Harper
Houf Peterson
Righellis Inc.

Virginia Garcia - Newberg

SEA-142

Preliminary Stormwater Management Report

November 21, 2022

Prepared For:

Scott Edwards Architecture
2525 E Burnside St.
Portland, OR 97214

SEA-142

Prepared By:

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Alex Simpson, PE

HHPR

ENGINEERS ◆ PLANNERS
LANDSCAPE ARCHITECTS ◆ SURVEYORS

Stormwater Management Report
Virginia Garcia Newberg

Prepared by: Harper Houf Peterson Righellis, Inc.
Date: November 21, 2022

Project Overview and Description:

The Virginia Garcia - Newberg project is located at 2251 E Hancock St. in Newberg, OR. The total project encompasses two separate lots. Tax lot 500 (map 3220AB) is 0.95 acres, and Tax lot 702 is 1 acre. The site is bordered to the south by E Hancock St., west, east, and north by private property. The proposed project will consist of renovations and expansion of the existing building on TL 500, and reconstruction of the north lot TL 702 to be used for parking. Stormwater management for the project will conform to the 2015 City of Newberg Engineering Design and Construction Standards.

Existing Site Conditions & Facilities

The existing south site is developed with a building, parking and landscaping. The north lot is developed with asphalt pavement and a shed structure. The site generally slopes from the north toward the south. There are existing storm sewers throughout the site that connect to the public storm system in E Hancock St.

Proposed Site Conditions

The proposed improvements will consist of construction of a new addition to the existing building on the south lot. The north lot will be reconfigured and redeveloped to provide additional parking area. Both sites will have improvements to landscaping and hardscaping. The project will result in a net decrease in impervious area on both sites for a total decrease of -9,390 SF of impervious area. The site's stormwater system will be reconstructed and connect to the existing public storm system in E Hancock St.

Methodology & Analysis:

The project will result in a net decrease in impervious area on both sites for a total decrease of -9,390 SF of impervious area. Per Chapter 4.6 and Figure 4.4 of the Newberg Design and Construction Standards, since the project results in a net decrease in impervious area, formal stormwater management is not required. A downstream analysis has not been completed, and no downstream deficiencies are known.

	Existing Impervious Area	Proposed Impervious Area	Net Change in Impervious Area
Main (south) Lot	34,530 SF	32,780 SF	-1,750 SF
North Lot	37,680 SF	30,040 SF	-7,640 SF
TOTAL	72,210 SF	62,820 SF	-9,390 SF

Conveyance

The proposed storm pipe system is designed to have the capacity to convey the runoff from a 10-year return frequency storm event without ponding. The site storm system was designed to convey all of the impervious area and contributing pervious areas for the entire site.

The intent is to maintain a minimum free flow velocity of 3.0 fps in all pipes. See the Appendix for pipe sizing calculations (minimum pipe slopes & sizes required to meet these conditions).

Engineering Conclusions:

The proposed development has appropriate stormwater facilities and a system that fulfills the required conveyance, water quality and water quantity based on City of Newberg requirements and standards.

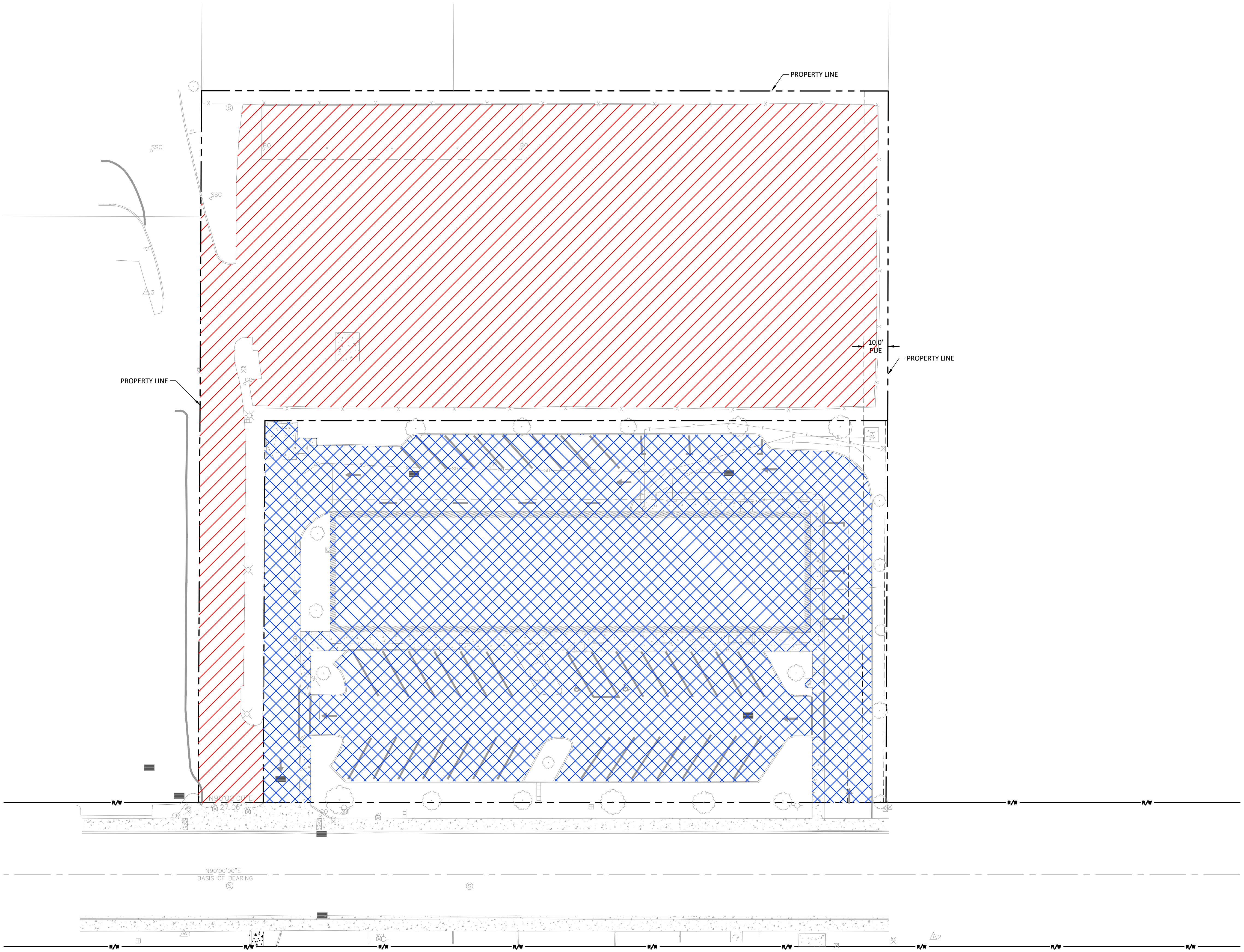
BASIN MAP

MAIN SITE (VIRGINIA GARCIA OWNED PARCEL):

EXISTING IMPERVIOUS AREA: 34,530 SF

NORTH SITE (GOODWILL OWNED):

EXISTING IMPERVIOUS AREA: 37,680 SF



LEGEND

NORTH IMPERVIOUS AREA

SOUTH IMPERVIOUS AREA

PLAN VIEW

SCALE: 1" = 20'

PRE-DEVELOPED IMPERVIOUS AREA

HHPR Harper Houf Peterson Righellis Inc.

0 10 20 40

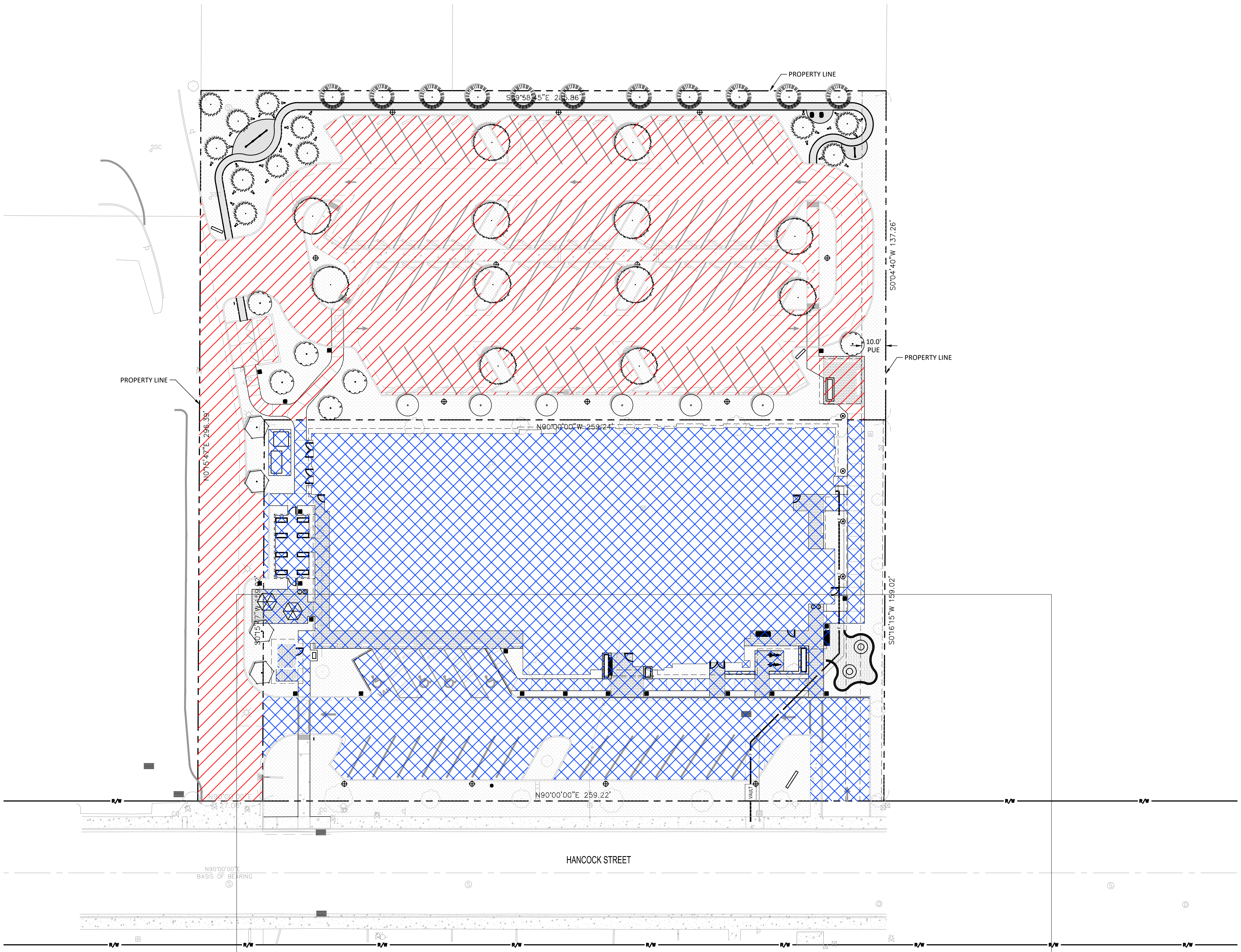
SCALE: 1" = 20'

