating eleven (11) exploratory test pits (TH-#1 through TH-#11) on September 7, 2018. The approximate location of the test pit explorations are shown in relation to the existing site improvements on the Site Exploration Plan, Figure No. 2.

The test pits were excavated using track-mounted 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 ranging from about 5.0 to 8.0 feet beneath existing site grades. Detailed logs of the test pits are presented on the Log of Test Pits, Figure No's. A-5 through A-10. The soils were classified in accordance with the Unified Soil Classification System (USCS), which is outlined on Figure No. A-4.

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 within any of the exploratory test pits (TH-#1 through TH-#11) excavated at the site at the time of excavating to depths of up to 8.0 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, maximum dry density and optimum moisture content, gradational characteristics, and Atterberg Limits as well as direct shear strength, consolidation and "R"-value 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

Two (2) Maximum Dry Density and Optimum Moisture Content tests were performed on representative samples 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-11.

Atterberg Limits

Two (2) Liquid Limit (LL) and Plastic Limit (PL) tests were performed on representative samples of the clayey, sandy silt and/or silty sand 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-12.

Gradation Analysis

Two (2) Gradation analyses were performed on representative samples of the 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-13.

Direct Shear Strength Test

One (1) Direct Shear Strength test was performed on a undisturbed and/or remolded sample 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-14.

Consolidation Test

One (1) Consolidation test was performed on a representative sample of the sandy, clayey silt subgrade soil to assess the compressibility characteristics of the underlying subgrade soils in accordance with ASTM Vol. 4.08 Part D-2435-80.

Conventional loading increments of 100, 200, 400, ... 12,800 psf were applied after the 100 percent time of primary consolidation was identified for each loading increment. The samples were unloaded and allowed to rebound after the completion of the loading sequence. Deflection versus time readings were recorded for all load increments from 100 through 12,800 psf. The deflection corresponding to 100 percent primary consolidation was plotted on the consolidation strain versus consolidation pressure curve, which is presented on Figure No. A-15.

<u>"R"-Value Tests</u>

Two (2) "R"-value tests were performed on remolded subgrade soil samples 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-16.

The following figures are attached and complete the Appendix:

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.

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

BONAVENTURE OF MILWAUKIE
Milwaukie, Oregon

PROJECT NO.	DATE	Figure A-4
1004.032.G	9/28/18	

BACKHOE COMPANY: Jim Smith Excavating BUCKET SIZE: 24 inches DATE: 9/07/18

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION TEST PIT NO. TH-#1 ELEVATION
0	X			13.1	GM	FILL: Gray-brown, dry, medium dense, crushed aggregate base rock
					ML/ SM	FILL: Medium brown, moist, moderately compacted, clayey, sandy SILT to silty SAND with occasional debris
5					ML	NATIVE GROUND: Dark gray-brown, moist, soft to medium stiff, sandy, clayey SILT with organics (Old Topsoil Zone)
					ML/ SM	Medium brown with gray mottling, moist to very moist, medium stiff to medium dense, clayey, sandy SILT to silty SAND
10						Total Depth = 5.0 feet No groundwater encountered at time of exploration
15						

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	TEST PIT NO. TH-#2 ELEVATION
0	X			12.8	ML/ SM	FILL: Medium brown, damp to moist, poorly to moderately compacted, clayey, sandy SILT to silty SAND with occasional concrete debris and trace of organics
5					ML	NATIVE GROUND: Dark gray-brown, moist, soft to medium stiff, sandy, clayey SILT with organics (Old Topsoil Zone)
					ML/ SM	GRay-brown with brown mottling, moist to very moist, medium stiff to medium dense, clayey, sandy SILT to silty SAND
10						Total Depth = 6.0 feet No groundwater encountered at time of exploration
15						

