Earth Engineers, Inc.

2411 S.E. 8th Avenue ● Camas ● WA 98607
Phone: 360-567-1806
www.earth-engineers.com

August 31, 2021
Revised December 3, 2021

Kevin Spence and Amena Syed
45 Eagle Crest Drive, Apartment 501
Lake Oswego, Oregon 97035

E-mail: k.w.spence@outlook.com
atsyed1@gmail.com
Phone: 503-453-4381

Subject: **Geotechnical Investigation Report**
Spence and Syed New Residence
14180 Northwest Germantown Road
Portland, Multnomah County, Oregon
EEI Report 21-156-1-*R1*

Dear Mr. Spence and Ms. Syed:

Earth Engineers, Inc. (EEI) is pleased to provide our attached ***revised*** Geotechnical Investigation Report for the above referenced project. This report includes the results of our field investigation, an evaluation of geotechnical factors that may influence the proposed construction, and geotechnical recommendations for the proposed project. ***The report revisions are changes that Karina Adams of Karina Adams Architecture LLC requested that do not change our geotechnical conclusions or recommendations. Revision additions are notated in bold, italics font.***

We appreciate the opportunity to perform this geotechnical study and look forward to continued participation during the design and construction phases of this project. If you have any questions pertaining to this report, or if we may be of further service, please contact our office at 360-567-1806.

Sincerely,
Earth Engineers, Inc.

Troy Hull, P.E., G.E.
Principal Geotechnical Engineer

Adam Reese, LEG, CEG
Principal Engineering Geologist

John Martin
Geotechnical
Engineering Associate

Attachment: Geotechnical Investigation Report

Distribution (electronic copy only):
Addressees
Karina Adams, Karina Adams Architecture LLC (karina@kadamsarch.com)

GEOTECHNICAL INVESTIGATION REPORT

Spence and Syed New Residence
14180 Northwest Germantown Road
Portland, Multnomah County, Oregon

Prepared for:

Kevin Spence and Amena Syed
45 Eagle Crest Drive, Apartment 501
Lake Oswego, Oregon 97035

Prepared by:

Earth Engineers, Inc.
2411 Southeast 8th Avenue
Camas, Washington 98607
Phone: 360-567-1806

EEI Report No. 21-156-1-*R1*

August 31, 2021
Revised December 3, 2021

Earth Engineers, Inc.

REGISTERED PROFESSIONAL ENGINEER
19206PE
OREGON
JULY 15, 1997
TROY M. HULL
EXPIRES: 6/30 23

Troy Hull, P.E., G.E.
Principal Geotechnical Engineer

Adam Reese, LEG, CEG
Principal Engineering Geologist

John Martin
Geotechnical Engineering Associate

TABLE OF CONTENTS

1.0 PROJECT INFORMATION

1.1 Project Authorization

Earth Engineers, Inc. (EEI) has completed a geotechnical investigation report for the proposed single-family residence to be located on a 4.6 acre vacant parcel, addressed 14180 Northwest Germantown Road, Portland, Oregon. Our services were authorized by Kevin Spence on July 28, 2021 by signing our Proposal No. 21-P273 issued on July 27, 2021.

1.2 Project Description

Our current understanding of the project is based on the information by Ms. Karina Adams of Karina Adams Architecture LLC to EEI Principal Engineering Geologist Adam Reese, LEG, CEG and EEI Geotechnical Engineering Associate John Martin via e-mail between July 16 and July 26, 2021 and on August 4, 2021. ***Additional plans were forwarded for this revision on October 29th, November 12th, and November 28th, 2021.*** We were provided the following documents:

- **Septic Evaluation Application, City of Portland, Bureau of Development Services, undated (blank form), 2-pages.**

- **Site Evaluation Report, City of Portland, Bureau of Buildings, dated August 6, 1990, 11-pages.** This report discusses the site, a soil study and test holes. No data was noted regarding test holes.

- **Lytle Property, by Penny Lytle, dated November 14, 2016, 1-page.** This plan shows a proposed development plan along with a well and 2 seep locations.

- **Geotechnical Engineering Report, Lytle Property, by Terra Dolce Consultants, Inc., dated March 18, 2019, 19-pages.** This report discusses geotechnical recommendations for the property, including 2 borings, 9 ½-feet deep each, and 2 test pits, one 7 ½-feet deep and the other 6 ½-feet deep, on the subject property.

- **Stormwater Drainage Control Certificate, Multnomah County, Land Use Planning Division and Stormwater Drainage Analysis, by Humber Design Group, dated June 22, 2021, 9-pages.** There is a plan attached with the location of the stormwater drainage area. The depth of the facility was not readily apparent to EEI at the initial review.

- **Site Plan, A1.0, by Karina E. Adams Architect, dated June 23, 2021, 1-page.** This plan depicts Olson benchmarks 1, 2 and 3, existing contours overlain with new driveway, house, garage, primary and secondary septic drainfields, stormwater infiltration area and an existing drain field to be decommissioned.

- **Septic Site Evaluation, City of Portland, Bureau of Development Services, rev. July 1, 2021, 4-pages.** This document shows the format for documenting the locations of the test pits relative to property lines, and each test pit, existing structures and/or permanent features.

- **Grading Plan, prepared by Humber Design Group, Inc., dated July 16, 2021, 1-page.** This plan shows the new construction site contours overlain an existing site contour.

- **Preliminary Civil Design Drawings, C1.0, C1.1 and C2.0, by Humber Design Group, Inc., dated July 16, 2021, 3-sheets.** These drawings show an existing water well and future utilities along with current contour elevations and planned elevations.

- **Septic Checksheet, City of Portland, Bureau of Development Services, dated July 20, 2021, 4-pages.** The Checksheet notes there will be a new drainfield area.

- **SER Procedure Info Sheet – 7-1-21, 4-pages.** This document list the septic site evaluation process, including the septic test pit specifications.

- **Infiltration test location exhibit 7-30-2021 2, base plan C2.0, by Humber Design Group, dated July 16, 2021, 1-sheet.** This plan has been marked-up with the location of the infiltration test, dimensioned from surveyed tree locations on the property.

- **2021_0731 Syed Spence A1.0 Marked Up Preliminary Sketch for Geotech Test Pits at Septic, base plan sheet A1.0, by Karina E. Adams Architect, dated July 31, 2021, 1-page.** This plan has been marked-up with the location of 2 septic test pits, dimensioned from surveyed tree locations on the property.

- **2021_0731 Syed Spence A4.0 Exterior Elevations Preliminary Sketch for Geotech Test Pit Depths at house, base plan sheet A3.0, by Karina E. Adams Architect, dated July 31, 2021, 1-page.** This plan shows west and east elevations of the Syed/Spence Residence. The plan shows existing site grades along with basement depths below the main floor level.

- **West side with 15 ft down right 3 ft down left, base plan.** This schematic was developed by Karina Adams during a telephone call on August 4, 2021. Karina Adams asked what the depth to rock had been following the geotechnical test pit excavations. This plan shows a projected rock profile depth based upon Test Pits 3 and 4 (TP-3 and TP-4).

- **East side with 15 ft down right 3 ft down left, base plan.** This schematic was developed by Karina Adams during a telephone call on August 4, 2021. Karina Adams asked what the depth to rock had been following our geotechnical test pit excavations. This plan shows a projected rock profile depth based upon EEI's Test Pits TP-3 and TP-4.