VIRGINIA GARCIA CLINIC

2251 E. HANCOCK, NEWBERG, OR 97132

21162 05/10/22

Scott Edwards Architecture



MAIN SITE (VIRGINIA GARCIA OWNED PARCEL):

PROPOSED IMPERVIOUS AREA: 32,780 SF

NET IMPERVIOUS AREA (POST-DEVELOPED - PRE-DEVELOPED): 32,780 SF - 34,530 SF = -1,750 SF

NORTH SITE (GOODWILL OWNED):

PROPOSED IMPERVIOUS AREA: 30,040 SF

NET IMPERVIOUS AREA (POST-DEVELOPED - PRE-DEVELOPED): 30,040 SF - 37,680 SF = -7,640 SF

OVERALL PROJECT:

NET IMPERVIOUS AREA: -9,390 SF

LEGEND

NORTH IMPERVIOUS AREA

SOUTH IMPERVIOUS AREA

PLAN VIEW
SCALE: 1" = 20'

POST-DEVELOPED IMPERVIOUS AREA

VIRGINIA GARCIA NEWBERG CLINIC
21162 07/22/22

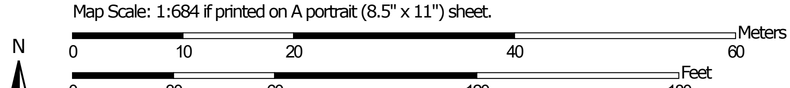
0 10 20 40
SCALE: 1" = 20'

Harper Houf Peterson Righellis Inc.

Scott Edwards Architecture

WEB SOIL SURVEY

Map Scale: 1:684 if printed on A portrait (8.5" x 11") sheet.



Map projection: Web Mercator Corner coordinates: WGS84 Edge tics: UTM Zone 10N WGS84

MAP LEGEND

Area of Interest (AOI)
- Area of Interest (AOI)

Soils
- Soil Map Unit Polygons
- Soil Map Unit Lines
- Soil Map Unit Points

Special Point Features
- Blowout
- Borrow Pit
- Clay Spot
- Closed Depression
- Gravel Pit
- Gravelly Spot
- Landfill
- Lava Flow
- Marsh or swamp
- Mine or Quarry
- Miscellaneous Water
- Perennial Water
- Rock Outcrop
- Saline Spot
- Sandy Spot
- Severely Eroded Spot
- Sinkhole
- Slide or Slip
- Sodic Spot
- Spoil Area
- Stony Spot
- Very Stony Spot
- Wet Spot
- Other
- Special Line Features

Water Features
- Streams and Canals

Transportation
- Rails
- Interstate Highways
- US Routes
- Major Roads
- Local Roads

Background
- Aerial Photography

MAP INFORMATION

The soil surveys that comprise your AOI were mapped at 1:24,000.

Warning: Soil Map may not be valid at this scale.

Enlargement of maps beyond the scale of mapping can cause misunderstanding of the detail of mapping and accuracy of soil line placement. The maps do not show the small areas of contrasting soils that could have been shown at a more detailed scale.

Please rely on the bar scale on each map sheet for map measurements.

Source of Map: Natural Resources Conservation Service
Web Soil Survey URL:
Coordinate System: Web Mercator (EPSG:3857)

Maps from the Web Soil Survey are based on the Web Mercator projection, which preserves direction and shape but distorts distance and area. A projection that preserves area, such as the Albers equal-area conic projection, should be used if more accurate calculations of distance or area are required.

This product is generated from the USDA-NRCS certified data as of the version date(s) listed below.

Soil Survey Area: Yamhill County, Oregon
Survey Area Data: Version 11, Sep 14, 2022

Soil map units are labeled (as space allows) for map scales 1:50,000 or larger.

Date(s) aerial images were photographed: Aug 19, 2015—Sep 13, 2016

The orthophoto or other base map on which the soil lines were compiled and digitized probably differs from the background imagery displayed on these maps. As a result, some minor shifting of map unit boundaries may be evident.

Map Unit Legend

Map Unit Symbol	Map Unit Name	Acres in AOI	Percent of AOI
2300A	Aloha silt loam, 0 to 3 percent slopes	2.0	100.0%
Totals for Area of Interest		**2.0**	**100.0%**

Yamhill County, Oregon

2300A—Aloha silt loam, 0 to 3 percent slopes

Map Unit Setting

National map unit symbol: 1j8b0
Elevation: 100 to 350 feet
Mean annual precipitation: 40 to 50 inches
Mean annual air temperature: 50 to 54 degrees F
Frost-free period: 165 to 210 days
Farmland classification: Prime farmland if drained

Map Unit Composition

Aloha and similar soils: 96 percent
Minor components: 4 percent
Estimates are based on observations, descriptions, and transects of the mapunit.

Description of Aloha

Setting

Landform: Terraces
Landform position (three-dimensional): Tread
Down-slope shape: Linear
Across-slope shape: Convex
Parent material: Loamy glaciolacustrine deposits

Typical profile

Ap - 0 to 8 inches: silt loam
BA - 8 to 15 inches: silt loam
Bt - 15 to 22 inches: silt loam
Bw1 - 22 to 31 inches: silt loam
Bw2 - 31 to 46 inches: silt loam
Bw3 - 46 to 60 inches: silt loam
C - 60 to 65 inches: very fine sandy loam

Properties and qualities

Slope: 0 to 3 percent
Depth to restrictive feature: More than 80 inches
Drainage class: Somewhat poorly drained
Capacity of the most limiting layer to transmit water (Ksat): Moderately high (0.20 to 0.57 in/hr)
Depth to water table: About 8 to 15 inches
Frequency of flooding: None
Frequency of ponding: None
Available water supply, 0 to 60 inches: Very high (about 12.0 inches)

Interpretive groups

Land capability classification (irrigated): 2w
Land capability classification (nonirrigated): 2w
Hydrologic Soil Group: C/D

Ecological site: R002XC007OR - Valley Swale Group
Forage suitability group: Somewhat Poorly Drained (G002XY005OR)
Other vegetative classification: Somewhat Poorly Drained (G002XY005OR)
Hydric soil rating: No

Minor Components

Dayton

Percent of map unit: 3 percent
Landform: Terraces
Landform position (three-dimensional): Tread
Down-slope shape: Linear
Across-slope shape: Concave
Hydric soil rating: Yes

Willamette

Percent of map unit: 1 percent
Landform: Terraces
Landform position (three-dimensional): Tread
Down-slope shape: Linear
Across-slope shape: Convex
Other vegetative classification: Well drained < 15% Slopes (G002XY002OR)
Hydric soil rating: No

Data Source Information

Soil Survey Area: Yamhill County, Oregon
Survey Area Data: Version 11, Sep 14, 2022

GEOTECHNICAL REPORT

CENTRAL
GEOTECHNICAL SERVICES, LLC

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July 21, 2022

Virginia Garcia Memorial Health Center
PO Box 6149
Aloha, Oregon 97007

Attention: Jarrod Sherwood (jsherwood@vgmhc.org)
Brian Jackson (brian@bcjarchitect.com)

Re: Geotechnical Investigation **CGS Project No. 22-025**
Proposed Building Addition
Virginia Garcia Memorial Health Center – Newberg Clinic
2251 E Hancock St #103
Newberg, Oregon 97132

Dear Mr. Sherwood,

Central Geotechnical Services, LLC is pleased to submit this Geotechnical Investigation Report for a proposed addition to the existing Virginia Garcia Memorial Health Center – Newberg Clinic located at 2251 E Hancock Street in Newberg, Oregon. The report was prepared for conformance with Yamhill County requirements and in accordance with our Professional Services Agreement, dated February 7, 2022.

This report provides:

- An overview of the project site including information related to the regional geology of the area.
- Geotechnical information, based on findings during surface reconnaissance and subsurface explorations.
- General construction recommendations.
- Recommendations for additional work as needed.

A set of appendices can be found at the end of this document.

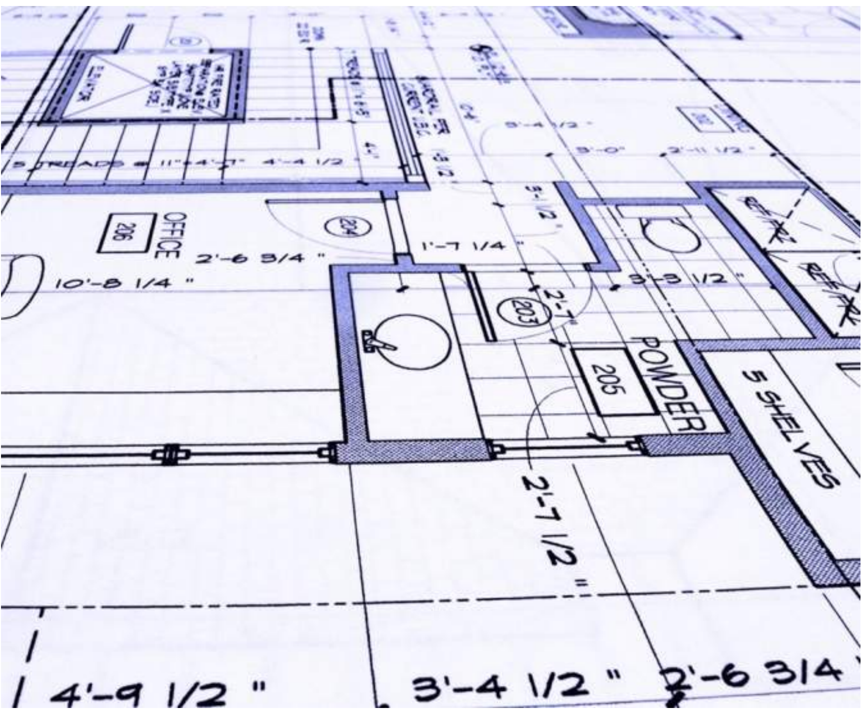
Thank you very much for the opportunity to work with you. If you feel obliged, we welcome referrals from our previous clients and would enjoy the opportunity to work with others in your professional and personal networks. Please feel free to call our office with questions about this report.

