REDMOND GEOTECHNICAL SERVICES

Geotechnical Investigation and Consultation Services

Proposed Newberg Apartments Development Project

Tax Lot No. 800

E Haworth Avenue and N Springbrook Road

Newberg (Yamhill County), Oregon

for

Grove Development

Project No. 1213.020.G
November 10, 2022

REDMOND GEOTECHNICAL SERVICES

November 10, 2022

Grove Hunt
Grove Development
6500 SW Beaverton Hillsdale Highway, Suite #3
Portland, Oregon 97255

Dear Mr. Hunt:

Re: Geotechnical Investigation and Consultation Services,
Proposed Newberg Apartments Development Site, Tax Lot No. 800,
E Haworth Avenue and N Springbrook Road, Newberg (Yamhill County), Oregon

Submitted herewith is our report entitled "Geotechnical Investigation and Consultation Services, Proposed Newberg Apartments Development Site, Tax Lot No. 800, E Haworth Avenue and N Springbrook Road, Newberg (Yamhill County), Oregon". The scope of our services was outlined in our formal proposal to Mr. Grove Hunt of Grove Development on October 15, 2022. Authorization of our services was provided by Mr. Grove Hunt of Grove Development on October 18, 2020.

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. 12-31-22

Cc: Mr. Curt Olson
Olson Group Architects

TABLE OF CONTENTS

Table of Contents (continued)

GEOTECHNICAL INVESTIGATION AND CONSULTATION SERVICES
PROPOSED NEWBERG APARTMENTS DEVELOPMENT SITE
TAX LOT NO. 800, E HAWORTH AVENUE & N SPRINGBROOK ROAD
NEWBERG (YAMHILL COUNTY), OREGON

INTRODUCTION

Redmond Geotechnical Services, LLC is please to submit to you the results of our Geotechnical Investigation at the site of the proposed new Newberg Apartments project located to the west of NE Lafayette Avenue and north of NE 9th Avenue in McMinnville (Yamhill County), Oregon. The general location of the subject site is shown on Site Vicinity Map, Figure No. 1. The purpose of our geotechnical investigation services at this time was to explore the existing subsurface soils and/or groundwater conditions across the subject site and to develop and/or provide appropriate geotechnical design and construction recommendations for the proposed new Newberg Apartments project.

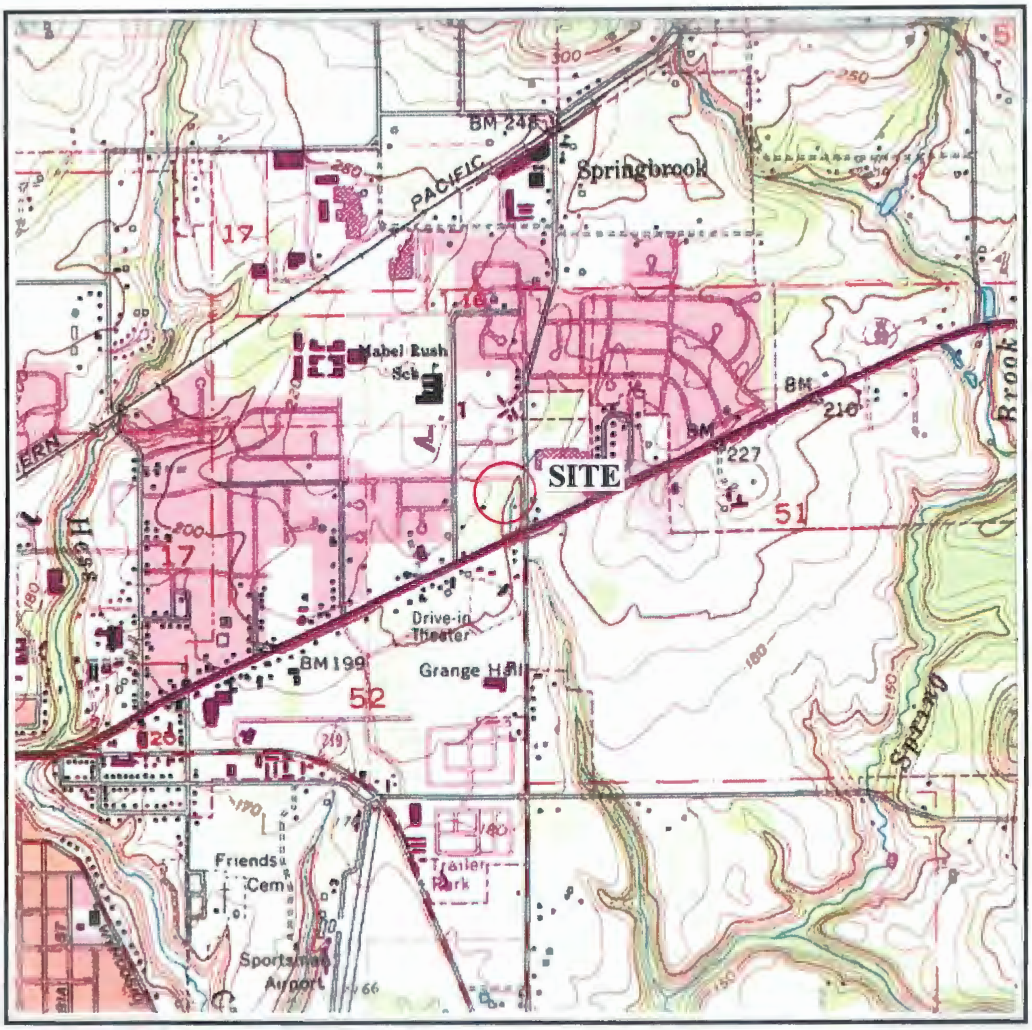
PROJECT DESCRIPTION

Based on a review of the proposed site development plan(s), we understand that present plans are to construct a new apartment building at the site. Reportedly, the proposed new apartment building will be a three-story wood-frame structure with a concrete slab-on-grade floor and will have a base and/or ground floor level footprint of approximately 7,500 square feet.