- ***Site Plan, A1.0, by Karina E. Adams Architect, dated October 27, 2021 and November 10, 2021, 1-page each plan date. This plan depicts Olson benchmarks 1, 2 and 3, existing contours overlain with new driveway, house, garage, septic drainfields and stormwater infiltration structures.***

- ***West and East Exterior Elevations, A3.0, by Karina E. Adams Architect, dated October 27, 2021, 1-page.***

- ***South and North Exterior Elevations, A3.1, by Karina E. Adams Architect, dated October 27, 2021, 1-page.***

- ***2021_1112 previous test pits graphic.jpg. This plan shows the boring and test pit locations from the previous geotechnical investigation overlain on Site Plan, A1.0, by Karina E. Adams Architect.***

- ***Geologic Hazard highlighted notes from Prefile Meeting PF-2021-14574 Notes Final.pdf (1-page). This document is page 5 of 10 and has a date of May 13, 2021. It discusses two triggers for a Geologic Hazard Permit (Type II).***

- ***Highlighted hillside_development_permit_handout.pdf (2-pages). This document has highlighted guidance from Multnomah County regarding the Hillside Development Overlay (now Geologic Hazards Overlay). The Overlay shows lands with average slopes of 25-percent or more. The handout also states that lands with average slopes of 25-percent or more must obtain a Geologic Hazards Permit.***

- ***Slope Hazard.pdf (1-page). This document is an e-mail from Bradlee Hersey to Karina Adams, dated June 3, 2021, and discusses the Geologic Hazard permit and the Multnomah County Land Use Planning-Reference Map showing the Slope Hazard overlay and tax lot boundaries, including the subject property and adjacent properties.***

- ***Slope Hazard AKA Geologic Hazard – Slope Layer.jpg (1-page). This image shows the Multnomah County Land Use Planning Reference Map showing the Slope Hazard overlay and tax lot boundaries for the subject property and adjacent properties.***

Briefly, we understand the plan is to construct a new single family home. There are plans and a geotechnical report (prepared by others) from a previous property owner. It appears from the documents that the currently proposed new home is similarly located to the one investigated in 2019 by others. ***Figure 1*** shows a portion of the current site plan and ***Figure 2*** shows the site plan and geotechnical investigation locations in 2019.

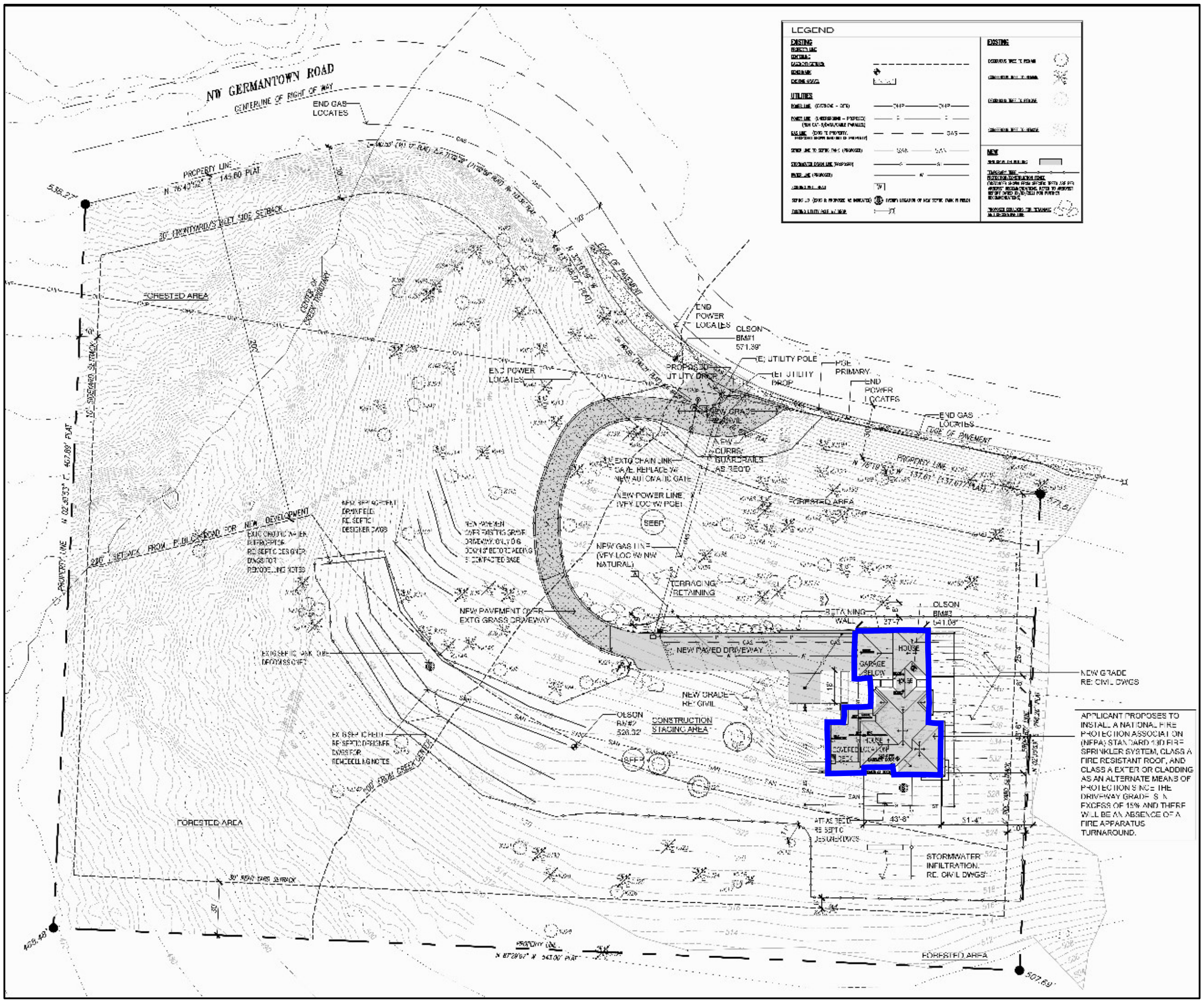


Figure 1: Site plan with property boundaries, topographic contours and location of proposed home footprint (outlined in solid blue) ***(base drawing source: Site Plan, A1.0, by Karina E. Adams Architect, dated November 10, 2021)***.

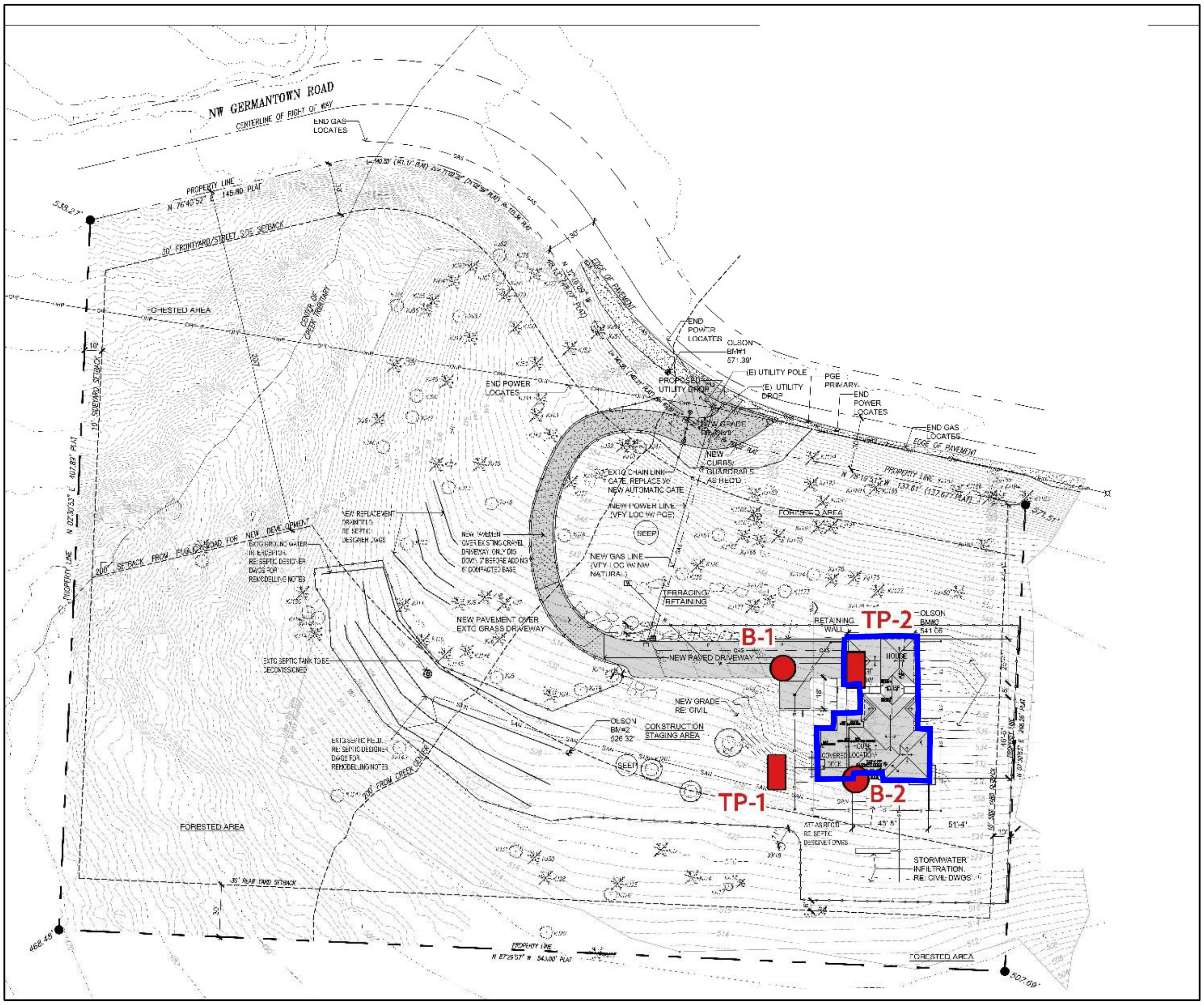


Figure 2: ***Site plan with property boundaries, topographic contours and location of proposed home footprint (outlined in solid blue) overlain with previous geotechnical study locations of borings and test pits (base drawing source: Site Plan, A1.0, by Karina E. Adams Architect, dated November 10, 2021)***.