Respectfully,

Central Geotechnical Services, LLC

____

Jose Serrano, P.E.
Associate Engineer



Virginia Garcia Memorial
HEALTH CENTER

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1.0 INTRODUCTION AND PURPOSE OF REPORT

The purpose of this Geotechnical Investigation Report is to engage with the owner/developer and provide technical insight and analysis for the project, based on various public data, local findings onsite, and experience.

After receiving direction from Jarrod Sherwood of Virginia Garcia Memorial Health Center, Central Geotechnical Services (Central Geotech) was requested to provide a Geotechnical Investigation, along with general recommendations for the design and construction of the proposed addition to the existing building. The construction recommendations cover topics such as investigative soils data, allowable soil bearing pressure, lateral pressures, compaction requirements, foundation placement, pavement, and seismic considerations.

This report intends to facilitate the preliminary focus of future development and initiate the requirements for the design and permitting of the proposed building addition.

1.1 Project Description

Based on phone discussions and review of documentation sent by you, we understand that the project involves the expansion of the existing building to accommodate additional medical and dental offices. The project will include an approximate 9,000 square foot, one-story building addition primarily to the north of the existing building, underground utilities, paved parking, and other associated features.

Structural plans have not been provided to our office at the time of this report. We presume the building addition will be supported on shallow, spread footings with a concrete slab-on-grade floor. The concrete slab may include thickened edge slab footings for interior walls and columns. Structural loading information is not available at this time; however, we expect that column loads will be less than 35 kips and wall loads less than 3 kips per lineal foot. Expected loads on the concrete slab-on-grade floor will be relatively moderate, with some forklifts and limited heavy equipment operated on the slab surface.

The pavement areas will include driveways and parking for passenger vehicles, light trucks, fire department apparatus access, and new parking stalls. The parking lot improvements may extend to the adjacent property on the north.

We were provided with the following documents:

1. Due Diligence Report prepared for the Virginia Garcia Memorial Health Center Newberg Clinic, prepared by Edwards Architecture LLP, dated January 28, 2022.
2. Topographical Survey showing existing conditions, prepared by Terracalc Land Surveying Inc., dated May 2, 2022. The area for borings and proposed addition was outlined in the plan by Brian C. Jackson Architect LLC.
3. Schematic Design showing the proposed addition, prepared by Scott Edwards Architecture, dated June 2, 2022.

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2.0 INVESTIGATION SUMMARY

2.1 Site Location and Surface Conditions

The project site is located at 2251 E Hancock St. #130 in Newberg, Yamhill County, Oregon. The 0.94-acre lot is identified as Yamhill County Map Tax Lot R3220AB 00500 and is zoned as C2 (Community Commercial). The property is bordered by paved parking lots owned by Goodwill Industries to the north and west, E Hancock Street to the west, and a small field to the east. A vicinity map of the site is shown in Figure 2-1, below.

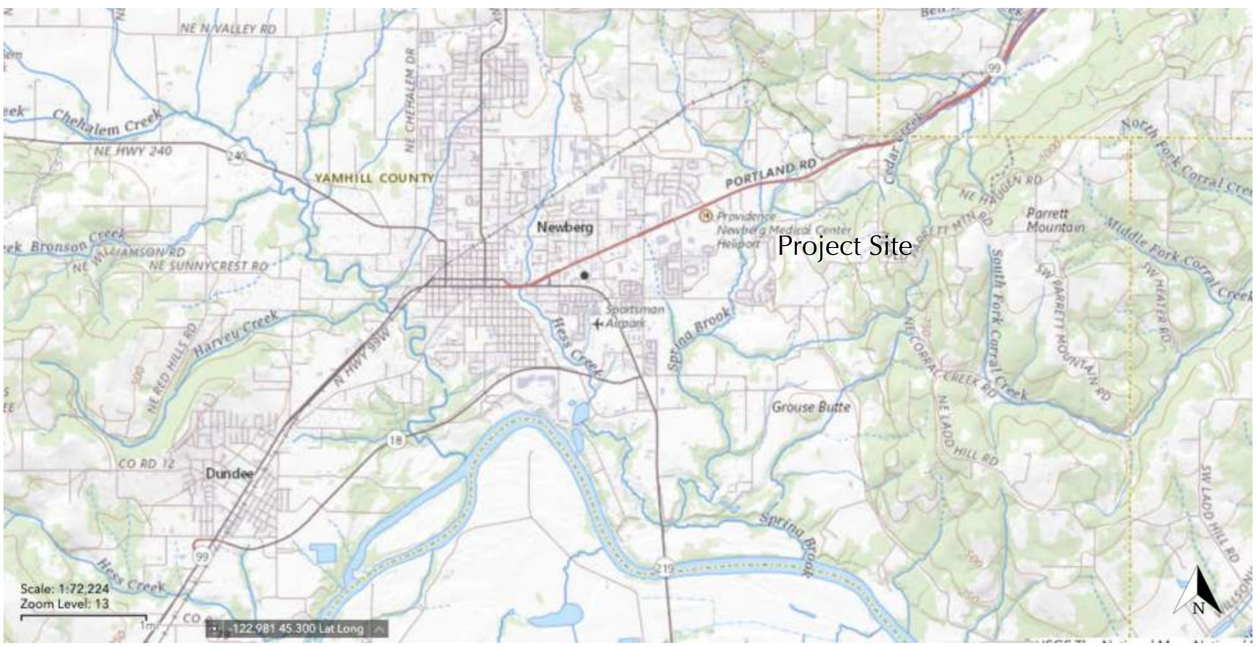

Figure 2-1: Area map of project site (Source: USGS National Map)

The site is located on a broad alluvial plain with gentle topography at an approximate elevation of 194 feet above sea level.

The site is developed with an existing building located at the center of the property, trash enclosure, paved parking lot and landscaping areas. The existing building is approximately 9,878 square feet on the first floor and 2,856 square feet on the second floor.

To understand how the site conditions may have changed over time, we reviewed historical aerial photographs from 1994, and 2000 through 2020 available on Google Earth Pro[1]. Based on a review of these photographs, the existing building was constructed between 1994 and 2000.

[1] Google Earth Pro, last updated in 2021.

The general topography in the site vicinity is shown in Figure 2-2, below.

Figure 2-2: Topography in site vicinity. Contour interval is 10 feet.
(Source: USGS National Map)

2.2 Site Geology

The site is located on the western margin of the Willamette Basin, a structural basin filled with a thick sequence of sedimentary strata. Basin sediment near the ground surface was deposited during repeated glacial outburst flooding of the Columbia River and its tributaries, known as the Missoula Floods. The Missoula Flood Deposit in the Willamette Basin is primarily clay, silt and fine sand deposited in short-lived floodwater lakes and as drifts of windblown silt known as loess[2]. The last flooding event occurred at the end of the last glacial period about 9,000 to 10,000 years ago[3]. Regional geologic mapping shows

[2] Madin, I.P., 1990, Earthquake Hazard Geology Maps of the Portland Metropolitan Area, Oregon; Oregon Department of Geology and Mineral Industries, Open File Report O-90-02, map scale 1:24,000.

[3] Waitt, R. B. Jr., 1985, Case for Periodic Colossal Jokulhlaups from Pleistocene Lake Missoula; Geological Society of America Bulletin, v. 96, no. 10, p. 1271-1286.

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the Missoula Flood Deposit in the site vicinity to be about 80 feet thick[4]. Underlying the Missoula Flood Deposit is a sequence of Holocene age marine sedimentary strata.

2.3 Tectonic and Seismic Setting

The Willamette Basin is subject to seismic events stemming from three possible sources: the Cascadia Subduction Zone (CSZ) at the interface between the Juan de Fuca Plate and the North American Plate, intraslab faults within the Juan de Fuca Plate, and crustal faults in the North American Plate.

The CSZ is seismically active. Intraslab events with inland epicenters, such as the 6.8 M_W Nisqually earthquake in 2001, have occurred on a frequent basis in the Puget Sound, contributing small to moderate magnitude ground motions in southern Washington. The maximum magnitude for a CSZ interface event is expected to be in the range of moment magnitude (M_W) 9.0 with an offshore epicenter located about 100 miles west of the project site.

Quaternary age (last 1.6 million years) crustal faults inventoried in the USGS National Fault and Fold Database that lie within 10 miles of the site are the Newberg Fault about 0.8 mile to the southwest, and Gales creek fault zone about 7.7 miles to the northwest.

The contribution of potential earthquake-induced ground motion from all known sources, including the faults described above, are provided by the seismic design parameters for the project site presented in the recommendations section of this report.

2.4 Liquefaction Hazard

Strong seismic shaking can result in ground failure due to the phenomenon known as liquefaction. Soil liquefaction occurs when saturated soil temporarily loses strength and behaves as a fluid in response to seismic shaking. Liquefaction is generally limited to loose, granular, cohesionless soil located below a shallow water table. Various types of ground deformation can occur including but not limited to slope movement, lateral spreading, sand boils, settlement, ground oscillation, and cracking.

A regional assessment of the susceptibility of the subsurface soils to earthquake-induced liquefaction was prepared by the Oregon Department of Geology and Mineral Industries (DOGAMI) in 1999[5]. The site is mapped as being located in an area with a low liquefaction hazard.