Support of the new apartment building structure is anticipated to include both conventional shallow continuous (strip) footings as well as individual (spread) column-type footings. Structural loading information, although unavailable at this time, is expected to result in maximum dead plus live continuous (strip) and individual (column) footing loads on the order of about 3.0 to 5.0 kips per lineal foot (klf) and 25 to 75 kips, respectively.

Other associated site improvements for the project will include new underground utility services, concrete curbs and sidewalks, and landscaping as well as new paved parking and drive areas.

Site grading and earthwork required to bring the subject property to finish design grades is anticipated to result in relatively minor cuts and/or fills of about one (1) to two (2) feet.



NEWBERG QUADRANGLE
OREGON
7.5-MINUTE SERIES

SCALE 1:24 000

1	0.5	0	KILOMETERS	1	2						
1000	500	0	METERS	1000	2000						
1	0.5	0	MILES	1							
1000	0	1000	2000	3000	4000	5000	6000	7000	8000	9000	10000

FEET

CONTOUR INTERVAL 10 FEET
NORTH AMERICAN VERTICAL DATUM OF 1988

SITE VICINITY MAP

Project No. 1213.020.G

NEWBERG APARTMENTS SITE
TL 800, EAST HAWORTH AVENUE

Figure No. 1

SCOPE OF WORK

The purpose of our geotechnical studies was to evaluate the overall site subsurface soil and/or groundwater conditions underlying the site with regard to the proposed new apartment building construction at the site and any associated impacts or concerns with respect to the new commercial development 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. A detailed field reconnaissance and subsurface exploration program of the soil and ground water conditions underlying the site by means of five (5) exploratory test holes. The exploratory test holes were excavated to depths of between four (4) and seven (7) feet beneath existing site grades at the approximate locations as shown on the Site Exploration Plan, Figure No. 2.

2. 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 and Atterberg Limits as well as consolidation and "R"-value tests.

3. 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.

4. Engineering analyses utilizing the field and laboratory data as a basis for furnishing recommendations for foundation support of the proposed new apartment building 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 and/or floor slab subgrades.

5. Development of various flexible pavement design sections for paved access drives and vehicle parking areas as well as for any heavy vehicle traffic areas.

SITE CONDITIONS

Site Geology

Available geologic mapping of the area and/or subject site indicates that the near surface soils consist of Quaternary age terrace deposits (Qtm). Characteristics include semi-consolidated gravel, sand, silt and clay forming very flat terraces of major extent along the Yamhill River. Generally, 10 to 30 feet of medium brown silty clay interbedded with very fine sandy silt believed to be related to Willamette Valley Silt including associated glacial erratics consisting of tiny fragments and pebble up to boulders greater than 4 feet in diameter. Soils are poorly drained and subject to seasonal high groundwater and ponding.

Surface Conditions

The subject property is generally rectangular in shape and is comprised of one (1) separate tax lot (TL 800) encompassing a total area of approximately 0.82 acres. The subject property is roughly bounded to the east by N Springbrook Road, to the north by E Haworth Avenue, and to the south and west by existing and developed commercial properties.

The subject property is presently unimproved and void of existing structures and/or site improvements. However, the subject property reportedly contains an existing underground storm sewer line within an existing 15-feet wide storm sewer easement.

Topographically, the subject site is characterized as relatively flat-lying to gently sloping terrain descending downward towards the east/northeast with overall topographic relief across the entire site estimated at about five (5) feet and ranges from a high of about Elevation 208 feet to a low of about Elevation 203 feet. Vegetation across the site generally consists of a light to moderate growth of grass and weeds.

Subsurface Soil Conditions

Our understanding of the subsurface soil conditions underlying the site was developed by means of five (5) exploratory test holes excavated to depths of between four (4) and seven (7) feet beneath existing site grades on August 20, 2022 with tracked excavation equipment. The location of the exploratory test holes were located in the field by marking off distances from existing and/or known site features and are shown in relation to the existing and/or proposed new site improvements on the Site Exploration Plan, Figure No. 2. Detailed logs of the test hole explorations, presenting conditions encountered at each location explored, are presented in the Appendix, Figure No's. A-5 through A-7.

The exploratory test hole explorations performed during this study were observed by staff from Redmond Geotechnical Services, LLC who logged the test hole explorations and obtained representative samples of the subsurface soils encountered beneath the site. All subsurface soils encountered at the site and/or within the exploratory test hole explorations were logged and classified in general conformance with the Unified Soil Classification System (USCS) which is outlined on Figure No. A-4.