The project is still in the preliminary design stage. There are some preliminary site grading plans, but detailed construction drawings including structural loads have not been provided to us. For the purposes of this report, we are assuming typical maximum house foundation loads of 4 kips per linear foot for wall footings, 30 kips per column footing, and 150 psf for floor slabs. As presented on the three drawings e-mailed to EEI from Karina Adams on August 4, 2021, below grade construction will include underground utilities and a basement. With regard to site grading, we assume that cuts and fills (excluding the basement) will be up to approximately 3-feet. Finally, we have assumed that the proposed buildings will be constructed in accordance with the 2021 Oregon Residential Specialty Code (ORSC) and/or the 2019 Oregon Structural Specialty Code (OSSC).

During the proposal phase for this geotechnical investigation, we reviewed the Statewide Landslide Information Layer for Oregon (SLIDO) online GIS mapping system (https://gis.dogami.oregon.gov/maps/slido/) for information regarding mapped landslides on or adjacent to the subject site. The area of the subject property has a pre-historic (greater than 150 years ago) landslide. The failure depth is logged as 41-feet. The identified head scarp has a height of 42-feet, which is located to the north of the property, across Northwest Germantown Road. The susceptibility to shallow landslides, classified as less than 15-feet, is mapped as low and moderate in the approximate location of the proposed home. Surrounding the mapped area of moderate susceptibility, the landslide susceptibility is mapped as "high" susceptibility on all sides. SLIDO maps the entire subject property area as having moderate and high susceptibility to deep landslides, classified as deeper than 15-feet.

In addition, we reviewed the Department of Geology and Mineral Industries (DOGAMI) Statewide Geohazards Viewer (https://gis.dogami.oregon.gov/maps/hazvu/) for additional information on the subject property. The subject property is mapped within a severe expected earthquake shaking area and a very high landslide hazard area based solely on topography. The subject property is also mapped less than 500-feet northeast of the Oatfield fault. It should be noted that the portal does not map the property within a liquefaction susceptibility area.

1.3 Purpose and Scope of Services

The purpose of our investigation was to explore the subsurface conditions at the site to provide geotechnical engineering recommendations for the new home, detached garage, ***a retaining wall along the north side of the garage which extends into a landscape area to the west, basement retaining walls,*** and asphalt paved driveway. Additionally, we were asked to provide bulk infiltration rates. Our site investigation consisted of advancing 6 test pits (TP-1 through TP-6) across the development site. The first 4 test pits (TP-1 though TP-4) were excavated for the proposed residence development; TP-5 and TP-6 were excavated in or near the previously identified "seep" locations. Three additional test pit excavations were completed, including 1 for the infiltration test, and 2 for septic test pits to be used for the septic evaluation/inspection by others. A rubber-tired Case 580 Extandahoe was used by subcontractor Dan Fisher Excavating to dig all of the investigation test pits, infiltration test location, and 2 septic test pits. Drive probe tests were performed within TP-1 through TP-4 locations.

Select soil samples were collected from the explorations and returned to our office for testing in our laboratory to determine the material properties for our evaluation. Laboratory testing was accomplished in general conformance with ASTM International, formerly known as American Society for Testing and Materials (ASTM), procedures.

This report briefly outlines the testing procedures, presents available project information, describes the site and subsurface conditions, and presents recommendations regarding the following:

- A discussion of subsurface conditions encountered including pertinent soil and groundwater conditions.
- Seismic design parameters in accordance with ASCE 7-16.

- Geotechnical related recommendations for foundation design including allowable bearing capacity, minimum footing dimensions and estimated settlements.
- Structural fill recommendations, including an evaluation of whether the in-situ soils can be used as structural fill.
- General retaining wall design parameter recommendations, including coefficient of friction and earth pressures.
- Floor slab support recommendations.
- Bulk infiltration rates to aid the Civil Engineer in designing an on-site stormwater disposal system.
- Other discussion on geotechnical issues that may impact the project.

Our scope of services did not include advanced lab testing or a site specific seismic hazard analysis. However, if desired by the client, those services can be provided with an expanded scope of services. Our scope also did not include recommendations for the septic system design, other than to comment on its impact on slope stability. Septic system design is outside our area of expertise.

2.0 SITE AND SUBSURFACE CONDITIONS

2.1 Site Location and Description

The subject property, 14180 Northwest Germantown Road, Portland, Oregon, is a 4.58-acre lot located on the south side of Northwest Germantown Road, west of Northwest Skyline Boulevard, in Portland, Multnomah County, Oregon. At the time of the investigation work conducted for this report, the property was undeveloped with the exception of a gravel driveway and a water well-head, which is shown on Figure 1.

The property slopes steeply to the south from Northwest Germantown Road, as can be seen in the topographic profile shown in Figure 1. Below the steep slope adjacent Northwest Germantown Road, the property has a much more gradual slope. Numerous trees are scattered across the property. Many of the trees are mature and are greater than 30-feet in height. Obvious signs of slope creep, evidenced by pistol-butted trees, were not observed. Below the tree canopy, the ground vegetation was primarily ferns with occasional blackberry and other forest floor vegetation. Outside of the tree canopy, the landscape was predominantly covered by grasses. Photos 1 and 2 below show the property from the north toward the south and southeast, showing the grass-covered open areas and the undergrowth below the tree canopy.

Photo 1: Looking south across property from Northwest Germantown Road entrance.

Photo 2: Looking southeast across property from Northwest Germantown Road entrance.

2.2 Mapped Soils and Geology

The underlying geologic units at the subject property, shown in Figure 3, are mapped as Units Qls and Tgww. Unit Qls – Landslide deposits (Holocene and Pleistocene)—Poorly sorted angular to subrounded bedrock blocks and fragments in weathered muddy matrix. Unit Tgww is a sub-unit of the Columbia River Basalt Group. The Columbia River Basalt Group is generally gray to dark-gray-black, variably vesicular, aphyric to sparsely plagioclase-phyric tholeiitic flood basalt and basaltic andesite flows of the Miocene Columbia River Basalt Group (CRBG).[1,] Also shown on Figure 3, the USGS map identifies the Sylvan-Oatfiled fault.

The surface soils on the subject property are mapped by United States Department of Agriculture (USDA) Soil Survey, as Cascade silt loam (7D), 15 to 30-percent slopes. This unit consists of somewhat poorly drained soils formed from a parent material of silty material formed hillslopes. A typical profile consists of decomposed plant material overlying silt loam overlying a silty clay loam.[2]

[1] Wells, R.E., et. al, 2020, Geologic map of the greater Portland metropolitan area and surrounding region, Oregon and Washington: U.S. Geological Survey Scientific Investigations Map SIM-3443, scale 1:63,360.
[2] Soil Survey Staff, Natural Resources Conservation Service, United States Department of Agriculture. Web Soil Survey. Available online at http://websoilsurvey.nrcs.usda.gov/ accessed 8/16/2021.