2.5 Subsurface Exploration

We completed a program of subsurface exploration at the site that included drilling of three geotechnical borings (B-1 through B-3) to a depth of about 26.5 feet below site grade on June 22, 2022. The borings

[4]Gannett, M.W. and Caldwell, R.R., 1998, Geologic Framework of the Willamette Lowland Aquifer System, Oregon and Washington; U.S. Geological Survey, Professional Paper 1424-A, map scale 1:250,000, 32 pages.
[5] Madin, I. P., Wang, Z., 1999, Relative Earthquake Hazard Maps for Selected Urban Areas in Western Oregon Dallas, Hood River McMinnville-Dayton-Lafayett, Monmounth-Independence, Newberg-Dundee, Sandy, Sheridan-Willamina, St-Helens-Columbia City-Scappoose, Oregon Department of Geology and Mineral Industries, Interpretive Map Series IMS-7

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were completed using a trailer-mounted drill rig operated by Dan Fischer Excavating of Forest Grove Oregon using solid stem auger techniques. Soil samples were obtained at selected depths. The approximate locations of the borings are presented in Figure 2-3, below.

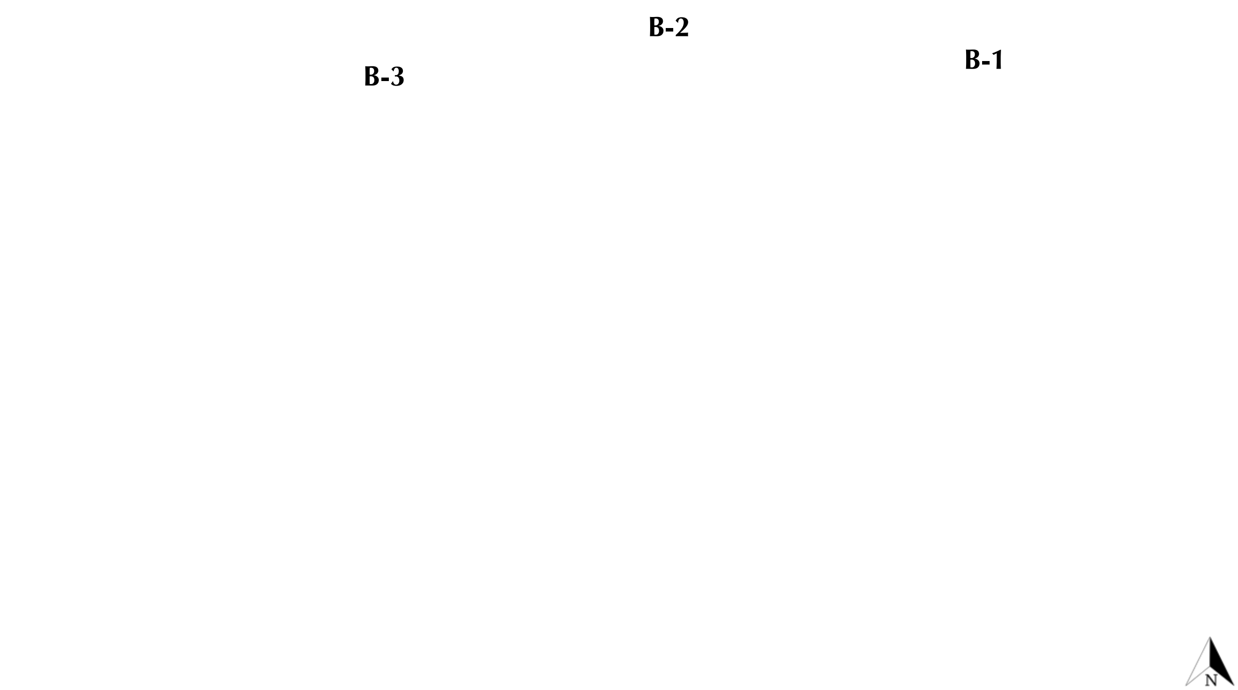

Figure 2-3: Site plan showing the approximate location of explorations. All Locations are approximate. (Source: Topographical Survey by Terracalc Land Surveying, Inc., dated May 2, 2022)

The soil and groundwater conditions encountered are described in the following sections. Field exploration procedures, summary logs of the explorations, and Soil Classification and Description Guidelines are presented in Appendix A.

2.6 Subsurface Conditions

We encountered two soil layers on the site: an upper layer of fill, and a lower layer of Missoula Flood Deposit. Each of the layers is described below.

2.6.1 Fill

The borings were located in a paved area with pavement thicknesses ranging from 3 to 7 inches of asphaltic concrete (AC) over 5 to 9 inches of aggregate base. Below the pavement section, we encountered a thin layer of fill in all exploratory borings that extended to depth of about 2 feet bgs. In general, the fill consisted of soft, brown, SILT (ML), with trace gravel.

2.6.2 Missoula Flood Deposit

Beneath the fill, we encountered SILT (ML) and fat CLAY (CH) belonging to the Missoula Flood Deposit. The SILT (ML) deposit extended to depths of 17 to 18 feet bgs. Based on correlations between standard penetration resistance and soil strength, the SILT was generally soft to medium-stiff, with N-values ranging between 3 and 9. An Atterberg Limits test result from B-1 at a depth of about 10 feet indicate a soil classification of SILT (ML) with a Liquid Limit of 45, a Plastic Limit of 28 and a Plasticity Index of 17.

The underlying fat CLAY (CH) deposit extended to the maximum depth explored of 26.5 feet. Based on correlations between standard penetration resistance and soil strength, the fat CLAY was generally medium-stiff to stiff, with N-values ranging between 5 and 11. An Atterberg Limits test result from B-3 at a depth of about 20 feet indicate a soil classification of fat CLAY (CH) with a Liquid Limit of 69, a Plastic Limit 27 and a Plasticity Index 42.

The results of the laboratory testing are shown on the boring logs and in Appendix B.

2.7 Groundwater Conditions

We encountered groundwater in all three of our exploratory borings at depths of 6.4, 7.3, and 7.9 feet bgs on June 22, 2022.

The observed groundwater conditions are specific to the locations of our explorations as well as the time of our exploration. Groundwater levels typically fluctuate and are generally higher (at shallower depths) during the wet season (October through June).

We expect that temporary perched groundwater conditions occur near to the ground surface during the wet-weather season in response to heavy rainfall events, due to the presence of low permeability clayey soil.

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3.0 CONCLUSIONS AND RECOMMENDATIONS

Based on the results of our geotechnical investigation, we consider the proposed building addition to be geotechnically feasible with shallow foundations designed and constructed in accordance with our recommendations.

CGS should observe the foundation excavation subgrade prior to placing structural fill, forms, or reinforcing bar to evaluate subgrade support conditions are as expected and perform construction observation and testing to evaluate compaction of engineered structural fill.

The following sections present our conclusions and recommendations for project design and construction.

3.1 Site Preparation and Removal of Existing Fill

Existing undocumented fill should be removed from planned structural areas. Based on our explorations, we expect the depth of excavation to remove undocumented fill will be about 2 feet below the existing ground surface.

Subsurface structures, such as existing footings and abandoned utilities, should be removed and the excavations backfilled utilizing only an approved granular material, placed and compacted in accordance with *Section 3.4 Engineered Structural Fill*. Removal of undocumented backfill adjacent to demolished and excavated structures is recommended. Excavation should include benching of the side walls so that backfill can be properly placed.

In preparation for construction, the existing asphalt should be removed from future structural areas. In vegetated areas, mulch and the heavily rooted topsoil zone should be stripped and removed from the site in all proposed structural areas and for a minimum 2-foot margin around such areas. Based on our explorations, the minimum depth of stripping will be approximately 12 inches. Greater removal depths may necessary in isolated areas to remove tree stumps, root balls, excessively rooted zones, or undocumented fills. Stripped material should be transported off-site for disposal or stockpiled for use in landscaping areas.

After stripping and the required site cutting have been completed, we recommend the areas be observed by a member of our geotechnical staff who will evaluate the subgrade by probing or other applicable means. Existing compacted fill beneath the asphalt may remain provided that CGS evaluates it to verify adequate compaction. The evaluation should be performed by proof-rolling with a loaded dump truck or similar vehicle, and should be observed by CGS. If soft areas are identified, the material should be excavated and replaced with compacted engineered structural fill.

It is possible that unrecognized areas of undocumented fill may be encountered on the site during construction. It is recommended that all uncontrolled fill soils be removed completely in preparation for foundations or other construction and be replaced with engineered structural fill in accordance with *Section 3.4 Engineered Structural Fill*.

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3.2 Temporary Excavations

The stability of temporary excavation slopes is a function of many factors, including soil type, soil density, slope inclination, slope height, the presence of groundwater, and the duration of exposure. Generally, the likelihood of slope failure increases as the cut is deepened and as the duration of exposure increases. For this reason, temporary slope safety should remain the responsibility of the contractor, who is continually present at the site and is able to monitor the performance of the excavation and modify construction practices to reflect varying conditions.

Regardless of inclination, temporary slopes should be protected from surface runoff of storm water. This can typically be accomplished using berms or swales located along the top of the slope, and by placing plastic tarpaulins over the slope.

We recommend that the excavation contractor maintain adequate slopes and setbacks in conformance with Occupational Safety and Health Administration (OSHA) Excavation Guidelines and all applicable regulations. Temporary cut slopes for the construction of basements or retaining walls should be limited to 1H:1V.

Excavations should not be made within the zone of influence of adjacent structures. Excavations within the influence zone of existing structures may require shoring, underpinning or other measures to provide temporary or permanent support.

3.3 Utility Trenches

Utility trench backfill in structural areas should consist of well-graded, granular fill limited to a maximum particle size of 1½ inches. Granular trench backfill should be compacted to at least 92% of the maximum dry density as determined by ASTM D1557. Excavator-mounted, vibratory-plate compactors are typically the most efficient for compaction of trench backfill. Lift thicknesses should be evaluated based on field density tests; however, care should be taken when operating vibratory compactors to prevent damage to pipes. An initial lift thickness over pipe may need to be up to 4 feet to protect the pipe from damage during compaction; however, thick lifts of loosely placed backfill should not be the standard practice for utility trench backfill. Native materials can be used for trench backfill in non-structural areas where a soft trench and future settlement of the backfill can be tolerated.

3.4 Engineered Structural Fill

Engineered structural fill is any fill material used for support of foundations, retaining walls, slab-on-grade floors, sidewalks, embankments, pavements, and similar features. The on-site soil is suitable for use as structural fill provided it can be separated from unsuitable material, be properly moisture conditioned, and compacted to the specified density as determined by standard testing in a soils lab. The on-site soil used as structural fill should be placed in lifts with a maximum uncompacted thickness of 8 inches.