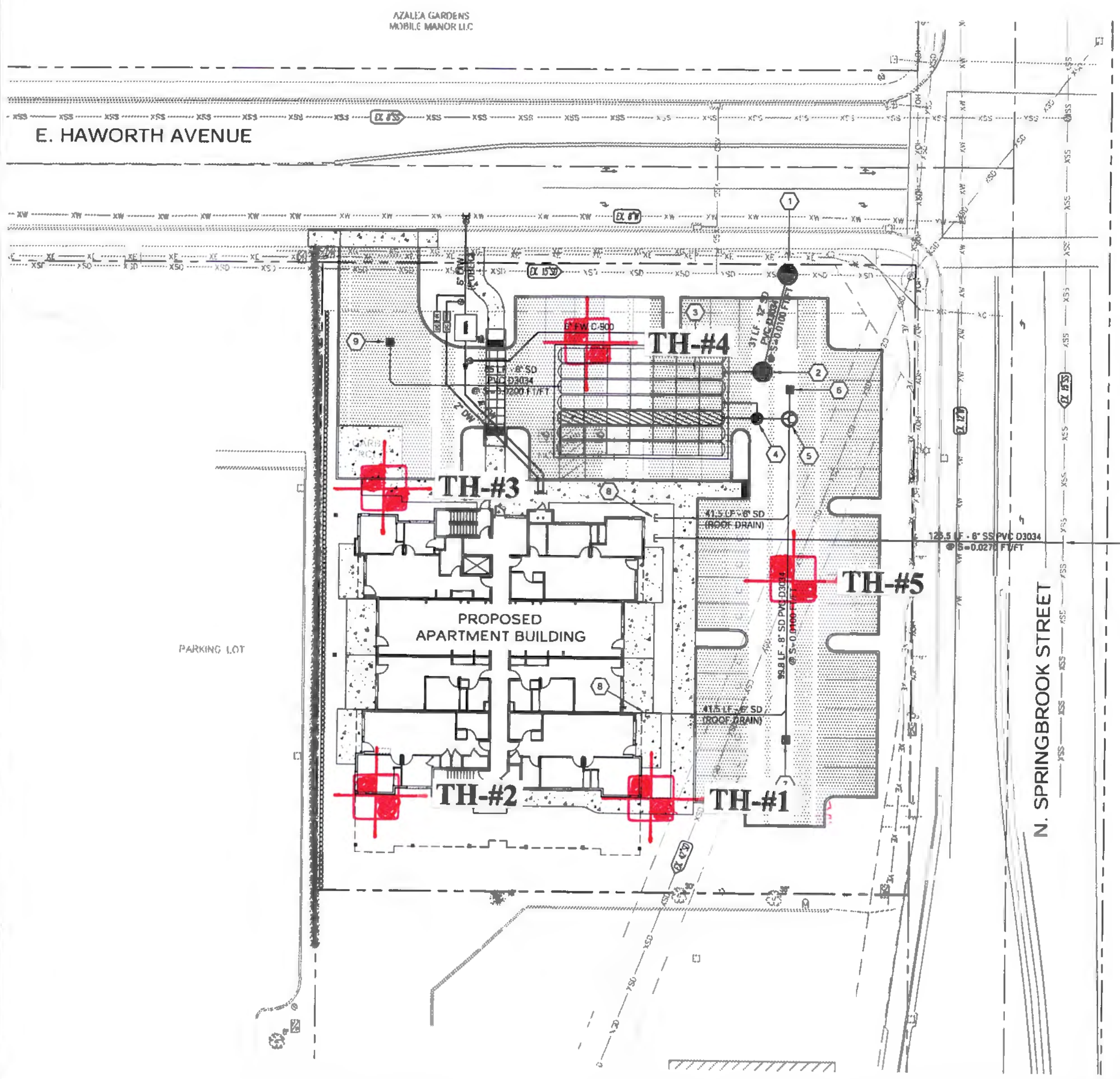
The test hole explorations revealed that the subject site is generally underlain at depth by native soil deposits comprised of terrace soil deposits (Qtm) of Quaternary age. Specifically, the subsurface soils underlying the project area generally consists of a upper layer of topsoil materials comprised of dark brown, wet, soft, organic, sandy, clayey silt which extends to a depth of about 12 inches. These surficial topsoil materials were inturn underlain by medium to olive-brown, very moist, medium stiff, sandy, clayey silt to the maximum depth explored of about seven (7) feet beneath the existing site and/or surface grades. These underlying clayey, sandy silt subgrade soil materials become sandier with depth and are best characterized by relatively low to moderate strength and moderate compressibility. In addition to the above, localized fill soils were also encountered in test hole TH-#5 and/or within the existing storm sewer easement. The fill soils are believed to be trench excavation spoils which were found to be moderately well compacted.

Groundwater

Groundwater was not encountered within the exploratory test hole explorations at the time of excavating at a depth of at least seven (7) feet beneath existing site grades. Additionally, based on a review of available water wells in the area as well as a review of the Depth to Seasonal High Groundwater prepared by Yamhill County, the apparent depth to seasonal high groundwater in the area of the subject site is greater than 10 feet. However, groundwater elevations at and/or below the subject site may fluctuate seasonally in accordance with rainfall conditions and/or changes in the site utilization.

INFILTRATION TESTING

We performed one (1) field infiltration test at the site on August 20, 2022. The infiltration test was performed in test hole TH-#4 at a depth of between three (3) and four (4) feet beneath the existing site and/or surface grades. The subgrade soils encountered in the infiltration test hole consisted of sandy, clayey silt. The infiltration testing was performed in general conformance with current EPA and/or the City of Newberg/Yamhill 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.



LEGEND

TH-#5 Indicates approximate location of exploratory test hole

SCALE 20 0 10 20

SITE EXPLORATION PLAN		
Project No. 1213.020.G	NEWBERG APARTMENTS SITE TL 800, EAST HAWORTH AVENUE	Figure No. 2

Based on the results of the field infiltration testing at the site, we have found that the native sandy, clayey silt subgrade soil deposits posses an ultimate infiltration rate on the order of about 0.4 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 hole explorations 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, gradation analyses and Atterberg Limits as well as consolidation and "R"-value tests. Results of the various laboratory tests are presented in the Appendix on Figure No's. A-8 through A-12.

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 Mw9 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 Portland 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 hole logs from our exploratory field explorations (TH-#1 through TH-#5) and laboratory test results indicates that the site is generally underlain by medium stiff, sandy, clayey silt to the maximum depth explored of about seven (7) feet beneath existing site grades. Additionally, groundwater was generally not encountered at the site during our field exploration work to depths of at least seven (7) feet.

As such, due to the anticipated depth to groundwater as well as the medium stiff and cohesive nature of the underlying sandy, clayey silt subgrade soil deposits beneath the site, it is our opinion that the native soil deposits located beneath the subject site do not have the potential for liquefaction during the design earthquake motions previously described. A more detailed liquefaction assessment was not part of the scope of work for this Geotechnical Investigation.

Landslides

No ancient and/or active landslides were observed or are known to be present on the subject site. Additionally, due to the relatively flat-lying to gently sloping nature of the subject site, the risk of seismic induced slope instability at the site resulting in landslides and/or lateral earth movements do not appear to present a potential geologic hazard.

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 Yamhill County and the City of Newberg. The FEMA (Federal Emergency Management Agency) flood maps should be reviewed as part of the design for the proposed new apartment building structure and its associated site improvements. Elevations of structures on the site should be designed based upon consultants reports, FEMA (Federal Emergency Management Agency), and Yamhill County requirements for the 100-year flood levels of any nearby creeks and/or streams.