LOG OF TEST PITS

PROJECT NO. 1004.032.G | BONAVENTURE OF MILWAUKIE | FIGURE NO. A-5

BACKHOE COMPANY: Jim Smith Excavating BUCKET SIZE: 24 inches DATE: 9/07/18

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION
						TEST PIT NO. TH-#3 ELEVATION
0					GM	FILL: Gray-brown, damp to moist, poorly to moderately compacted, slightly clayey, silty and sandy GRAVEL with cobbles and trace of organics
5					ML/SM	FILL: Medium brown, moist to very moist, poorly to moderately compacted, clayey, sandy SILT to silty SAND with occasional debris and trace of organics
					ML	NATIVE GROUND: Dark gray-brown, very moist, soft to medium stiff, sandy, clayey SILT with organics (Old Topsoil Zone)
10					GM	Medium brown, very moist, medium dense to dense, slightly clayey, silty and sandy GRAVEL with cobbles
15						Total Depth = 7.0 feet No groundwater encountered at time of exploration
						TEST PIT NO. TH-#4 ELEVATION
0					ML/SM	FILL: Medium brown, damp to moist, poorly to moderately compacted, clayey, sandy SILT to silty SAND with gravel and trace organics
					ML	NATIVE GROUND: Dark gray-brown, moist to very moist, soft to medium stiff, sandy, clayey SILT with organics (Old Topsoil Zone)
5					ML/SM	Medium brown with gray mottling, very moist to wet, soft to loose, clayey, sandy SILT to silty SAND
					GM	Gray-brown, very moist to wet, medium dense to dense, slightly clayey, silty and sandy GRAVEL with cobbles
10						Total Depth = 7.0 feet No groundwater encountered at time of exploration
15						

LOG OF TEST PITS

PROJECT NO. 1004.032.G | BONAVENTURE OF MILWAUKIE | FIGURE NO. A-6

BACKHOE COMPANY: Jim Smith Excavating BUCKET SIZE: 24 inches DATE: 9/07/18

SOIL DESCRIPTION

TEST PIT NO. TH-#5 ELEVATION

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION
0	X	X	101.2	12.6	ML/SM	FILL: Medium brown, damp to moist, poorly to moderately compacted, clayey, sandy SILT to silty SAND with occasional gravel and debris
5	X			17.7	ML	NATIVE GROUND: Dark gray-brown, moist to very moist, soft to medium stiff, sandy, clayey SILT with trace organics (Old Topsoil Zone)
					GM	Dark gray-brown, very moist, medium dense to dense, slightly clayey, silty and sandy GRAVEL with cobbles
10						Total Depth = 7.0 feet No groundwater encountered at time of exploration
15						

TEST PIT NO. TH-#6 ELEVATION

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION
0	X	X	97.7	13.4	ML/SM	FILL: Medium brown, damp to moist, poorly to moderately compacted, clayey, sandy SILT to silty SAND with occasional gravel and denris
5					ML	NATIVE GROUND: Dark gray-brown, moist to very moist, soft to medium stiff, sandy, clayey SILT with organics (Old Topsoil Zone)
	X			24.4	ML/SM	Medium brown with gray mottling, moist to very moist, medium stiff to medium dense, clayey, sandy SILT to silty SAND
10						Total Depth = 7.0 feet No groundwater encountered at time of exploration
15						

LOG OF TEST PITS

PROJECT NO. 1004.032.G	BONAVENTURE OF MILWAUKIE	FIGURE NO. A-7

BACKHOE COMPANY: Jim Smith Excavating BUCKET SIZE: 24 inches DATE: 9/07/18

SOIL DESCRIPTION

TEST PIT NO. TH-#7 ELEVATION

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION
0					ML/ SM	FILL: Medium brown, damp to moist, poorly to moderately compacted, clayey, sandy SILT to silty SAND with concrete, brick and organics
5					GM	FILL: Gray-brown, moist, moderately compacted, slightly clayey, silty and sandy GRAVEL with cobbles and trace organics
					ML	NATIVE GROUND: Dark gray-brown, moist to very moist, soft to medium stiff, sandy, clayey SILT with organics (Old Topsoil ZOne)
10					GM	GrayUbrown, moist to very moist, medium dense to dense, slightly clayey, silty and sandy GRAVEL with cobbles
						Total Depth = 8.0 feet No groundwater encountered at time of exploration
15						