Imported granular material should be used for engineered structural fill if the on-site material cannot be properly moisture conditioned. Imported granular fill should consist of crushed aggregate that is fairly

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well-graded between coarse and fine material and have less than 5 percent by weight passing the U.S. Standard No. 200 Sieve. Use of alternative granular fill material such as pit-run or quarry-run rock or sand should be evaluated for suitability by CGS prior to its use. Granular fill should be placed in lifts with a maximum uncompacted thickness of 6 inches.

All engineered structural fill should be compacted to at least 92% of the maximum dry density determined by the Modified Proctor ASTM D1557 or equivalent. CGS should perform density testing of engineered structural fill to verify that adequate compaction is achieved. Proof-rolling with a loaded dump truck or water truck may be allowed in certain circumstances under the guidance of CGS onsite to evaluate fill compaction.

Regardless of material or location, structural fill should be placed over firm, unyielding subgrade prepared in accordance with *Section 3.1 Site Preparation and Removal of Existing Fill* of this report. The condition of the subgrade should be verified by a CGS representative before filling or construction begins. Fill compaction should be verified by in-place density tests taken during fill placement to confirm that compaction meets project specifications.

3.5 Foundations

The proposed building addition may be supported on shallow, spread footings bearing on a minimum 12-inch-thick granular fill pad placed over competent subgrade soil. Foundation design, construction, and setbacks requirements should conform to the Oregon Structural Specialty Code (OSSC) and other governing codes as applicable.

We recommend an allowable soil bearing pressure of 1,500 pounds per square foot (psf) be used for footing design. The recommended allowable bearing pressure applies to the total of dead plus long-term live loads. The allowable bearing pressure may be increased by a factor of 1.33 for short-term loads such as those resulting from wind or seismic forces.

Total static settlement of footings founded as recommended is expected to be less than 1 inch. Differential settlement is estimated to be less than ¾ inches over a horizontal span of 20 feet. Most of the settlement will occur during construction as the loads are applied. These estimates are based on maximum wall loads of 3,000 pounds per lineal foot and a maximum column load of 35 kips. For heavier loads, CGS should be consulted.

For protection against frost heave and maximizing bearing strength, perimeter footings should be embedded at least 18 inches below exterior finish grade. Interior footings should be embedded at least 12 inches below floor slabs. Minimum footing widths should be determined by the project architect/designer/structural engineer in accordance with applicable design codes. Excavations adjacent to footings should not extend beneath a 1H:1V plane projected downwards from the bottom edge of the footing or be backfilled with engineered structural fill.

Footing excavations should be trimmed neat and carefully prepared. Loose, wet or otherwise softened subgrade should be removed from footing areas prior to placing crushed rock backfill, forms and reinforcing steel. In wet weather conditions, we recommend that a several-inch-thick layer of granular

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material (typically 3/4"-0 crushed aggregate) be placed at the base of footing excavations. The granular material reduces water softening of subgrade soils, reduces subgrade disturbance during placement of forms and reinforcement, and provides a clean environment for reinforcing steel. To be effective, the granular material should be placed on firm, well-drained subgrade and lightly compacted until well-keyed using a small vibratory plate compactor.

We recommended that CGS observe the foundation excavation subgrade prior to placing structural fill, formwork, or reinforcing steel to evaluate subgrade support conditions are within recommended specifications.

3.5.1 Lateral Resistance for Spread Footings

Lateral loads on the proposed structures imposed by wind or seismic forces can be resisted by a combination of sliding resistance on the base of footings and passive earth pressure on the sides of footings. We recommend an ultimate coefficient of friction of 0.35 for footings bearing on undisturbed, native soil, and 0.5 for footings bearing on granular engineered structural fill.

Passive earth pressures on the sides of buried spread footings may be calculated using an allowable equivalent fluid pressure of 300 pcf per foot of embedment. For this value, backfill against the footing should be compacted to at least 92% of the maximum dry density as obtained from ASTM D1557. The upper foot of embedment should be neglected unless protected by pavement or concrete slabs on grade.

3.6 Slab on Grade Floors

Satisfactory subgrade support for lightly-loaded building floor slabs can be obtained on undisturbed native soil or on newly placed structural fill. The modulus of subgrade reaction for design of floor slabs may be taken as 100 pounds per cubic inch.

A minimum 8-inch-thick layer of imported granular material should be placed and compacted over the prepared subgrade to assist as a capillary break and blanket drain. Imported granular material should consist of clean crushed rock or sand that is fairly well-graded between coarse and fine, contains no deleterious materials, has a maximum particle size of 1½ inches, and less than 5 percent by weight passing the U.S. Standard No. 200 Sieve. The imported granular material may be placed in one lift and should be compacted until well-keyed, about 85 percent of the maximum dry density as determined by ASTM D 1557. An underslab drainage pipe system is recommended for human occupancy areas with concrete slab floors.

A vapor retarder manufactured for use beneath floor slabs should be installed above the base rock and according to the manufacturer's recommendations. Careful attention should be made during construction to prevent perforating the retarder, and to seal edges and utility penetrations. We recommend following ACI 302.1, Chapter 3 with regard to installing a vapor retarder.

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3.7 Retaining Walls

The preliminary plans do not show any retaining walls for the project. Because the project is in the preliminary design phase, it is unclear whether structural retaining walls will be included. Lateral pressures presented in this report are to be considered as general guidelines, should retaining walls be included. CGS should be consulted for feature-specific recommendations.

The design engineer for the retaining wall must take into consideration the state at which the soil retention walls will be placed, whether under active, passive, or at-rest pressures. The possibility of additional non-seismic surcharge loading should also be considered.

Our recommended lateral earth pressures for design of retaining walls presented as equivalent fluid pressures are summarized in Table 3-1, below. Active and at-rest pressures should be modelled as a static triangular pressure profile with the resultant total force acting at one-third height of the exposed wall face. The recommended values are based on imported, free-draining granular backfill, a wet density of 135 pounds per cubic foot and a friction angle of 35 degrees for the retained soils. The tabulated design parameters are to be used for well-drained backfill conditions with no hydrostatic pressures behind the walls. Walls that may deflect by at least 0.01 times their height may be designed with active earth pressures. Walls that may not deflect should be designed with at-rest pressures.

Table 3-1 - Equivalent Fluid Pressure Acting on Retaining Walls

Wall Type	Backfill Slope	Backfill Equivalent Fluid Pressure (pcf)
Active (Yielding wall)	Level	35
	2H:1V	50
At-Rest (Non-yielding wall)	Level	50
	2H:1V	70

Passive earth pressures on retaining walls may be calculated using an allowable equivalent fluid pressure of 300 pcf per foot of embedment. For this value, backfill against the wall footing should be compacted to at least 92% of the maximum dry density of obtained from ASTM D1557. The upper foot of embedment should be neglected unless protected by pavement or concrete slabs on grade.

If the wall will be subjected to the influence of surcharge loading, the wall should be designed for an additional horizontal pressure. For uniform surcharge pressures, a uniformly distributed lateral pressure of 0.3 times the vertical surcharge pressure should be added. The influence zone of an applied vertical load is generally considered to be a 45-degree plane projected downward from the bottom edge of the footing. Traffic surcharges may be estimated using an additional vertical load of 250 psf (2 feet of additional fill), in accordance with local practice, or as determined by the type of traffic expected to apply the surcharge loads.

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It is difficult to accurately predict the additional lateral forces that will be generated on a retaining wall during an earthquake. Some factors affecting the magnitude of earthquake forces on the wall are the size and duration of the earthquake, the distance from the earthquake epicenter of the site, and the mass of soil retained by the wall. Retaining walls that are designed only for active earth pressures may fail when additional forces are generated by the earthquake.

A simple approach based on the work of Seed and Whitman (1970), is to include in the design analysis an additional horizontal force (P_E) to account for the additional loads imposed on the retaining wall by the earthquake (dynamic load)[6]. In this case, the static force is calculated and then an additional dynamic force (as shown below) is added to the wall for failure analysis.

$$P_E = \frac{3}{8}(0.5 * PGA_M)\gamma_t H^2$$

Where PGA_M = Peak Ground Acceleration (see Table 3-3)
γ_t = total unit weight of soil
H = height of retaining wall

The resultant of this equation is an ultimate value given in pounds per linear foot of wall. An adjustment factor selected by the structural engineer is typically applied to the ultimate value for structural design purposes. The location of this earthquake-induced force can be assumed to act at a distance of 0.6H up from the base of the wall.

Because P_E is a short-term loading that may never occur during the life of the retaining wall, it is common to allow a one third increase in the bearing pressure and passive resistance for the earthquake analysis. Also, for the analysis of sliding and overturning of the retaining wall, it is common to accept a lower factor of safety (1.1 to 1.2) under the combined static and earthquake loads.[7]

A layer of compacted aggregate that is a minimum of 1-foot-wide should be placed behind all retaining walls to allow for proper drainage and placed utilizing the compaction recommendations described in this report. All structural retaining walls should be backfilled with an imported, free-draining granular material such as ¾"-0 crushed rock with no more than 5% passing the No. 200 sieve. Only light-weight compaction equipment should be used immediately behind retaining walls, so that compactive effort does not damage the wall.

At the base of the retaining walls and continuous with the wall backfill aggregate, a wall subdrain should be installed to divert water from the retaining structures. The wall subdrain should consist of a 3- or 4-inch-diameter, perforated, gravity drainpipe (ADS Highway Grade or better) enveloped in at least 4 cubic feet per lineal foot of clean, drain rock. The drain rock should be wrapped within geotextile filter fabric with a minimum 1-foot overlap at joints to prevent fines from washing into the drain rock. A diagram of a typical wall subdrain can be found in Appendix C as a recommended guideline for construction.

[6] Seed, H.B. and Whitman, R.V., 1970, Design of Earth Retaining Structures for Dynamic Loads: ASCE Specialty Conference, Lateral Stresses in the Ground and Design of Earth Retaining Structures, Cornell University, Ithaca, New York, p. 103-147.

[7] Day, Robert W. "Geotechnical Engineer's Portable Handbook". Second Edition, 2012. Pg. 16.18, Table 16.5, Topic (1).