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 suitable for the proposed new Newberg Apartments building and the associated site improvements described herein provided that the recommendations contained within this report are properly incorporated into the design and construction of the project.

The primary features of concern at the site are 1) the moisture sensitivity of the near surface sandy, clayey silt subgrade soils, 2) the possible presence of old and/or abandoned building foundations and/or site improvements the site and 3) the presence of the existing storm sewer line and easement.

With regard to the moisture sensitivity of the near surface sandy, clayey silt subgrade soils, we are generally of the opinion that all site grading and earthwork operations be scheduled for the drier summer months which is typically June through September. In regard to the possible presence of old building foundations and/or site improvements across the site, we recommend that any old building foundations and/or basements as well as utility services located within the proposed new apartment building footprint be removed in their entirety down to an approved native subgrade soil. Additionally, we anticipate that the site may contain some existing undocumented fill materials. As such, we are of the opinion that all existing fill materials present and/or encountered at the site should be considered non-structural and, as such, should be removed in their entirety. Further, all abandoned drywells and/or septic tanks as well as prior underground heating oil tanks and/or UST tank cavity's encountered at the site should be filled with a controlled density fill (CDF) and/or structural fill materials as recommended by the Geotechnical Engineer. In regard to the presence of the existing storm sewer line and/or easement, we understand that the trench backfill materials may not be compacted to the requirements of structural fill. As such, we recommend that the building foundations for the proposed new apartment building be located at least five (5) feet or more from the edge of the existing easement. Additionally, we recommend that the upper 1 to 2 feet of the trench backfill materials located beneath the proposed new pavements be removed and/or recompacted to the requirement of structural fill. In this regard, we recommend that close monitoring of all site grading and earthwork operations be performed by the Geotechnical Engineer.

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 Newberg Apartments project.

Site Preparation

As an initial step in site preparation, we recommend that the proposed new apartment building area and any associated structural and/or site improvement area(s) be stripped and cleared of all existing site improvements, any existing 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 pavement materials will generally be about 6 to 12 inches. However, localized areas requiring deeper removals, such as any existing fill materials, existing and/or old foundation remnants and/or large tree root systems, will 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 silty 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 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 dental 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 three (3) lineal feet of the perimeter (limits) of the proposed apartment structure and/or pavements should be considered as structural fill. All aspects of the site grading and earthwork operations should be monitored and approved 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 Newberg Apartments building is suitable for support of the three-story wood-frame structure provided that the following foundation design recommendations are followed. The following section(s) of this report present specific foundation design and construction recommendations for the planned new apartment building structure.

Shallow Foundations

In general, conventional shallow continuous (strip) footings and individual (spread) column footings may be supported by approved native (untreated) sandy, clayey silt subgrade soil materials and/or structural fill soils based on an allowable contact bearing pressure of about 2,000 pounds per square foot (psf). However, where higher allowable contact bearing pressures are desired and/or required, an allowable contact bearing pressure of up to 2,500 psf may be used for design where foundations are supported by a minimum of at least 6 inches or more of properly compacted (structural fill) crushed aggregate base rock (granular) fill material placed directly above and/or by the existing and approved native medium stiff, sandy, clayey silt subgrade soil materials. These recommended allowable contact bearing pressures are intended for dead loads and sustained live loads and may be increased by fifty percent (50%) 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.

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 three-story 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 0.35 and 0.50 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 250 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. Additional moisture protection, where needed, can be provided by using a 10-mil polyolefin geo-membrane sheeting 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 200 pounds per square inch per inch 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.

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 an assumed subgrade soil strength ("R"-value). Based on a laboratory subgrade "R"-value of 32 (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 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 0.5 inches of asphaltic concrete and 2.0 inches of aggregate base rock. Additionally, for wet weather construction, we recommend a minimum gravel base rock thickness of at least 12 inches. Further, 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 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.

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.

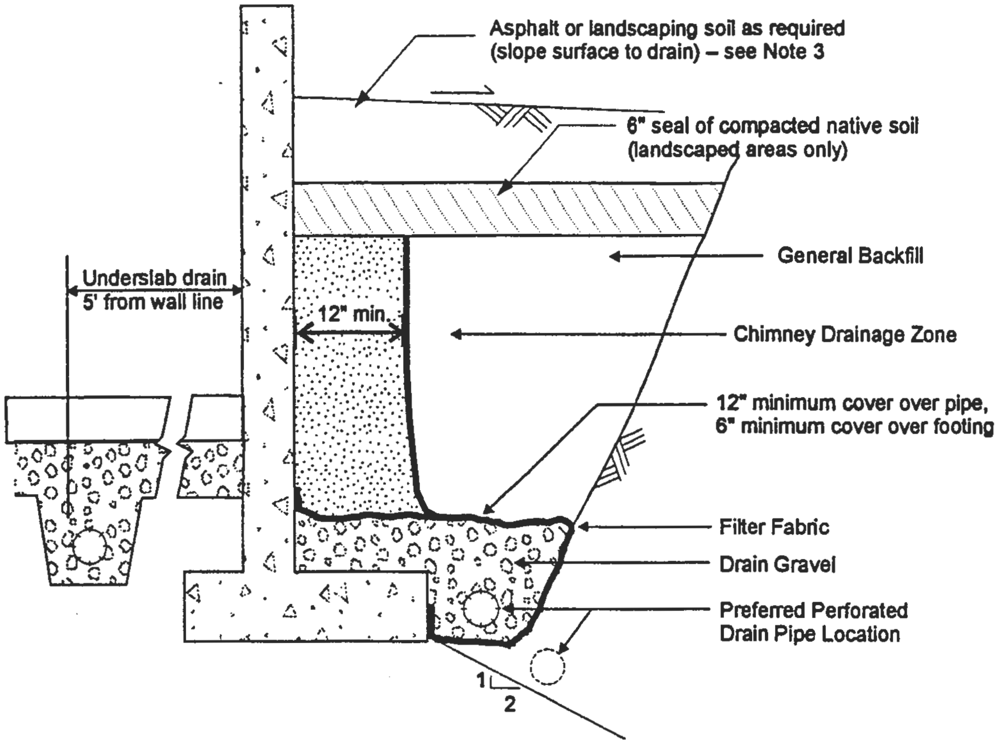
Depending on the time of year in which trench excavations occur, trench dewatering may be required in order to maintain dry working conditions if the invert elevations of the proposed utilities are located at and/or below the groundwater level. If groundwater is encountered during utility excavation 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 office building and landscaping areas as well as adjacent properties or buildings are directed away from the new apartment structures foundations and/or floor slabs. All roof drainage should be directed into conduits that carry runoff water away from the new apartment building 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 apartment building.