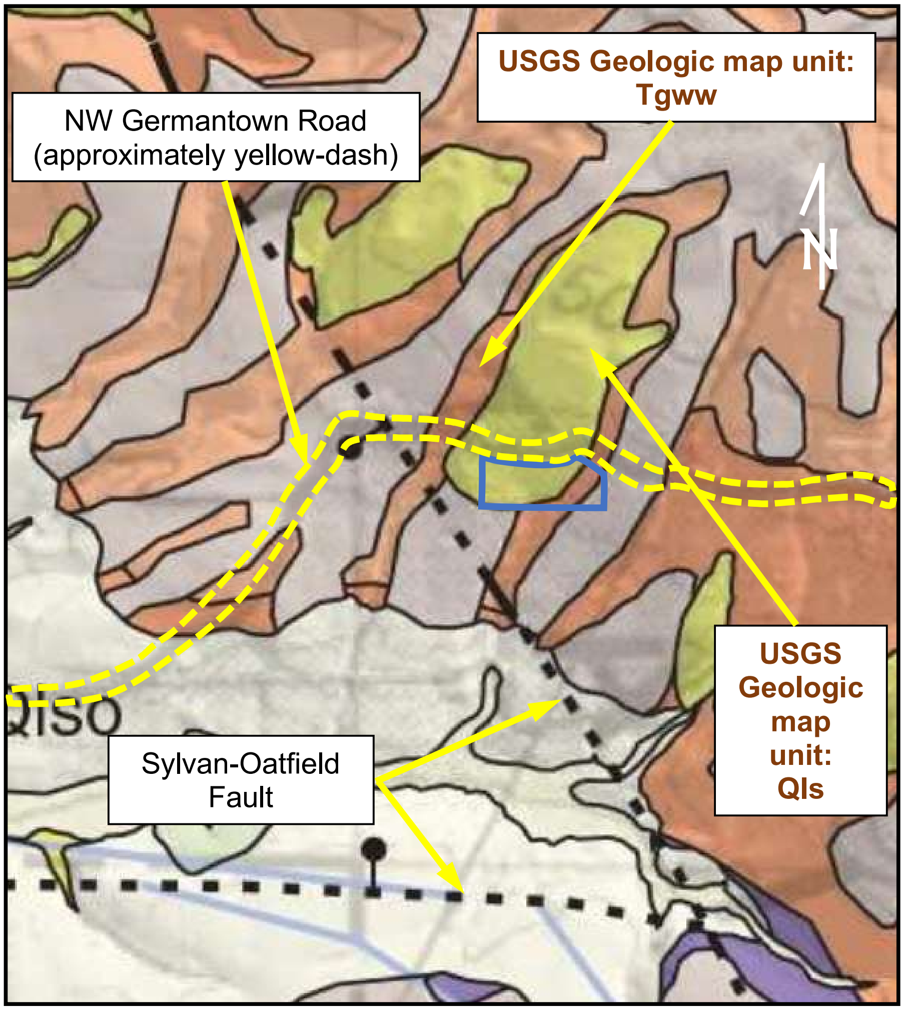


Figure 3: Map of the geologic units in the development area (approximately the solid blue boundary) and the approximate alignment of Northwest Germantown Road (yellow-dash boundary) (base image source: USGS Geologic map of the Greater Portland Metropolitan Area and Surrounding Region, Oregon and Washington, 2020, Reference 1).

2.3 Subsurface Materials

As discussed above, 6 test pits (TP-1 through TP-6) were excavated across the site for the proposed home development. A rubber-tired Case 580 Extandahoe was used by subcontractor Dan Fisher Excavating to dig the 6 test pits, 1 infiltration tests pit and 2 septic test pits. Drive probe tests were performed at the TP-1 through TP-4 locations. The locations of the explorations are shown in Appendix B

Select soil samples were collected from the explorations and returned to our office for testing in our laboratory to determine the material properties for our evaluation. Laboratory testing was accomplished in general conformance with ASTM International, formerly known as American Society for Testing and Materials (ASTM) procedures.

Laboratory testing of collected soil samples was performed to determine material properties for our engineering evaluation. The laboratory testing was accomplished generally in accordance with ASTM procedures. The testing performed included moisture content tests (ASTM D2216), fines content determinations (ASTM D1140). The test results have been included on the exploration logs located in Appendix C. The collected soil samples will be retained for at least 90 days from the date of this report.

As noted above, we supplemented our test pits (TP-1 through TP-4) with drive probe tests, advanced adjacent to the corresponding test pits. The drive probe test is based on a "relative density" exploration device used to determine the distribution and to estimate strength of the subsurface soil and decomposed rock units. The resistance to penetration is measured in blows-per-1/2 foot of an 11-pound hammer which free falls roughly 3.5 feet driving a 3/4-inch diameter pipe into the ground. This measure of resistance to penetration can be used to estimate relative density of soils. For a more detailed description of this geotechnical exploration method, please refer to the Slope Stability Reference Guide for National Forests in the United States, Volume I, United States Department of Agriculture, EM-7170-13, August 1994, P 317-321.

The materials encountered in the test pits may be divided into 3 general strata: topsoil/fill, silt and bedrock. The 3 general strata in the test pits is described below:

Topsoil: Topsoil observed in the explorations was up to 1 ½-feet deep.

Silt (ML): The soil underlying the topsoil was classified as silt by the visual-manual method. Generally this soil layer was stiff to hard, brown silt with moisture contents between 10- and 34-percent. The test pits showed that the brown silt increased in content of decomposed coarse sand to decomposed bedrock at the base of the test pits. Drive probe blow counts in the silt layer ranged from 10 to 56 blows per 6 inches of penetration with an average of 26 blows.

Bedrock: The underlying basalt bedrock is weathered at the surface and becomes competent with depth. The depth of bedrock encountered on the northern end of the proposed home location was approximately 5-feet. At the downhill, southern end, the depth to bedrock was approximately 15-feet.

The classifications noted above were made in accordance with the USCS as shown in Appendix D. The above subsurface description is of a generalized nature to highlight the major subsurface stratification features and material characteristics. The exploration logs provided in Appendix C should be reviewed for specific information at specific locations. These records include soil descriptions, stratifications, and locations of the samples. The explorations performed are not adequate to accurately identify the underlying bedrock across the site. The stratifications shown on the logs represent the conditions only at the actual exploration locations. Variations may occur and should be expected between locations. The stratifications represent the approximate boundary between subsurface materials and the actual transition may be gradual. Water level information obtained during field operations is also shown on these logs.

2.4 Groundwater Information

At the time of our explorations, we did not encounter a clearly identifiable static groundwater level. Several, less than 2-foot deep springs, were commented on in the 2019 Terra Dolce report. The Terra Dolce report also identified two "seeps". During our sitework, we noticed areas with heavier grass vegetation, which may be indicative of shallow groundwater for at least some portions of the year. Test pits 5 and 6 were excavated in the approximate locations of the two seeps, where heavier vegetation was observed. No groundwater was encountered in the two test pits.

We also reviewed the Oregon Water Resources well log registry website (https://apps.wrd.state.or.us/apps/gw/wl_well_report_map), which provides an estimate of the depth to groundwater in the area. According to well logs in the northwest quarter of Section 9, the shallowest depth to groundwater was 84 feet below the existing ground surface at the well location. It should be noted, as discussed above, that subsurface groundwater levels can fluctuate seasonally, especially during periods of extended wet or dry weather or from changes in land use.

2.5 Seismicity

In accordance with ASCE 7-16, we recommend a Site Class D (stiff soil profile with an average standard penetration resistance of between 15 and 50 blows per foot) when considering the average of the upper 100 feet of bearing material beneath the proposed foundations. This recommendation is based on the stability of the test pit side walls, drive probe test blow counts, as well as our local knowledge of the area geology. Inputting our recommended Site Class as well as the site latitude and longitude into the Structural Engineers Association of California (SEAOC) – OSHPD Seismic Design Maps website (http://seismicmaps.org) which is based on the United States Geological Survey, we obtained the seismic design parameters shown in Table 1 below.

Table 1: Seismic Design Parameter Recommendations (ASCE 7-16)

PARAMETER	RECOMMENDATION
Site Class	D
S_s	0.913g
S_1	0.42g
F_a	1.2
F_v	null-see section 11.4.8
S_{MS} (=S_s x F_a)	1.095g
S_{M1} (=S_1 x F_v)	null-see section 11.4.8
S_{DS} (=2/3 x S_s x F_a)	0.73g
Design PGA (=S_{DS} / 2.5)	0.292g
MCE_G PGA	0.415g
F_{PGA}	1.2
PGA_M=(MCE_G PGA x F_{PGA})	0.497g

Note: Site latitude = 45.5850616, longitude = -122.8208781

The return interval for the ground motions reported in the table above is 2 percent probability of exceedance in 50 years.