TEST PIT NO. TH-#8 ELEVATION

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION
0					GM	FILL: Medium brown, moist, moderately compacted, slightly clayey, silty and sandy GRAVEL with cobbles and trace organics
					ML	NATIVE GROUND: Dark brown, moist to very moist, soft, organic, sandy, clayey SILT (Old Topsoil Zone)
5	X			18.8	SM/ SP	Gray-brown to bluish-gray, very moist to wet, loose, silty to slightly silty, fine to medium SAND
						Total Depth = 7.0 feet No groundwater encountered at time of exploration
10						
15						

LOG OF TEST PITS

PROJECT NO. 1004.032.G BONAVENTURE OF MILWAUKIE FIGURE NO. A-8

BACKHOE COMPANY: Jim Smith Excavating BUCKET SIZE: 24 inches DATE: 9/07/18

SOIL DESCRIPTION

TEST PIT NO. TH_#9 ELEVATION

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION
0					ML/ SM	FILL: Medium brown, damp to moist, poorly to moderately compacted, clayey, sandy SILT to silty SAND with gravel and miscellaneous construction debris
					ML	NATIVE GROUND: Dark brown, moist to very moist, soft, organic, sandy, clayey SILT (OLd Topsoil Zone)
5					SM	Medium to olive-brown, very moist, loose to medium dense, silty, fine to medium SAND
					GM	Gray-brown, very moist, medium dense to dense, slightly clayey, silty and sandy GRAVEL with cobbles
10						Total Depth = 7.0 feet No groundwater encountered at time of exploration
15						

TEST PIT NO. TH-#10 ELEVATION

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION
0					ML/ SM	FILL: Medium brown, damp to moist, poorly to moderately compacted, clayey, sandy SILT to silty SAND with organics and construction debris
	X	X	99.1	14.4		
	X	X	90.3	17.7		
5					ML	NATIVE GROUND: Dark gray-brown, very moist, soft to medium stiff, sandy, clayey SILT with organics (Old Topsoil Zone)
					SM	Gray-brown, very moist, loose to medium dense, slightly clayey, silty SAND
10						Total Depth = 8.0 feet No groundwater encountered at time of exploration
15						

LOG OF TEST PITS

PROJECT NO. 1004.032.G BONAVENTURE OF MILWAUKIE FIGURE NO. A-9

BACKHOE COMPANY: Jim Smith Excavating BUCKET SIZE: 24 inches DATE: 9/07/18

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION TEST PIT NO. TH-#11 ELEVATION
0					ML/SM	FILL: Medium brown, moist, moderately compacted, clayey, sandy SILT to silty SAND with occasional gravel and trace organics
					ML	NATIVE GROUND: Dark gray-brown, moist, soft to medium stiff, slightly organic, sandy, clayey SILT (OLd Topsoil Zone)
5					ML/SM	Medium brown with gray mottling, moist to very moist, medium stiff to medium dense, clayey, sandy SILT to silty SAND
						Total Depth = 5.0 feet No groundwater encountered at time of exploration
10						
15						