Groundwater was not encountered at the site within any of the exploratory test holes (TH-#1 through TH-#5) at the time of excavating to a depth of at least seven (7) feet beneath existing site grades. However, although groundwater elevations in the area may fluctuate seasonally and may temporarily pond/perch near the ground surface during periods of prolonged and/or heavy rainfall, based on our current understanding of the project, we are generally of the opinion that the reported static groundwater levels in the area of the subject site represent the seasonal high groundwater elevation(s) at and/or near to the subject site.

As such, based on our current understand of the site grading required to bring the subject site to finish design grades, we are of the opinion that an underslab drainage system is not required for the proposed new apartment building structure. However, due to the presence of sandy, clayey silt subgrade soils within the foundation bearing level of the proposed new apartment building structure, we are generally of the opinion that a perimeter footing/foundation drainage system should be used around the perimeter of the proposed apartment structure. Additionally, a foundation drain is recommended for any below grade footing and/or retaining walls. A typical recommended perimeter footing and/or retaining wall drain detail is shown on Figure No. 3.



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. 1213.020.G	NEWBERG APARTMENTS SITE TL 800, EAST HAWORTH AVENUE	Figure No. 3

Design Infiltration Rates

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

Subgrade Soil Type	Recommended Infiltration Rate
Sandy, clayey SILT (ML)	0.2 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 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 "D" be used for design.

Using this information, the structural engineer can select the appropriate site coefficient values (F_a and F_v) from ASCE 7-17 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:

Table 1. IBC Seismic Design Parameters

Site Class	S_s	S_1	Fa	Fv	S_{MS}	S_{M1}	S_{DS}	S_{D1}
D	0.850	0.410	1.160	1.890	0.986	0.776	0.658	0.517

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 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 Newberg Apartments building 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 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, structural fill placement, footing excavations and construction as well as any 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 Newberg Apartments structure 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 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 construction 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.

REFERENCES

Adams, John, 1984, Active Deformation of the Pacific Northwest Continental Margin: Tectonics, v.3, no. 4, p. 449-472.

Applied Technology Council, ATC-13, 1985, Earthquake Damage Evaluation Data for California.

Atwater, B.F., 1992, Geologic evidence for earthquakes during the past 2000 years along the Copalis River, southern coastal Washington: Journal of Geophysical Research, v. 97, p. 1901-1919.

Atwater, B.F., 1987a, A periodic Holocene recurrence of widespread, probably coseismic Subsidence in southwestern Washington: EOS, v. 68, no. 44.

Atwater, B.F., 1987b, Evidence for great Holocene earthquakes along the outer coast of Washington State: Science, v. 236, no. 4804, pp. 942-944.

Campbell, K.W., 1990, Empirical prediction of near-surface soil and soft-rock ground motion for the Diablo Canyon Power Plant site, San Luis Obispo County, California: Dames & Moore report to Lawrence Livermore National Laboratory.

Carver, G.A., and Burke, R.M., 1987, Late Holocene paleoseismicity of the southern end of the Cascadia Subduction zone [abs.]: EOS, v. 68, no. 44, p. 1240.

Chase, R.L., Tiffin, D.L., Murray, J.W., 1975, The western Canadian continental margin: In Yorath, C.J., Parker, E.R., Glass, D.J., editors, Canada's continental margins and offshore petroleum exploration: Canadian Society of Petroleum Geologists Memoir 4, p. 701-721.

Crouse, C.B., 1991a, Ground motion attenuation equations for earthquakes on the Cascadia Subduction Zone: Earthquake Spectra, v. 7, no. 2, pp. 201-236.

Crouse, C.B., 1991b, Errata to Crouse (1991a), Earthquake Spectra, v. 7, no. 3, p. 506.

Darienzo, M.E., and Peterson, C.D., 1987, Episodic tectonic subsidence recorded in late Holocene salt marshes, northern Oregon central Cascadia margin: Tectonics, v. 9, p. 1-22.

Darienzo, M.E., and Peterson, C.D., 1987, Episodic tectonic subsidence recorded in late Holocene salt marshes northwest Oregon [abs]: EOS, v. 68, no. 44, p. 1469.

EERI (Earthquake Engineering Research Institute), 1993, The March 25, 1993, Scotts Mill Earthquake, Western Oregon's Wake-Up Call: EERI Newsletter, Vol. 27, No. 5, May.

Geomatrix, 1995 Seismic Design Mapping, State of Oregon: Final Report to Oregon Department of Transportation, January.

Geologic Map Series (GMS-49), Map of Oregon Seismicity, 1841-1986 dated 1986.

Geologic Map of the McMinnville Quadrangle, Oregon by James L. Bela dated 1981.

Grant, W.C., and McLaren, D.D., 1987, Evidence for Holocene Subduction earthquakes along the northern Oregon coast [abs]: EOS v. 68, no. 44, p. 1239.

Grant, W.C., Atwater, B.F., Carver, G.A., Darienzo, M.E., Nelson, A.R., Peterson, C.D., and Vick, G.S., 1989, Radiocarbon dating of late Holocene coastal subsidence above the Cascadia Subduction zone-compilation for Washington, Oregon, and northern California, [abs]: EOS Transactions of the American Geophysical Union, v. 70, p. 1331.

International Conference of Building Officials (ICBO), 1994, Uniform Building Code: 1994 Edition, Whittier, CA. 1994.