Per Section 11.4.8 of ASCE 7-16 a site-specific seismic site response analysis (i.e. SHAKE software or equivalent) is required for structures on Site Class D and E sites with S_1 greater than or equal to 0.2g. The S_1 value for this site is greater than 0.2g as shown in Table 1 above. Therefore, a site response analysis is required as part of the design phase. However, Section 11.4.8 does provide an exception for not requiring a site response analysis (reference Sections 11.4.8.1, 11.4.8.2 and 11.4.8.3). The project Structural Engineer should determine if the proposed building will meet any of the exceptions—if the building does not meet the exception requirements, then EEI should be retained to perform a site-specific site response analysis.

We understand a Supplement 1 dated December 12, 2018 has been issued for ASCE 7-16 to correct some issues in the original publication. One of the corrections in the Supplement pertains to Table 11.4-2 for determining the value of the Long-Period Site Coefficient, F_V, which is then used to calculate the value of T_S. The T_S value is needed for one of the exceptions in Section 11.4.8. Without the correction in Supplement 1, it would not be possible to determine F_V and calculate T_s. Based on Supplement 1, the F_V value may be determined from the following table.

Table 2: Long-Period Site Coefficient, F_V (corrected Table 11.4-2 in ASCE 7-16).

	Mapped Risk-Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameter at 1-s Period					
Site Class	S_1<=0.1	S_1<=0.2	S_1<=0.3	S_1<=0.4	S_1<=0.5	S_1>=0.6
A	0.8	0.8	0.8	0.8	0.8	0.8
B	0.8	0.8	0.8	0.8	0.8	0.8
C	1.5	1.5	1.5	1.5	1.5	1.4
D	2.4	2.2 [a]	2.0 [a]	1.9 [a]	1.8 [a]	1.7 [a]
E	4.2	3.3 [a]	2.8 [a]	2.4 [a]	2.2 [a]	2.0 [a]
F	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8	See Section 11.4.8

Note: use linear interpolation for intermediate values of S_1.

[a] See requirements for site-specific ground motions in Section 11.4.8. These values of F_V shall be used only for calculation of T_S.

2.6 Infiltration Study

An infiltration test, IT-1, was performed in the infiltration test pit shown on Appendix B, on August 3, 2021. The test was conducted in accordance with the 2020 City of Portland Stormwater Management Manual, Section 2.3.2 and 2.3.3.

In accordance with the Encased Falling Head test procedure, six-inch diameter solid PVC casings (stand pipe) were embedded approximately six inches into the soil at the bottom of the test pit excavation. Approximately three inches of clean gravel was then placed in the bottom of each stand pipe to protect the soil at the bottom of the pipe from scouring. A minimum 4-hour presoak was performed at IT-1 because, after filling the standpipe with 12 inches of water, all of the water did not seep away in less than 10 minutes.

After the 4-hour minimum presoak was completed, infiltration test trials were performed by filling each solid six-inch diameter standpipe casing with 12 inches of water above the soil at the bottom of the pipe, and measuring the water level after set time intervals. A total of 3 trials were performed at IT-1 (a minimum of three is required), until the percent change in measured infiltration rate between two successive trials was minimal. The results of our infiltration testing and washed gradation are summarized in Table 3 below. Note that the reported infiltration rates in Table 3 are ultimate values and *do not include a factor of safety.*

Table 3: Summary of Infiltration Testing and Washed Gradation Results.

Test No.	Approximate Test Location	Approximate Test Depth	Visual Soil Classification	Washed Gradation (% passing #200 sieve)	Measured Infiltration Rate (inches/ hour)
IT-1	340-feet south of Northwest Germantown Road	3.5 feet below existing grade	brown silt	94	**< 0.01**

2.7 Geologic Hazards

As discussed in section 1.2, the Department of Geology and Mineral Industries (DOGAMI) Statewide Landslide Information Layer for Oregon (SLIDO) online GIS mapping system (https://gis.dogami.oregon.gov/maps/slido/) and the DOGAMI Statewide Geohazards Viewer (HAZVU) (https://gis.dogami.oregon.gov/maps/hazvu/), were reviewed for information regarding mapped landslides on or adjacent to the subject site and landslide hazards. As shown in Figure 4, the area of the subject property is within a large mapped pre-historic (greater than 150 years ago) landslide zone. Figures 4 and 5 show the HAZVU historic landslides and landslide hazard susceptibility plotted over a bare earth LIDAR (Light Detection and Ranging) data view. HAZVU shows the property mapped within a very high landslide hazard area based solely on topography. The LIDAR image enables identification of the drainage area at the northwest corner of the property and Germantown Road. Also, HAZVU mapped the property within a severe expected earthquake shaking area, which is consistent with the Oatfield fault, mapped approximately 500-feet southwest of the property (see Figure 3).

The SLIDO database identifies the larger slide failure depth as 41-feet; the head scarp has a height of 42-feet, which is located to the north of the property, across Northwest Germantown Road. The smaller interior landslide, just to the east of the property, is identified in the SLIDO database as a historic landslide (less than 150 years ago), head scarp height at 20-feet, and a failure depth of 20-feet.

SLIDO maps susceptibility to shallow landslides, classified as less than 15-feet, is mapped as low and moderate in the approximate location of the proposed home. Uphill of the home location and below Germantown Road, susceptibility to shallow landslides is mapped as high.

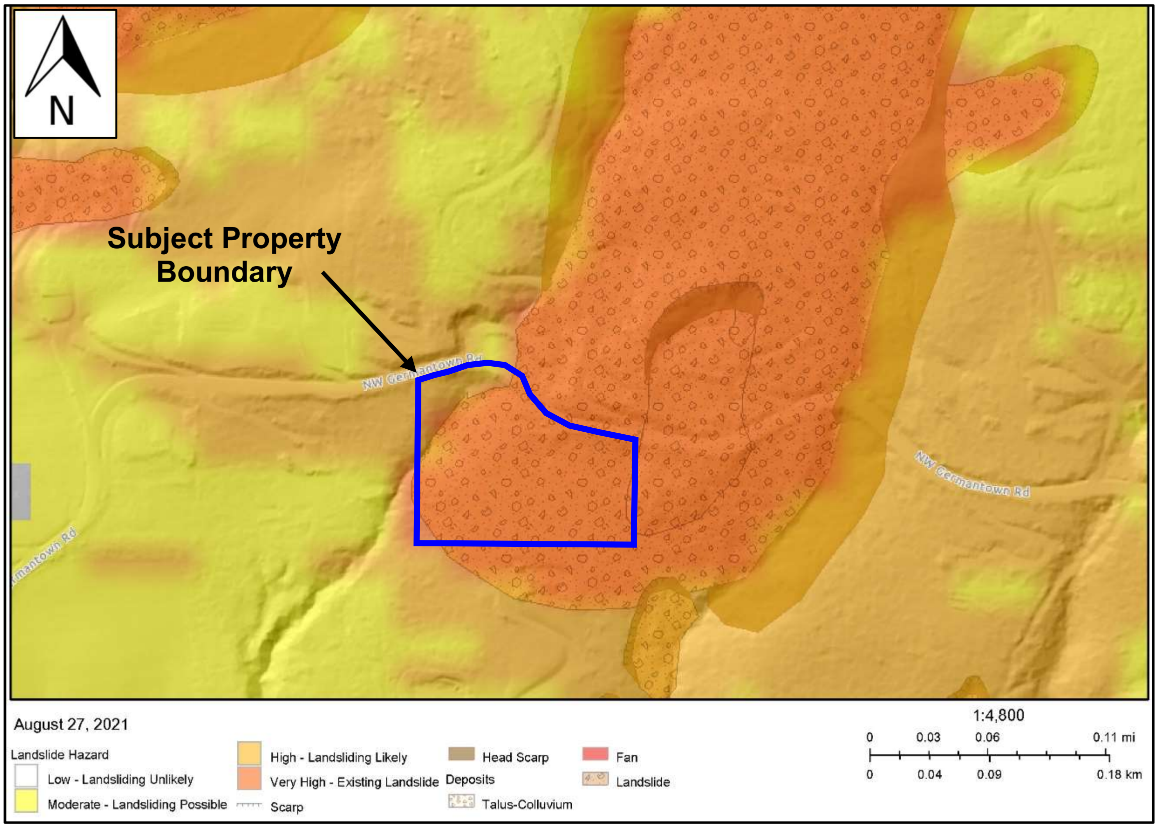


Figure 4: Landslides and landslide hazards as shown by HAZVU and the property boundary approximated by the blue outlined region. (https://gis.dogami.oregon.gov/maps/hazvu/).