DEPTH (FEET)						TEST PIT NO. ELEVATION
0						
5						
10						
15						

LOG OF TEST PITS

PROJECT NO. 1004.032.G BONAVENTURE OF MILWAUKIE FIGURE NO. A-10

MAXIMUM DENSITY TEST RESULTS

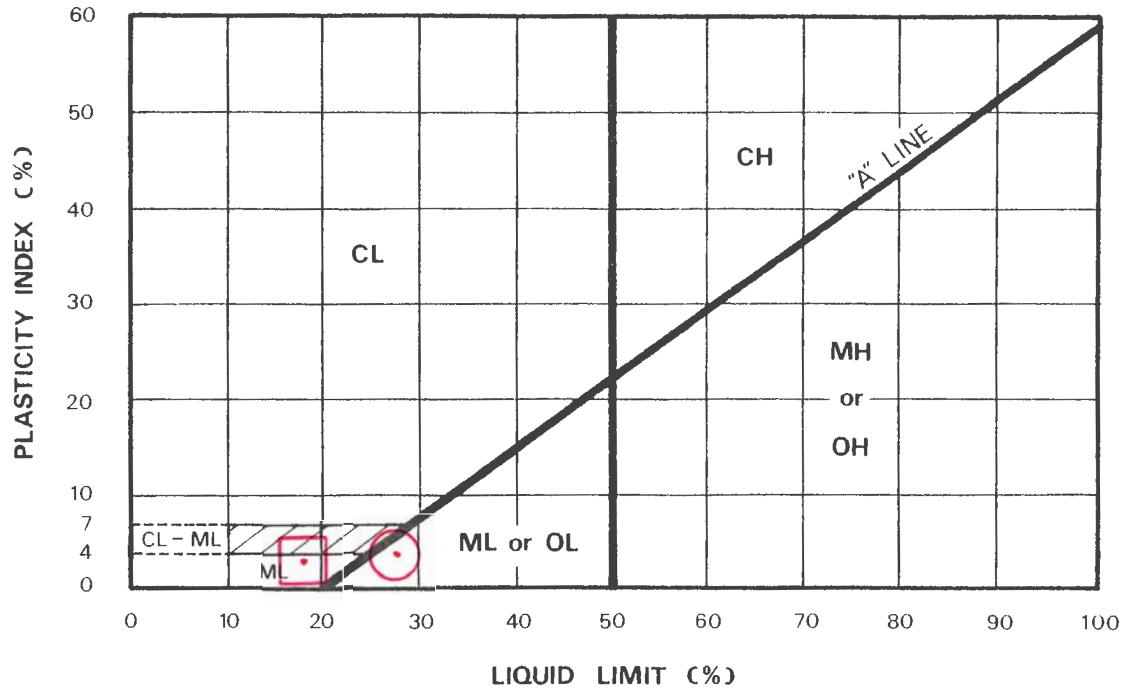
SAMPLE LOCATION	SOIL DESCRIPTION	MAXIMUM DRY DENSITY (pcf)	OPTIMUM MOISTURE CONTENT (%)
TH-#5 @ 1.5'	Medium brown, clayey, sandy SILT to silty SAND with gravel (ML/SM)	110.0	17.0
TH-#10 @ 1.5'	Medium brown, clayey, sandy SILT to silty SAND with gravel (ML/SM)	112.0	16.0

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.: 1004.032.G | BONAVENTURE OF MILWAUKIE | FIGURE NO.: A-11



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-#6	5.5	24.4	27.8	4.1	60.3		ML
⊙	TH-#8	5.5	18.8	18.5	3.7	23.2		SM

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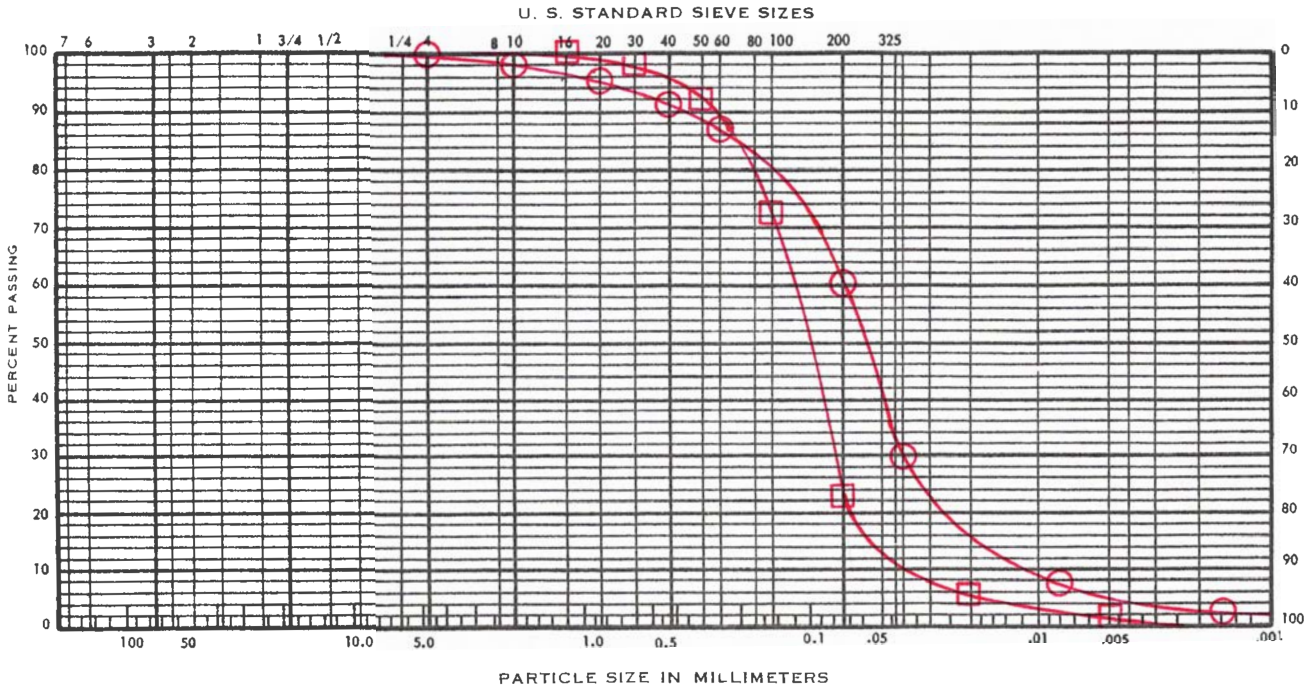
PLASTICITY CHART AND DATA