Joyner, W.B., and Boore, D.M., 1998, Measurement, characterization and prediction of strong ground motion: Earthquake Engineering and Soil Dynamics II – Recent Advances in Ground Motion Evaluation, ASCE Geotech. Special Publ. No. 20, p. 43-102.

Riddihough, R.P., 1984, Recent movements of the Juan de Fuca plate system: Journal of Geophysical Research, v. 89, no. B8, p. 6980-6994.

Youngs, R.R., Day, S.M., and Stevens, J.L., 1998, Near field ground motions on rock for large Subduction earthquakes: Earthquake Engineering and Soil Dynamics II – Recent Advances in Ground Motion Evaluation, ASCE Geotech. Special Publ. No. 20, p. 445-462.

Appendix A

Test Pit Logs and Laboratory Data

APPENDIX

FIELD EXPLORATIONS AND LABORATORY TESTING

FIELD EXPLORATION

Subsurface conditions at the site were explored by excavating five (5) exploratory test holes on August 20, 2022. The approximate location of the test hole explorations are shown in relation to the proposed and/or existing site improvements on the Site Exploration Plan, Figure No. 2.

The test holes 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 holes were excavated to a depth of between four (4) and seven (7) feet beneath existing site grades. Detailed logs of the test holes are presented on the Log of Test Pits, Figure No's. A-5 through A-7. 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 a continuous log 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 holes at the time of excavating to depths of up to seven (7) feet beneath existing site 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 density and optimum moisture content, gradational characteristics and Atterberg Limits as well as 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 hole exploration 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 log 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. The test was conducted to help establish various engineering properties for use as structural fill materials at the site. The test results are shown on Figure No. A-8.

Gradation Analysis

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

Atterberg Limits

One (1) Liquid Limit (LL) and Plastic Limit (PL) test was performed on a representative sample of the sandy, clayey 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-10.

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-11.

"R"-Value Tests

One (1) "R"-value test was performed on a remolded subgrade soil sample 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-12.

The following figures are attached and complete the Appendix:

Figure No. A-4	Key To Exploratory Boring Logs
Figure No's. A-5 through A-7	Log of Test Pits
Figure No. A-8	Maximum Dry Density Test Results
Figure No. A-9	Gradation Test Results
Figure No. A-10	Atterberg Limits Test Results
Figure No. A-11	Consolidation Test Results
Figure No. A-12	"R" Resistance 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 — CLEAR SQUARE SIEVE OPENINGS

	200		40	10	4	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
PO BOX 20547 • PORTLAND, OREGON 97294

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

NEWBERG APARTMENTS SITE
TL 800, EAST HAWORTH AVENUE

PROJECT NO.	DATE	Figure A-4
!@!#.)@).G	11/09/22	

BACKHOE COMPANY: Blackhorn Excavation BUCKET SIZE: 24 inches DATE: 10/20/22

SOIL DESCRIPTION

TEST PIT NO. TH-#1 ELEVATION 205'±

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			21.1	ML	Medium to olive-brown with gray mottling, very moist, medium stiff, sandy, clayey SILT
5	X			20.6		
						Total Depth = 6.0 feet No groundwater encountered at time of exploration
10						
15						

TEST PIT NO. TH-#2 ELEVATION 207'±

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 olive-brown with gray mottling, very moist, medium stiff, sandy, clayey SILT
5						
						Total Depth = 7.0 feet No groundwater encountered at time of exploration
10						
15						

LOG OF TEST PITS

PROJECT NO. 1213.020.G | NEWBERG APARTMENTS SITE | FIGURE NO. A-5

BACKHOE COMPANY: Blackhorn Excavation BUCKET SIZE: 24 inches DATE: 10/20/22

TEST PIT NO. TH-#3 ELEVATION 207'±

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			20.7	ML	Medium to olive-brown with gray mottling, very moist, medium stiff, sandy, clayey SILT
5	X			19.8		
						Total Depth = 6.0 feet No groundwater encountered at time of exploration
10						
15						

TEST PIT NO. TH-#4 ELEVATION 206'±

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			19.8	ML	Medium to olive-brown with gray mottling, very moist, medium stiff, sandy, clayey SILT
5						
						Total Depth = 7.0 feet No groundwater encountered at time of exploration
10						
15						

LOG OF TEST PITS

PROJECT NO. 1213.020.G NEWBERG APARTMENTS SITE FIGURE NO. A-6

REDMOND GEOTECHNICAL SERVICES

BACKHOE COMPANY: Blackhorn Excavation BUCKET SIZE: 24 inches DATE: 10/20/22

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	SOIL DESCRIPTION TEST PIT NO. TH-#5 ELEVATION 205'±
0					ML	FILL" Medium to olive-brown, very moist to wet, medium stiff (moderately Compacted), sandy, clayey SILT
4.0						Total Depth = 4.0 feet No groundwater encountered at time of exploration
5						
10						
15						

DEPTH (FEET)	BAG SAMPLE	DENSITY TEST	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	SOIL CLASS. (U.S.C.S.)	TEST PIT NO. ELEVATION
0						
5						
10						
15						

LOG OF TEST PITS

PROJECT NO. 1213.020.G | NEWBERG APARTMENTS SITE | FIGURE NO. A-7

MAXIMUM DENSITY TEST RESULTS

SAMPLE LOCATION	SOIL DESCRIPTION	MAXIMUM DRY DENSITY (pcf)	OPTIMUM MOISTURE CONTENT (%)
TH-#3 @ 2.5'	Medium to olive-brown, sandy, clayey, SILT (ML)	108.0	15.0