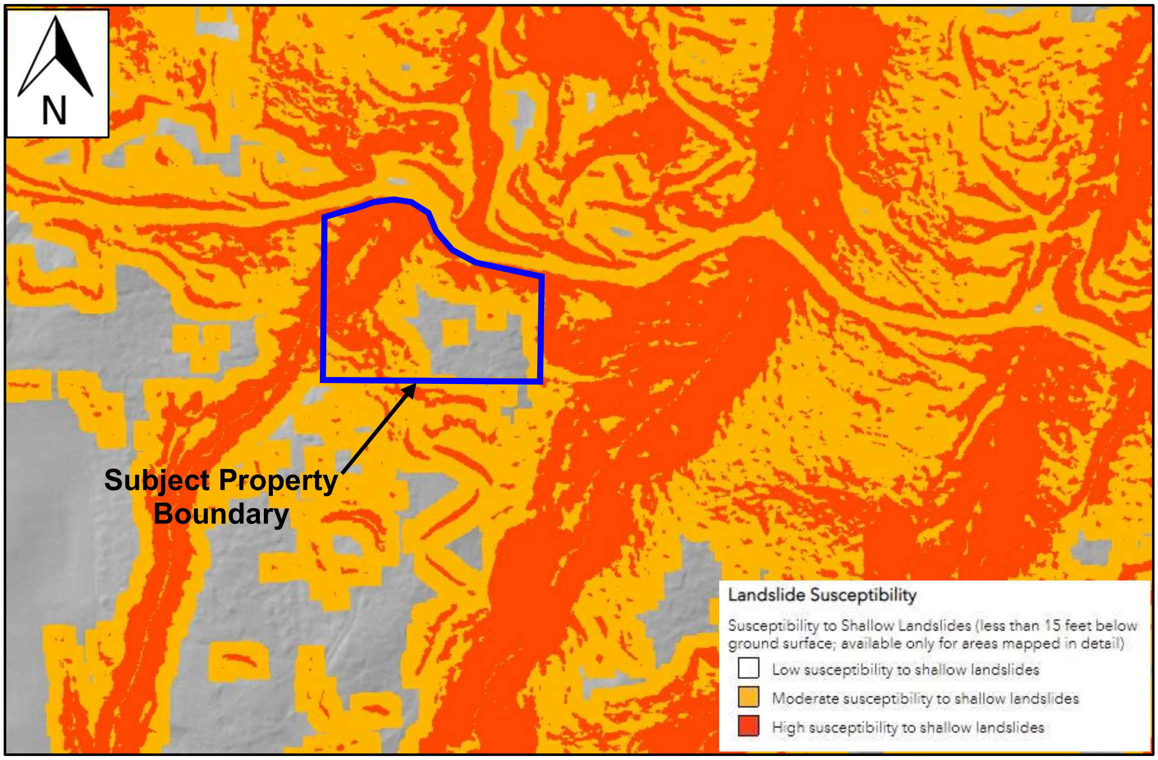


Figure 5: Landslides hazard susceptibility as shown by HAZVU and the property boundary approximated by the blue outlined region. (https://gis.dogami.oregon.gov/maps/hazvu/).

Notes on Site Plan, A1.0, by Karina E. Adams Architect, dated November 10, 2021 (Site Plan), show that west of the planned new home, the existing septic field will be remodeled and there will be a "new replacement drainfield." Above the existing septic field is a line labeled "EXTG GROUND WATER INTERCEPTOR". Our assumption is that this line is constructed as an interceptor of subsurface groundwater flow (i.e. greater than 12-inches below the ground surface), from the hillside above, and the "interceptor" line does not capture and re-direct surface water run-off. The septic drainfield areas shown are on approximately 5H:1V slopes, interpolated from contour elevations shown on the Site Plan. The new stormwater infiltration area has an approximate slope of 3H:1V, interpolated from contour elevations shown on the Site Plan. Figure 6 shows the groundwater interceptor line along with the originally planned areas and currently planned areas for the septic drainfields and stormwater infiltration. Based on the site reconnaissance by EEI Principal Engineering Geologist Adam Reese, he recommended (based on qualitative assessment) that the moderately sloping (i.e. approximately 5H:1V on average in the septic drainfield area and the approximately 3H:1V on average in the stormwater infiltration area) areas of the newly proposed drainfields are acceptable from a geotechnical standpoint.

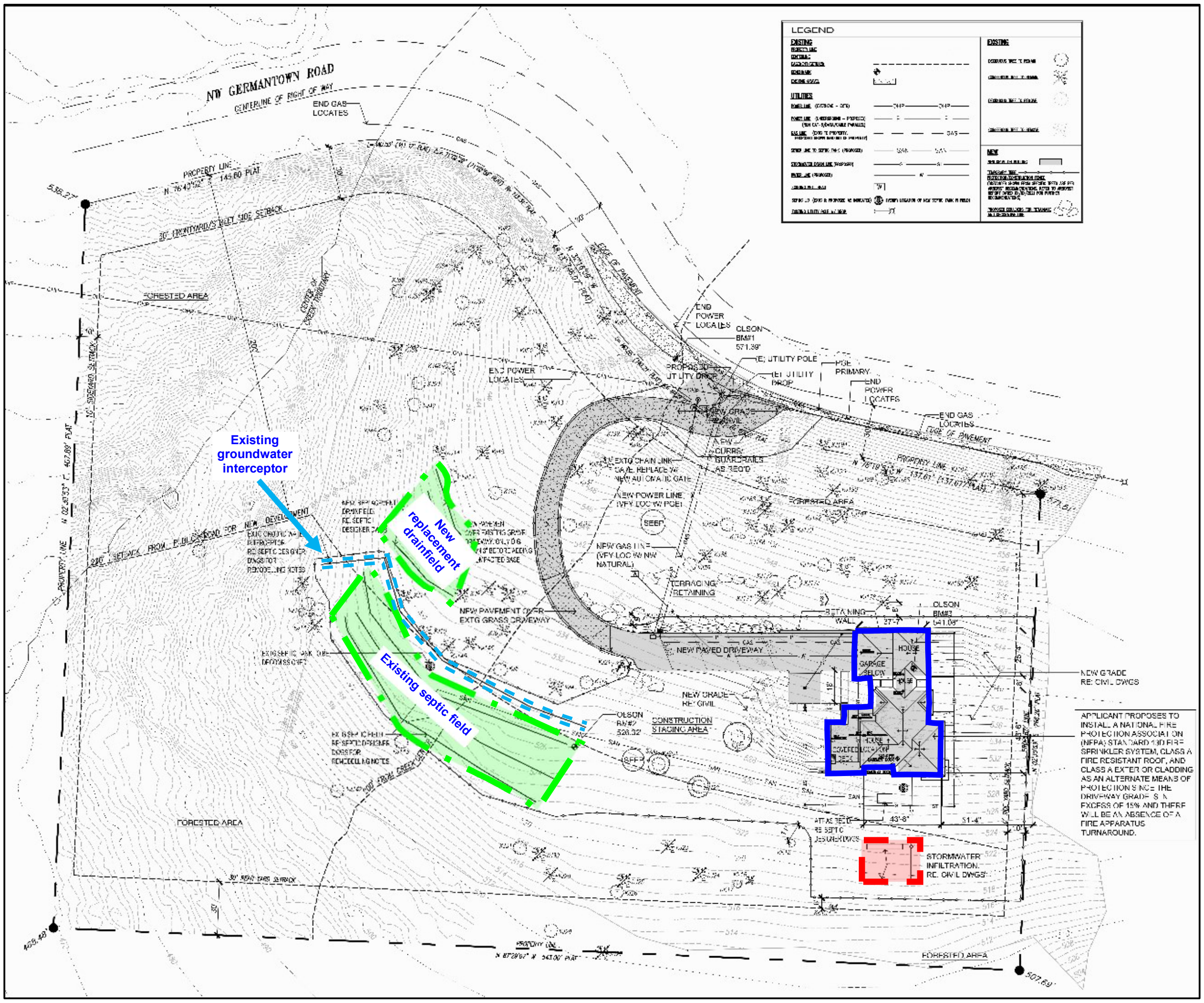


Figure 6: Site plan with property boundaries, topographic contours, location of proposed home footprint (outlined in solid blue), stormwater infiltration area currently planned (outlined in red-dash lines), existing groundwater interceptor line (outlined in light blue-dot double lines), and septic drainfield areas currently planned (outlined in green-dash dot lines) (base drawing source: Site Plan, A1.0, by Karina E. Adams Architect, dated November 10, 2021).