Retaining walls in living areas or other moisture sensitive areas should include water proofing and wall panel drains as specified by the wall designer.

3.8 Pavement Profiles

We do not have specific information on the frequency and type of vehicles that will use the development on a daily basis. For design purposes, we assumed that post-construction traffic will be primarily light duty passenger vehicles averaging no more than five heavy trucks per day. Our pavement recommendations are based on a typical subgrade density for silt using a California Bearing Ratio value of 3.

We recommend the minimum pavement section profiles presented in Table 3-2, below, to support the anticipated traffic loads over a design life of 20 years. For areas where service trucks back and turn, a Portland Cement Concrete (PCC) pavement section should be used, or the AC pavement thickness increased to 5 inches. The recommended minimum PCC section is 6 inches of PCC over 8 inches of 1½"-0 crushed rock compacted to at least 95% of ASTM D1557.

Table 3-2 - Recommended Minimum Dry-Weather Pavement Section

Material	Drive Aisles (inches)	Parking (inches)	Compaction Standard
Asphaltic Concrete (AC)	3	2.5	92% of Rice Density AASHTO T-209
Crushed Aggregate Base ¾"-0 (leveling course)	2	2	95% of Modified Proctor
Crushed Aggregate Base 1½ "-0	8	6	95% of Modified Proctor
Subgrade Soil	12	12	95% of Modified Proctor

These thicknesses are intended to be the minimum acceptable for construction completed during an extended period of dry weather. If pavement areas are constructed during wet weather, CGS should review the subgrade and proposed construction methods immediately prior to the placement of base course so that specific recommendations can be provided. Wet-weather pavement construction may require cement amendment or an additional 6 inches of crushed aggregate base.

AC pavement should conform to Section 00744 of the Standard Specification for Highway Construction, Oregon Highway Specifications, and Yamhill County requirements. We recommend graded half-inch or three-quarter inch, Dense Hot Mix Asphalt Concrete for Design Level 2 using Performance Grade

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Asphalt PG-64-22 which is appropriate for low to moderate volume pavements in Western Oregon. The aggregate base should conform to Section 02630 of the 2021 ODOT Oregon Standard Specifications for Construction with the addition that no more than 5 percent of the material by dry weight passes the U.S. Standard No. 200 Sieve. Aggregate base contaminated with soil during construction should be removed and replaced before paving.

As a matter of good construction practice, we recommend placing a woven separation fabric between the soil subgrade and the aggregate such as Contech C200 or US200. The fabric should conform to the minimum property values presented in Table 02320-4 – Subgrade Geotextile (Separation), in Section 02320 of the 2021 ODOT Oregon Standard Specifications for Construction.

We recommend that CGS conduct density testing and a proof roll performance test of the pavement subgrade prior to placement. Subgrade and base rock should be compacted to at least 95% of the maximum dry density obtained from ASTM D1557. Subgrade strength should be evaluated visually by proof-rolling directly on the subgrade with a loaded dump truck during dry weather and on top of base course in wet weather. Soft areas which rut, pump, or weave by more than ¼ inch should be stabilized prior to paving.

3.9 Drainage Considerations

Site drainage should include foundation drainage, surface runoff collection, and conveyance to a properly designed and permitted storm water drainage facility. As a matter of good construction practice, we recommend that perimeter footing subdrains be installed for all buildings. Perimeter subdrains should consist of perforated drainpipe enveloped in a zone of drain rock that is wrapped in a non-woven geotextile filter fabric. The subdrain should be connected to a non-perforated drainpipe conveyance to storm drain facilities. A diagram of typical footing subdrain is presented in Appendix C as a recommended guideline for construction.

Water should not be allowed to pond beneath floor slabs or within crawl spaces. Floor slab and crawl space subgrade should be sloped to drain to a suitable low point drain outlet or sump. The drain location and routing should be carefully considered to ensure drainage occurs as intended. It might be necessary to install underslab drainage and provide for sump pumps, depending on the below grade depth of floor slabs.

We recommend that all roof drains and subdrains be connected to a non-perforated drainpipe leading to storm drain outlet facilities. Pavement surfaces and open space areas should be sloped such that surface water runoff is collected and routed to suitable discharge points. Ground surfaces adjacent to buildings should be sloped to drain away from the buildings.

3.10 Seismic Design Considerations

At this time, we presume that the building will be designed to resist earthquake loading in accordance with the 2017 ASCE 7-16 standard methodology and as prescribed by the 2019 OSSC. Based on the

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results of drilling exploration, SPT soil strength tests and laboratory tests, we designate the building site to be Seismic Site Class D.

Site coefficients and spectral response acceleration parameters determined for the site using the ASCE Hazard Tool in accordance with the standard ASCE 7-16 methodology are presented in Table 3-3, below. These values are based on risk-targeted maximum considered earthquake (MCE_r) ground motions for the 0.2 and 1 second spectral response accelerations provided in the 2019 OSSC. The values are the lessor of deterministic and probabilistic estimates (2% chance of exceedance in 50 years at 5% critical dampening) of ground motion based on USGS hazard map data available in 2008 and updated in 2014.

Table 3-2 - Seismic Design Parameters (ASCE 7-16)

Parameter	Value
Location (Lat., Lon. in degrees)	45.3015, -122.9567
Mapped Maximum Considered Earthquake Spectral Response Acceleration (USGS Mapping Standardized to Site Class B)	
Short Period, S_S	0.852 g
1 Second Period, S_1	0.412 g
Design Site Coefficients (Site Class D)	
F_a	1.159
F_v	N/A
Design Spectral Response Acceleration Parameter (Site Class D)	
S_{DS} (2/3 x F_a x S_s)	0.658 g
S_{D1} (2/3 x F_v x S_1)	N/A*
Seismic Design Category	N/A*
Peak Ground Acceleration (PGA_M)	0.474 g

*Values not available - Section 11.4.8 of ASCE 7-16 requires site-specific ground motion procedures for structures on Site Class D with S_1 greater than or equal to 0.2g. Please consult Central Geotech regarding details or to have a site-specific analysis completed.

For the alternative simplified design procedure prescribed in Section 12.14 of ASCE 7-16, the value of S_{DS} used to determine seismic base shear is provided in Table 3-3. The values in Table 3-3 assume that a fundamental period (T) of 0.5 seconds or less, and a damping ratio of 5% are appropriate to characterize the structure.

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3.11 Additional Geotechnical Services

Because the future performance and integrity of the structural elements will depend largely on proper site preparation, drainage, fill placement, and construction procedures, construction monitoring and testing (geotechnical special inspection) by experienced geotechnical personnel should be considered an integral part of the design and construction process. Consequently, we recommend that CGS be retained to provide the following post-investigation services:

- Review construction plans and specifications to verify that our design criteria presented in this report have been properly integrated into the design.
- Attend pre-construction meetings and conferences with the design team and contractor to discuss geotechnical related construction issues.
- Observe fill areas and footing subgrade both before fill material or base rock is placed and before footings are constructed in order to verify the soil conditions.
- Prepare a post-construction letter-of-compliance summarizing our field observations, inspections, and test results.

4.0 LIMITATIONS OF REPORT

We have prepared this report for the exclusive use of Jarrod Sherwood, Virginia Garcia Medical Clinic, and members of the design team, for this specific project only. The report should be provided in its entirety to prospective contractors for bidding and estimating purposes; however, the conclusions and interpretations presented should not be construed as a warranty of the subsurface conditions. Experience has shown that soil and groundwater conditions can vary significantly over small distances. Inconsistent conditions can occur between explorations that may not be detected by a geotechnical study. If, during future site operations, subsurface conditions are encountered which vary appreciably from those described herein, Central Geotech should be notified for review of the recommendations of this report, and revision of such if necessary.

We recommend that Central Geotech be retained to review the plans and specifications and verify that our recommendations have been interpreted and implemented as intended. Sufficient geotechnical monitoring, testing and consultation should be provided during construction to confirm that the conditions encountered are consistent with those indicated by explorations. Recommendations for design changes will be provided should conditions revealed during construction differ from those anticipated. Should Central Geotech not be retained for Design or Construction related services further into the development process, this report and its recommendations should be considered void, as we cannot take on responsibility for construction operations that were unobserved by our office.

Within the limitations of scope, schedule and budget, the analysis, conclusions, and recommendations presented in this report were prepared in accordance with generally accepted professional principles and practices in the fields of geotechnical engineering and engineering geology in this area at the time the

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report was prepared. No warranty, express or implied, is made. The scope of our work did not include environmental assessments or evaluations regarding the presence or absence of wetlands or hazardous or toxic substances in the soil, surface water, or groundwater at this site.

5.0 SIGNATURES

Thank you very much for the opportunity to work with you. If you feel obliged, we welcome referrals from our previous clients and would enjoy the opportunity to work with others in your professional and personal networks.

Central Geotechnical Services, LLC

REGISTERED PROFESSIONAL
ENGINEER
97231PE
OREGON
5/11/2021
JOSE R. SERRANO
EXPIRES: 06/30/24

Jose R. Serrano, P.E.
Associate Engineer

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APPENDIX A:

FIELD EXPLORATION PROCEDURES

BORING LOGS

SOIL CLASSIFICATION DESCRIPTION AND GUIDELINES

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FIELD EXPLORATION PROCEDURES

Exploratory Borings

Borings B-1 through B-3 were drilled to a depth of 26.5 feet bgs on June 22, 2022, with a trailer-mounted drill rig using hollow stem auger drilling techniques. The approximate locations of the borings are shown in Figure 2-4.

A member of Central Geotech's geotechnical staff directed the exploration, recorded observed soil and groundwater conditions, and obtained soil samples for laboratory testing. In-situ soil strength was evaluated with Standard Penetration Testing (SPT). SPT tests utilize a 2-inch-diameter split-spoon sampler driven with a 140-pound hammer over a 30-inch free fall. The number of blows to drive the sampler 6 inches is recorded in three successive trials. The sum of the number of blows required to advance the sampler the second and third intervals is the "standard penetration resistance" or "N-value".

Samples obtained from the borings were examined, sealed in plastic bags, and transported to our office for further evaluation. Selected samples were tested in our soil laboratory for moisture content. Summary logs of borings are presented in Appendix A.