BONAVENTURE OF MILWAUKIE
Milwaukie, Oregon

PROJECT NO.	DATE	
1004.032.G	9/28/18	Figure A-12

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-#6	5.5		ML	Medium brown, clayey, sandy SILT to silty SAND
─○─	TH-#8	5.5		SM	GRay-brown, silty fine to medium SAND

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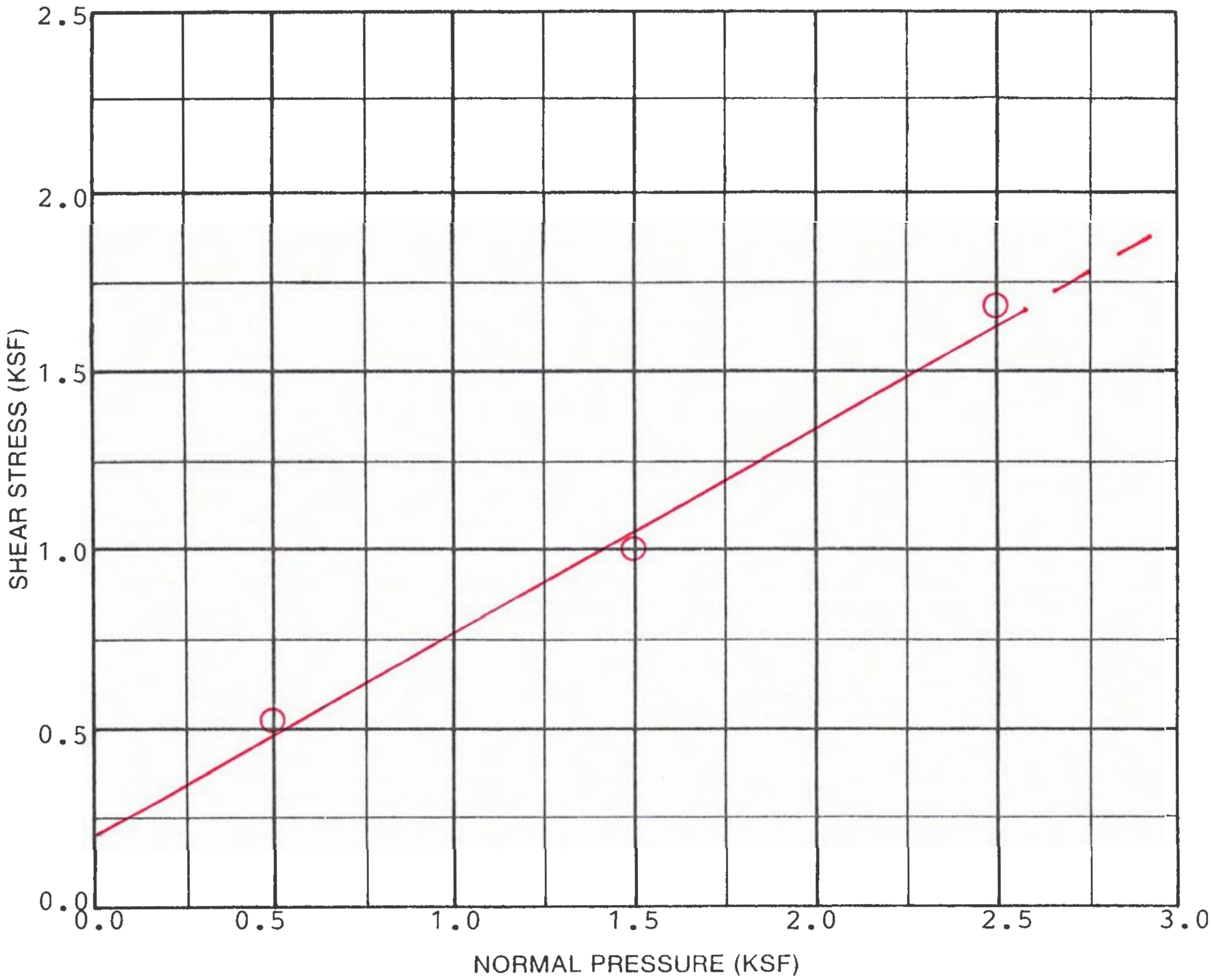
GRADATION TEST DATA