In our opinion, the home construction does not trigger a Geologic Hazards Permit requirement per the "Geologic Hazard highlighted notes from Prefile Meeting PF-2021-14574 Notes Final.pdf," (1-page) and the "highlighted hillside development permit_handout.pdf," (2-pages) from Karina Adams. These documents discuss two triggers for a Geologic Hazard, 1) if development will occur within the County's mapped hazard areas identified on the Geologic Hazard Overlay map, or 2) if development will occur on land with an average slope of 25-percent or more. Based on our review of two other documents from Karina Adams on November 28th which show the Multnomah County Geologic Hazard Overly map, and our review of the map (accessed November 29th, 2021), the Geologic Hazard Overlay zones do not extend onto the subject property tax lot. Figure

7 shows the Multnomah County Land Use Planning-Reference Map with the Geologic Hazards overlay. Also, based upon the topographic survey of existing conditions, presented on the Site Plan, A1.0, by Karina E. Adams Architect, dated October 27, 2021 and November 10, 2021, the proposed house location is not situated on a slope of 25-percent or steeper.

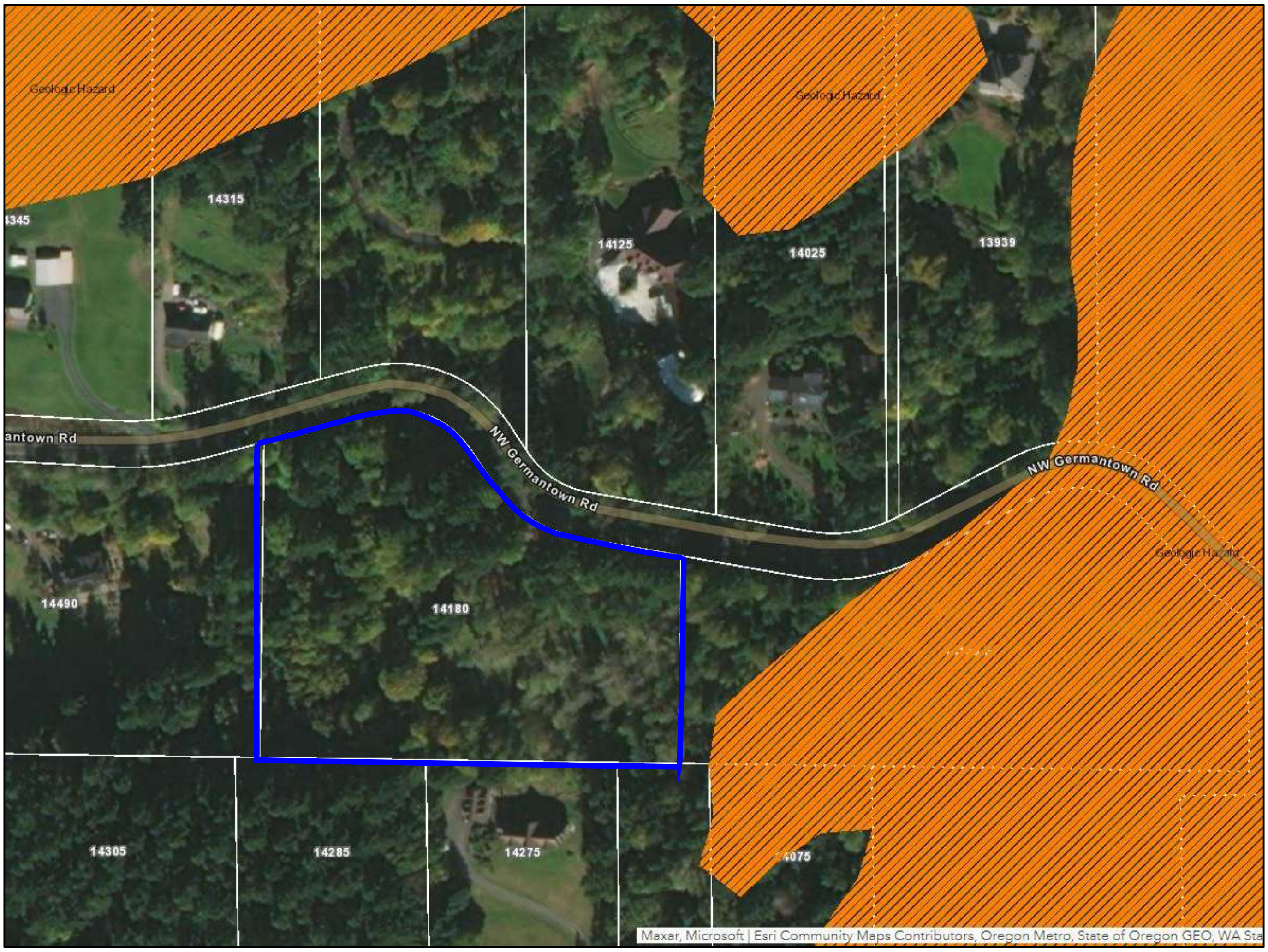


Figure 7: Geologic Hazard Overlay map showing nearest areas identified by Multnomah County Land Use Planning. The property tax lot boundaries are shown and the proposed home tax lot is outlined in solid blue (base image source: https://www.arcgis.com/apps/webappviewer/index.html?id=9c6906dd2ff1459b9d6c7d0a0de4afb2, accessed November 29, 2021).

3.0 EVALUATION AND FOUNDATION RECOMMENDATIONS

3.1 Geotechnical Discussion

The following geotechnical factors may influence the proposed construction:

- **Presence of loose/compressible soils** – Topsoil discovered across the site should be removed beneath future structures. Our test pits and drive probe testing indicated the loose soils are approximately 1 to 1 ½-feet deep across the property.

- **Bedrock** – The depth to bedrock below the planned house, including the daylight basement, could impact the proposed construction. The upper 1-foot (approximate) of bedrock was observed to be in a weathered condition and may be rippable with hydraulic excavation equipment. However, the deeper bedrock is expected to be very dense and may require specialized rock/pavement excavation equipment to excavate into the non-weathered bedrock. In other words, if hard bedrock is encountered within the planned excavations, it could be difficult and impact construction costs and timing.

  The depth to bedrock also affects our foundation recommendations. We anticipate in some areas of the house and detached garage, bedrock may be encountered at the base of the foundation excavation. In other areas, bedrock may be several feet below the bottom of the proposed footings. To mitigate this issue, we recommend that the house and detached garage be designed for pin pile support. Then wherever the footing excavations expose the hard bedrock, the pin piles at those locations may be deleted (i.e. the foundations may be partially supported on conventional spread footings on bedrock and partially on pin piles driven to the bedrock stratum).

- **Landslide Hazards** – The site is located within a historic landslide deposit. Based on the material properties observed in our explorations, the landslide soils are present at depths of 5- to 15-feet bgs. We recommend that the daylight basement retaining wall and building foundations should founded directly on the bedrock or pin piles.

In summary, provided the recommendations in this report are adhered to, we do not foresee any major issues that would preclude site development or the proposed construction. The above-mentioned factors are listed to draw the attention of the reader to the issues to address during design and construction of the proposed building.

3.2 Site Preparation

At the start of site preparation, our test pit explorations should be located, excavated to their bottoms and backfilled with properly compacted granular structural fill. The repair of the test pits should be observed and documented by the Geotechnical Engineer's representative.