Soil Classification and Description

Soil samples were classified in the field in general accordance with the Unified Soil Classification System and guidelines presented in ASTM D2488, *Standard Practice for Description and Identification of Soils (Visual-Manual Procedure)*. The physical characteristics of the samples noted in the field were modified based on laboratory test results, where appropriate, in accordance with ASTM terminology, though certain terminology that incorporates current local engineering practice may be used. The term which best described the major portion of the sample is used to describe the soil type. A one-page summary chart of Soil Classification Description and Guidelines is included in this Appendix.

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BORING LOGS

CENTRAL GEOTECHNICAL SERVICES, LLC

BORING LOG B-1

Virginia Garcia Memorial Health Center	2251 East Hancock Street, Newberg, OR	CGS Project No. 22-025

SUBSURFACE PROFILE

Depth (ft)	Lithologic Description
0–2	3-inches of AC over 5-inches of 3/4-0" crushed Aggregate Base (ROAD BASE)
2–6	Soft SILT (ML), trace angular gravel, trace mixed orgaincs, micaceous, brown, damp (FILL)
6–18	Soft to medium-stiff, SILT (ML), trace fine roots, trace medium-grained sand, micaceous, light brown to brown, damp to moist (MISSOULA FLOOD DEPOSIT) Moisture increases to wet below 7.5 feet bgs
18–26.5	Stiff, fat CLAY (CH), trace sand, dark gray with weak orange and gray mottling, moist to wet (MISSOULA FLOOD DEPOSIT)
26.5	Boring terminated at 26.5 feet bgs Groundwater measured at 6.4 feet bgs at termination

SAMPLE

Sample Number	Recovery (in.)	Sample Type	SPT Count/ RQD	Moisture Content (%)	GW Observation	Lab Analysis
1	17	Split Spoon	3-3-4	36		
2	18	Split Spoon	2-2-3	36	▽	
3	18	Split Spoon	2-1-2	37		
4	18	Split Spoon	3-3-3	39		Liquid Limit: 45 Plastic Limit: 28 Plasticity Index: 17
5	18	Split Spoon	3-4-4	39		
6	18	Split Spoon	4-5-6	33		
7	18	Split Spoon	6-4-6	43		

LEGEND

- ▽ GW Level at End of Drilling
- GW Seepage Zone
- Split Spoon Sample
- Grab Sample
- Core
- Shelby Tube

Date Performed:	June 22, 2022
Logged By:	T. Howard & R. Pavlenko
Surface Elevation:	~ 193 Feet (MSL)
Method:	4 3/8" Solid Stem Auger

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BORING LOG B-2

Virginia Garcia Memorial Health Center	2251 East Hancock Street Newberg, OR	CGS Project No. 22-025

SUBSURFACE PROFILE

Depth (ft)	Lithologic Description
0–2	7-inches of AC over 5-inches of 3/4-"0 crushed Aggregate Base (ROAD BASE)
2–6	Soft SILT (ML), trace angular gravel, trace mixed organics, micaceous, brown, damp (FILL)
6–17	Soft to medium-stiff, SILT (ML), trace fine roots, trace medium-grained sand, micaceous, light brown with weak orange and gray mottling, damp to moist (MISSOULA FLOOD DEPOSIT) Color transitions to dark gray below 10.5 feet, and moisture increases to wet
17–26.5	Medium-stiff to stiff, fat CLAY, trace sand, dark gray weak orange and gray mottling, moist to wet (MISSOULA FLOOD DEPOSIT)
26.5	Boring terminated at 26.5 feet bgs Groundwater measured at 7.9 feet bgs at termination

SAMPLE

Sample Number	Recovery (in.)	Sample Type	SPT Count/ RQD	Moisture Content (%)	GW Observation	SI / Well Construction	Lab Analysis
1	16	Split Spoon Sample	3 3 4				
2	18	Split Spoon Sample	2 3 3		GW Level at End of Drilling (~7.9 ft)		
3	18	Split Spoon Sample	2 2 2				
4	18	Split Spoon Sample	3 3 3				
5	18	Split Spoon Sample	3 2 4				
6	18	Split Spoon Sample	2 2 3				
7	18	Split Spoon Sample	3 4 6				

LEGEND

GW Level at End of Drilling
GW Seepage Zone
Split Spoon Sample
Grab Sample
Core
Shelby Tube

Date Performed: June 22, 2022
Logged By: T. Howard & R. Pavlenko
Surface Elevation: ~ 194 Feet (MSL)
Method: 4 3/8" Solid Stem Auger

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BORING LOG B-3

Virginia Garcia Memorial Health Center	2251 East Hancock Street, Newberg, OR	CGS Project No. 22-025

Depth (ft)	Lithologic Description	Sample Number	Recovery (in.)	Sample Type	SPT Count/ RQD	Moisture Content (%)	GW Observation	SI / Well Construction	Lab Analysis
0–2	4-inches of AC over 9-inches of 3/4-0" crushed Aggregate Base (ROAD BASE)								
2–6	Soft SILT (ML), trace angular gravel, trace mixed organics, micaceous, brown, damp (FILL)	1	12	Split Spoon	3-4-5	35			
		2	16	Split Spoon	2-3-3	36			
6–18	Soft to stiff, SILT (ML), trace fine roots, trace medium-dense sand, micaceous, light brown with weak orange and gray mottling, damp to moist (MISSOULA FLOOD DEPOSIT) Moisture increases to wet at 10 feet bgs						GW Level at End of Drilling (~7.3 ft)		
		3	17	Split Spoon	2-2-2	35			
		4	18	Split Spoon	2-1-2	38			
		5	24	Shelby Tube					
		6	18	Split Spoon	2-2-3	37			
18–26.5	Stiff, fat CLAY (CH), trace sand, dark gray with weak orange and gray mottling, moist to wet (MISSOULA FLOOD DEPOSIT)	7	18	Split Spoon	4-4-6	32			Liquid Limit: 69 Plastic Limit: 27 Plasticity Index: 42
		8	14	Split Spoon	4-4-5	39			
26.5	Boring terminated at 26.5 feet bgs Groundwater measured at 7.3 feet bgs at termination								

LEGEND

- GW Level at End of Drilling
- GW Seepage Zone
- Split Spoon Sample
- Grab Sample
- Core
- Shelby Tube

Date Performed:	June 22, 2022
Logged By:	T. Howard & R. Pavlenko
Surface Elevation:	~ 194 Feet (MSL)
Method:	4 3/8" Solid Stem Auger

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SOIL CLASSIFICATION DESCRIPTION AND GUIDELINES

SOIL CLASSIFICATION

Major Divisions			Symbol	Typical Descriptions
Coarse Grained (More Than 50% Retained By No. 200 Sieve)	Gravel	Clean Gravels	GW	Well-Graded Gravels And Gravel/Sand Mixtures, Little Or No Fines
			GP	Poorly-Graded Gravels, Gravel/Sand Mixtures, Little Or No Fines
		Gravels With Fines	GM	Silty Gravels, Gravel/Sand/Silt Mixtures
			GC	Clayey Gravels, Gravel/Sand/Clay Mixtures
	Sand	Clean Sands	SW	Well-Graded Sand And Gravelly Sands, Little Or No Fines
			SP	Poorly-Graded Sand And Gravelly Sands, Little Or No Fines
		Sands With Fines	SM	Silty Sands, Sand/Silt Mixtures
			SC	Clayey Sands, Sand/Clay Mixtures
Fine Grained (More Than 50% Passing By No. 200 Sieve)	Silts And Clays	Liquid Limit Less Than 50	ML	Inorganic Silts, Silt With Slight Plasticity
			CL	Inorganic Clay, Clay With Low To Medium Plasticity
			OL	Organic Silts, Organic Silty Clays With Low Plasticity
		Liquid Limit More Than 50	MH	Inorganic Silts, Clayey Silts
			CH	Inorganic Clays Of High Plasticity, Fat Clays
			OH	Organinc Clays Of Medium To High Plasticity
Highly Organic Soils			PT	Peat, Humus And Other High Orgainc Soils

SOIL CHARACTERISTICS

Granular Soil		Cohesive Soil		
Relative Density	**Standard Penetration Test**	**Consistency**	**Standard Penetration Test**	**Unconfined Strength (Tsf)**
Very-Loose	0 - 4	Very-Soft	< 2	< 0.25
Loose	4 - 10	Soft	2 - 4	0.25 - 0.5
Medium-Dense	10 - 30	Medium-Stiff	4 - 8	0.5 - 1.0
Dense	30 - 50	Stiff	8 - 16	1.0 - 2.0
Very-Dense	> 50	Very-Stiff	16 - 32	2.0 - 4.0
Standard Penetration Tests Record The Number Of Blows Required To Drive A Split-Spoon Sampler 12 Inches (N-Value)		Hard	32 - 50	> 4.0
		Very-Hard	> 50	

ADDITIONAL SOIL CLASSIFICATION TERMS

Moisture Content	
Dry	Absence Of Moisture, Dusty, Dry To The Touch
Damp	Some Moisture But Leaves No Moisture On Hand
Moist	Leaves Moisture On Hand
Wet	Visible Free Water, Likely From Below Water Table

Groundwater Seepage	
Slow	< 1.0 gpm
Moderate	1.1 - 3.0 gpm
Rapid	> 3.0 gpm

Structure	
Stratified	Alternating Layers of Material or Color > 6 mm
Laminated	Alternating Layers of Material or Color < 6 mm
Fissured	Breaks Along Definite Fracture Planes
Slickenslided	Striated, Polished Or Glossy Fracture Planes
Blocky	Cohesive Soil That Can Be Broken Down Into Angular Lumps Which Resist Further Breakdown
Lenses	Small Pockets Of Different Soils, Note Thickness
Homogeneous	Uniform Color And Appearance Througout

Minor Constituents	
Trace (Clay, Silt, Sand, or Gravel)	< 15 percent
Some (Clay, Silt, Sand, or Gravel)	16 to 30 percent
Clayey, Silty, Sandy, Gravelly	31 to 49 percent

Caving	
Minor	Isolated Spalling
Moderate	Common Spalling
Severe	Will not stand vertical

Plasticity	
Nonplastic	Cannot Be Rolled At Any Water Content
Low	3 mm Thread Can Barely Be Rolled But Not Under The Plastic Limit
Medium	Can Be Rolled To 3 mm Thread , Crumbles When Drier Than Plastic Limit.
High	Can Easily Be Rolled To 3 mm Thread. Can Be Rerolled Several Times.