EXPANSION INDEX TEST RESULTS

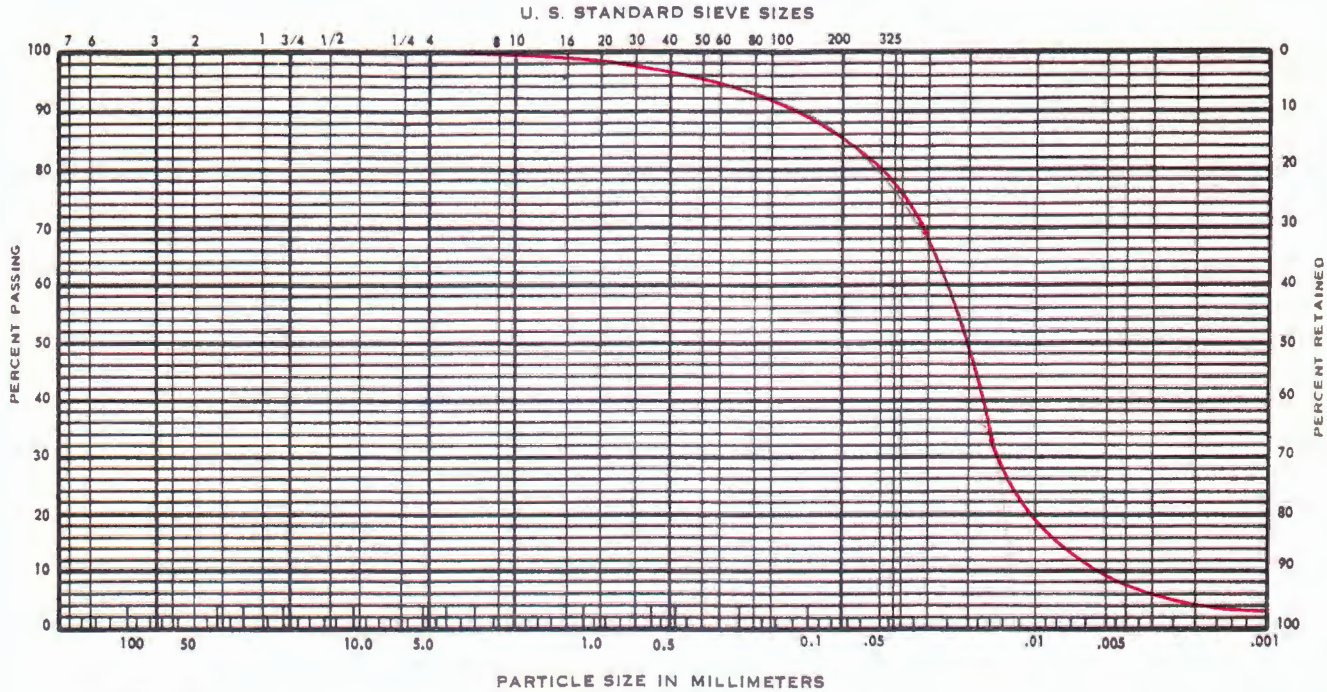
SAMPLE LOCATION	INITIAL MOISTURE (%)	COMPACTED DRY DENSITY (pcf)	FINAL MOISTURE (%)	VOLUMETRIC SWELL (%)	EXPANSION INDEX	EXPANSIVE CLASS.

MAXIMUM DENSITY & EXPANSION INDEX TEST RESULTS

PROJECT NO.: 1213.020.G | NEWBERG APARTMENTS SITE | FIGURE NO.: A-8

UNIFIED SOIL CLASSIFICATION SYSTEM

(ASTM D 422-72)



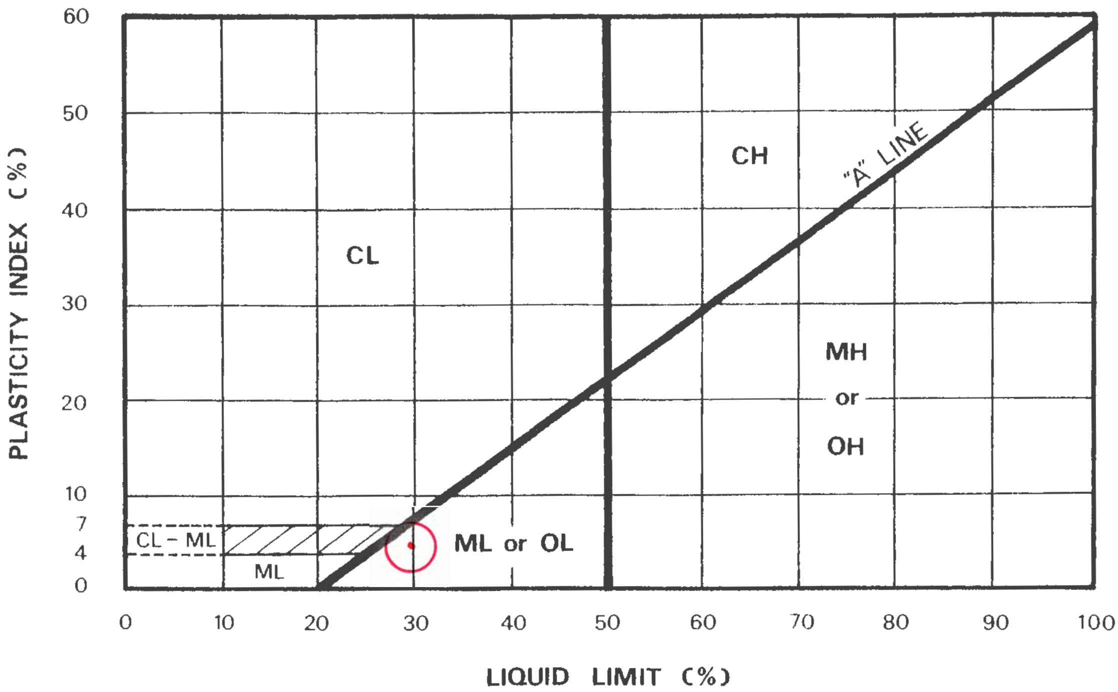
COBBLES	GRAVEL		SAND			SILT AND CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	

KEY SYMBOL	BORING NO.	SAMPLE DEPTH (feet)	ELEV. (feet)	UNIFIED SOIL CLASSIFICATION SYMBOL	SAMPLE DESCRIPTION
——	TH-#3	2.5		ML	Medium to olive-brown, sandy, clayey SILT