BONAVENTURE OF MILWAUKIE
Milwaukie, Oregon

PROJECT NO.	DATE	FIGURE
1004.032.G	9/28/18	A-13



SAMPLE DATA	
DESCRIPTION: Medium brown, clayey, sandy SILT to silty SAND (ML/SM) (Remolded)	
BORING NO.: TH-#5	
DEPTH (ft.): 1.5	ELEVATION (ft):
TEST RESULTS	
APPARENT COHESION (C): 200 psf	
APPARENT ANGLE OF INTERNAL FRICTION (ϕ): 28°	

TEST DATA				
TEST NUMBER	1	2	3	4
NORMAL PRESSURE (KSF)	0.5	1.5	2.5	
SHEAR STRENGTH (KSF)	0.5	1.0	1.6	
INITIAL H_2O CONTENT (%)	16.0	16.0	16.0	
FINAL H_2O CONTENT (%)	16.4	12.1	7.8	
INITIAL DRY DENSITY (PCF)	98.0	98.0	98.0	
FINAL DRY DENSITY (PCF)	98.8	102.9	106.8	
STRAIN RATE: 0.02 inches per minute				

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DIRECT SHEAR TEST DATA		
BONAVENTURE OF MILWAUKIE Milwaukie, Oregon		
PROJECT NO. 1004.032.G	DATE 9/28/18	Figure A-14

BORING : TH-#5
DEPTH (ft) : 1.5
SPEC. GRAVITY : 2.5 (assumed)

DESCRIPTION : clayey, sandy SILT (FILL)
LIQUID LIMIT : 18.5
PLASTIC LIMIT : 14.8

	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	PERCENT SATURATION	VOID RATIO
INITIAL	16.0	98.0	91.1	
FINAL	7.8	106.6	97.2	

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CONSOLIDATION TEST DATA

BONAVENTURE OF MILWAUKIE
Milwaukie, Oregon

PROJECT NO.	DATE	
1004.032.G	9/28/18	Figure A-15

RESULTS OF R (RESISTANCE) VALUE TESTS

SAMPLE LOCATION: TH-#6

SAMPLE DEPTH: 1.5 feet bgs

Specimen	A	B	C
Exudation Pressure (psi)	219	329	431
Expansion Dial (o.ooo1”)	0	1	2
Expansion Pressure (psf)	0	3	8
Moisture Content (%)	17.6	14.4	11.1
Dry Density (pcf)	96.4	101.2	107.6
Resistance Value, “R”	18	29	36
“R”-Value at 300 psi Exudation Pressure = 28			

SAMPLE LOCATION: TH-#10

SAMPLE DEPTH: 1.5 feet bgs

Specimen	A	B	C
Exudation Pressure (psi)	208	326	439
Expansion Dial (0.0001”)	0	1	2
Expansion Pressure (psf)	0	3	8
Moisture Content (%)	17.3	14.1	10.7
Dry Density (pcf)	98.9	103.1	109.7
Resistance Value “R”	19	33	40
“R”-Value at 300 psi Exudation Pressure = 32			