Dilatancy	
None	No Visible Changes in the Specimen
Slow	Water Slowly Appears and Dissapears
Rapid	Water Quickly Appears and Dissapears

ODOT ROCK HARDNESS CLASSIFICATION CHART

Hardness Designation	Field Identification/Excavation Methods	Approx. Strength (Unconfined Compressive Strength)
Extremely-Soft (RO)	Can Be Indented With Thumbnail. May Be Moldable Or Friable With Finger Pressure.	< 100 Psi
Very-Soft (R1)	Crumbles Under Firm Blows With Geology Pick. Scratched With Finger Nail.	100 - 1,000 psi
Soft (R2)	Can Be Peeled By Knife Or Pick. Shallow Indentation Made By Frim Blow Of Geology Pick.	1,000 - 4,000 psi
Medium-Hard (R3)	Can Be Scratched By Knife Or Pick, Specimen Can Be Fractured With A Single Blow Of Hamer Or Geology Pick / Excavation Often Requires Medium To Large Equipment With Ripper Teeth.	4,000 - 8,000 psi
Hard (R4)	Can Be Scratched With Knife Or Pick Only With Difficulty. Several Hammer Blows Required To Fracture Specimen / Excavation Requires Large Equipment, Rock Chipper, Expansive Compound Fracturing Or Blasting.	8,000 - 16,000 psi
Very-Hard (R5)	Cannont Be Scratched By Knife Or Sharp Pick. Specimen Requires Many Blows Of Hammer To Fracture Or Chip. Hammer Rebounds After Impact / Expansive Compound Fracturing Or Blasting Required To Excavate.	> 16,000 psi

Chart Taken From Oregon Department Of Transportation Soil/Rock Classification Manual. Modified To Include Typical Excavation Methods.

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APPENDIX B:

LABORATORY TEST RESULTS

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ATTERBERG LIMITS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
VGMH	VGMH	22-025	B-1 @ 10'
		REPORT DATE	FIELD ID
		6/30/22	B-1 @ 10'
		DATE SAMPLED	SAMPLED BY
		6/22/22	

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
	Boring B-1 at 10 feet	SILT (ML)

LABORATORY TEST DATA

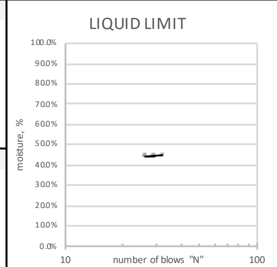
METHOD	TEST PROCEDURE
Wet preparation, Method A - Multipoint	ASTM D4318 & D2216

ATTERBERG LIMITS

liquid limit =	45
plastic limit =	28
plasticity index =	17

LIQUID LIMIT DETERMINATION

	1	2	3	4
wet soil + pan mass, g =	7.9	7.9	7.5	7.8
dry soil + pan mass, g =	5.6	5.6	5.3	5.5
pan mass =	0.4	0.4	0.4	0.4
N (blows) =	29	26	32	27
moisture, % =	44.2%	44.2%	44.9%	45.1%

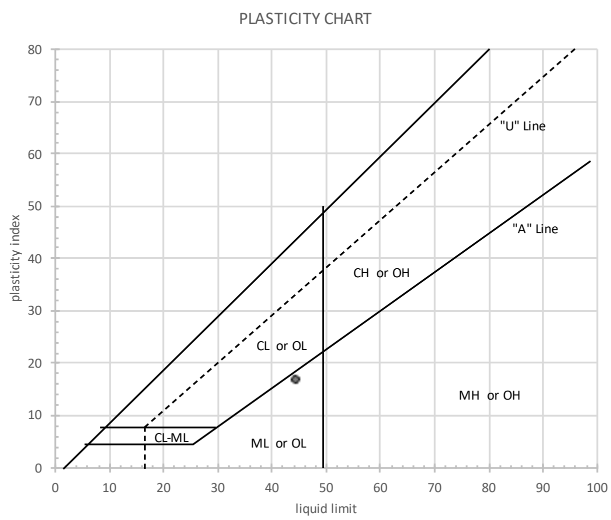


SHRINKAGE

shrinkage limit =	
shrinkage ratio =	

PLASTIC LIMIT DETERMINATION

	1	2	3	4
wet soil + pan mass, g =	10.6	10.3	11.4	10.3
dry soil + pan mass, g =	8.4	8.1	8.9	8.2
pan mass, g =	0.4	0.4	0.4	0.4
moisture, % =	27.5%	28.6%	29.4%	26.9%



ADDITIONAL DATA

% gravel =	
% sand =	
% silt and clay =	
% silt =	
% clay =	
moisture content =	

DATE TESTED	TESTED BY
6/30/22	LMB

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ATTERBERG LIMITS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
VGMH	VGMH	22-025	B-3 @ 20'
		REPORT DATE	FIELD ID
		6/30/22	B-3 @ 20'
		DATE SAMPLED	SAMPLED BY
		6/22/22	

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
	Boring B-3 at 20 feet	Fat CLAY (CH)

LABORATORY TEST DATA

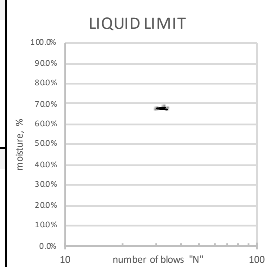
METHOD	TEST PROCEDURE
Wet preparation, Method A - Multipoint	ASTM D4318 & D2216

ATTERBERG LIMITS

liquid limit =	69
plastic limit =	27
plasticity index =	42

LIQUID LIMIT DETERMINATION

	1	2	3	4
wet soil + pan mass, g =	10.1	11.3	11.0	12.1
dry soil + pan mass, g =	6.2	6.9	6.7	7.3
pan mass =	0.4	0.4	0.4	0.4
N (blows) =	34	30	33	26
moisture, % =	67.2%	67.7%	68.3%	69.6%

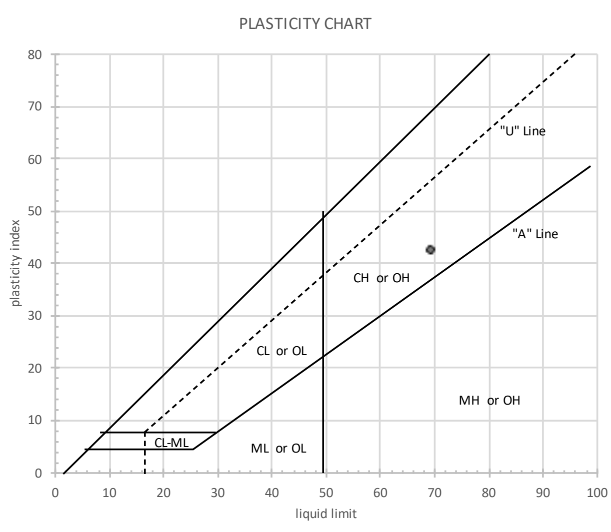


SHRINKAGE

shrinkage limit =	
shrinkage ratio =	

PLASTIC LIMIT DETERMINATION

	1	2	3	4
wet soil + pan mass, g =	10.4	11	9.6	9.7
dry soil + pan mass, g =	8.3	8.7	7.7	7.6
pan mass, g =	0.4	0.4	0.4	0.4
moisture, % =	26.6%	27.7%	26.0%	29.2%



ADDITIONAL DATA

% gravel =	
% sand =	
% silt and clay =	
% silt =	
% clay =	
moisture content =	

DATE TESTED	TESTED BY
6/30/22	LMB

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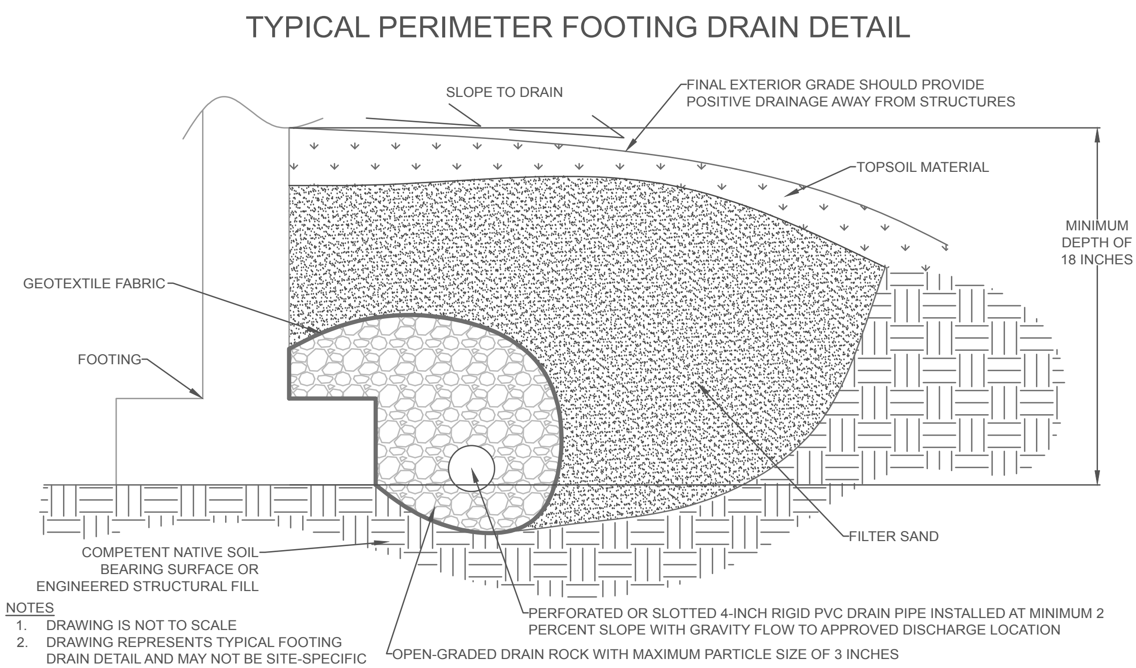
APPENDIX C:

TYPICAL PERIMETER FOOTING SUBDRAIN DETAIL



Guideline drawing for reference only