REDMOND GEOTECHNICAL SERVICES

PO Box 20547 • Portland, Oregon 97294

GRADATION TEST DATA		
NEWBERG APARTMENTS SITE TL 800, EAST HAWORTH AVENUE		
PROJECT NO.	DATE	FIGURE A-9
1213.020.G	11/09/22	



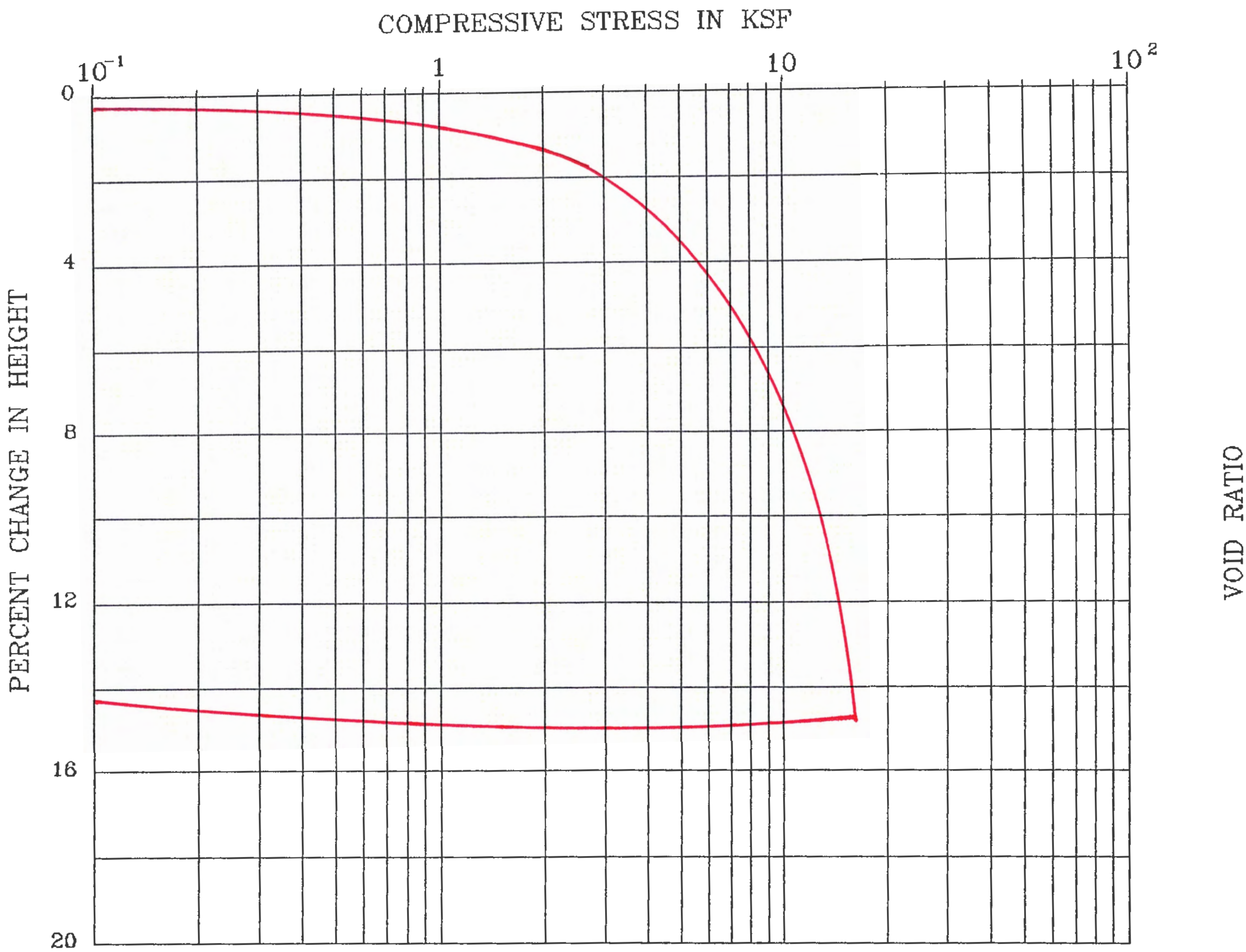
KEY SYMBOL	BORING NO.	SAMPLE DEPTH (feet)	NATURAL WATER CONTENT %	LIQUID LIMIT %	PLASTICITY INDEX %	PASSING NO. 200 SIEVE %	LIQUIDITY INDEX	UNIFIED SOIL CLASSIFICATION SYMBOL
⊙	TH-#3	2.5	20.7	29.9	5.3	85.3		ML

REDMOND GEOTECHNICAL SERVICES
PO Box 20547 • Portland, Oregon 97294

PLASTICITY CHART AND DATA

NEWBERG APARTMENTS SITE
TL 800, east haworth avenue

PROJECT NO.	DATE	
1213.020.G	11/09/22	Figure A-10



BORING : TH-#3
DEPTH (ft) : 2.5
SPEC. GRAVITY : 2.5 (assumed)

DESCRIPTION : sandy, clayey SILT (ML)
LIQUID LIMIT : 29.9
PLASTIC LIMIT : 24.6

	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	PERCENT SATURATION	VOID RATIO
INITIAL	20.7	88.4	81.1	
FINAL	13.3	107.7	93.6	

REDMOND GEOTECHNICAL SERVICES
PO Box 20547 • Portland, Oregon 97294

CONSOLIDATION TEST DATA

NEWBERG APARTMENTS SITE
TL 800, EAST HAWORTH AVENUE

PROJECT NO.	DATE	
1213.020.G	11/09/22	Figure A-11

RESULTS OF R (RESISTANCE) VALUE TESTS

SAMPLE LOCATION: TH-#4

SAMPLE DEPTH: 2.0 feet bgs

Specimen	A	B	C
Exudation Pressure (psi)	214	328	436
Expansion Dial (o.ooo1")	0	0	1
Expansion Pressure (psf)	0	0	3
Moisture Content (%)	20.6	16.4	12.1
Dry Density (pcf)	103.7	107.2	111.5
Resistance Value, "R"	18	33	45
"R"-Value at 300 psi Exudation Pressure = 32			

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-12