Once the test pits are repaired, we envision that topsoil, fill soils (if any), and any other deleterious soils, will need to be stripped from beneath the proposed improvement areas to expose the underlying stiff soils (i.e. silt or clayey silt). Once the building pads, driveways and parking areas are stripped of the topsoil and excavated to grade, surface disturbances from excavation and grading activities should be re-compacted using a smooth drum roller or a heavy diesel plate compactor. The prepared areas should then be proof-rolled with a fully loaded, rubber tire dump truck or water truck and evaluated by the Geotechnical Engineer's representative to determine if any areas of unsuitable soils or soft subgrade exist. Alternatively, the exposed subgrades can be visually evaluated by a representative of the Geotechnical Engineer using a ½-inch diameter steel soil probe. Unsuitable soils should be overexcavated and replaced with compacted structural fill. Once the subgrades are approved, the placement of the compacted crushed rock structural fill, reinforcing steel, concrete, etc. may begin.

Any utilities present beneath the proposed construction will need to be located and rerouted as necessary and any abandoned pipes or utility conduits should be removed to inhibit the potential for subsurface erosion. Utility trench excavations should be backfilled with properly compacted structural fill in accordance with Section 3.3.

3.3 Structural Fill

Any structural fill to be placed should be free of organics or other deleterious materials, have a maximum particle size generally less than 3 inches, be relatively well graded, and have a liquid limit less than 45 and plasticity index less than 25. In our professional opinion, the native, on-site silt soils (classified as ML), free of organics and properly moisture conditioned, would be appropriate for use as structural fill during dry weather construction—except the fine-grained soils should not be used for structural fill beneath footings. We recommend fill be moisture conditioned to within 3 percentage points below and 2 percentage points above optimum moisture as determined by ASTM D1557 (Modified Proctor). If water must be added, it should be uniformly applied and thoroughly mixed into the soil by disking or scarifying. Note that based on our past experience, the native fine-grained soils can be difficult to properly compact. They can usually only be placed in the dry summer months and after they have been dried out to about optimum moisture content.

Additionally, we recommend a select granular material be used in the overexcavation (if any) and backfilling operations beneath the footings discussed above – specifically a crushed rock gravel that is relatively well graded, have a maximum particle size of 1½-inches, with no more than 15 percent passing the U.S. #200 sieve (0.075 mm). If the gravel is used as fill during wet weather conditions, then we recommend it contain no more than 5 percent passing the #200 sieve.

Fill should be placed in relatively uniform horizontal lifts on the prepared subgrade which has been stripped of deleterious materials and approved by the Geotechnical Engineer or their representative. Structural fill thickness should not exceed 12 inches per loose lift. The type of compaction equipment used will ultimately determine the maximum lift thickness. Structural fill should be compacted to at least 95 percent of maximum dry density as determined by ASTM

D1557 (Modified Proctor). Each lift of structural fill should be tested by a representative of the Geotechnical Engineer prior to the placement of subsequent lifts.

The fill should extend horizontally outward beyond the exterior perimeter of the building and pavements at least 5 feet and 3 feet, respectively, prior to sloping.

Any structural fill placed on slopes at or greater than 5H:1V should be properly benched. Level benches excavated into the existing slope should be a minimum of 4 feet wide laterally, and should be cut into the slope for no more than every five feet of vertical rise. The placement of fill should begin at the base of the fill. All benches should be inspected by a representative of the Geotechnical Engineer and approved prior to placement of structural fill lifts. If evidence of seepage is observed in the bench excavations, a supplemental drainage system may need to be designed and installed to prevent hydrostatic pressure buildup behind the fill. Final fill and/or cut slopes should be kept at or below a slope of 2H:1V.

To reiterate, each lift of compacted engineered fill should be tested by a representative of the Geotechnical Engineer prior to placement of subsequent lifts.

3.4 Foundation Recommendations

At this time, we have not been provided with maximum foundation loading. For the purposes of this report, we have assumed typical maximum column loads of 30 kips, wall loads of 4 kips per linear foot, and floor slab loads of 150 pounds per square foot (psf). The project Structural Engineer should review our assumptions and we should be notified as soon as possible if the actual maximum foundation loads differ from the assumed foundation loads.

Based on our subsurface explorations, the results of our laboratory and field testing, assumed maximum loading, and our current limited understanding of the project, it is our professional opinion that the proposed home and detached garage may be supported with spread footings founded on hard unweathered basalt bedrock or pin piles driven to refusal on bedrock (i.e. as described in Section 3.1 above, the structures may be partially supported on piles and partially on spread footings). At the northern, uphill, area of the garage, bedrock may be encountered at 5-feet bgs, while at the downhill, southern end of the home, bedrock may be encountered at 15-feet bgs.

We recommend the pin piles be 4-inch minimum diameter, schedule 80 steel pipe piles driven to practical refusal using a hydraulic 1,500-pound hammer or equivalent. Refusal for a 4-inch diameter pipe pile using a hammer of this size should be defined as less than 1-inch of penetration in 16 seconds. When practical, this refusal criteria should be met for the last 60 seconds of pile driving.

Assuming the piles are driven to refusal using this criteria, the allowable axial capacity for a pile installed vertically would be up to 20.0 kips in compression. This allowable axial capacity assumes a factor of safety of 2.0. We recommend a maximum lateral load resistance of 1.0 kip for each vertical pile as long as they are spaced a distance of at least 6D (measured from center to center) where D represents the diameter of the pile. If additional lateral load resistance is needed, we can provide drilled and grouted tieback recommendations upon request. The structural engineer will need to determine the number and placement of the deep foundations based on the geometry and the loads imposed by the proposed residence.

Based on the known subsurface conditions we anticipate that properly constructed pin pile foundations driven to refusal could experience static settlements of no more than 1-inch and 1/2-inch of total and differential settlement, respectively.

The foundation excavations should be observed by a representative of EEI prior to steel or concrete placement to assess that the foundation materials are capable of supporting the design loads and are consistent with the materials discussed in this report. Unsuitable soil zones encountered at the bottom of the foundation excavations should be removed and corrected as recommended by the geotechnical engineer. Cavities formed as a result of excavation of unsuitable soil zones should be backfilled with lean concrete or compacted structural fill.

3.5 Floor Slab on Grade Recommendations

For the purposes of this report, we have assumed that maximum floor slab loads will not exceed 150 psf. Based on the existing soil conditions, the design of slabs-on-grade can be based on a subgrade modulus (k) of 125 pci for slabs placed within 3 feet of the existing site grades. This subgrade modulus value represents an anticipated value which would be obtained in a standard in-situ plate test with a 1-foot square plate. Use of this subgrade modulus value for design or other on-grade structural elements should include appropriate modification based on dimensions as necessary.

In order to provide a localized (i.e. shallow) uniform subgrade reaction beneath any proposed slab-on-grade, we recommend that a minimum 4-inch thick, free draining, granular mat be placed beneath the floor slab to enhance drainage and provide a capillary break to limit migration of moisture through the slab and provide increased subgrade strength. If additional protection against moisture vapor is desired a suitable vapor retarding membrane may also be incorporated into the design and can be placed on the granular mat to act as a vapor barrier as required by codes or manufacturer requirements. Factors such as cost, special considerations for construction and the specific floor coverings suggest that decisions on the use of vapor retarding membranes be made by the architect and the owner. The floor slabs should have an adequate number of joints to reduce cracking resulting from any differential movement and shrinkage.

3.6 Retaining Wall Recommendations

Retaining walls are planned along the north side of the garage, running east-west, and extending west past the front of the garage, to retain an uphill landscaped area. In addition, we understand that three of the basement walls will be retaining walls.

Retaining wall footings should be designed in general accordance with the recommendations contained in Section 3.4 above. Lateral earth pressures on walls, which are not restrained at the top, may be calculated on the basis of an equivalent fluid pressure of 35 pcf for level backfill, and 60 pcf for sloping backfill with a maximum 2H:1V slope. Lateral earth pressures on walls that are restrained from yielding at the top (i.e. stem walls) may be calculated on the basis of an “at-rest” equivalent fluid pressure of 55 pcf for level backfill, and 90 pcf for sloping backfill with a maximum 2H:1V slope. The stated equivalent fluid pressures do not include surcharge loads, such as foundation, vehicle, equipment, etc., adjacent to walls, hydrostatic pressure buildup, or earthquake loading. Surcharge loads on walls should be calculated based on the attached calculations/formulas shown in Appendix E.

Lateral loads may be resisted by frictional resistance between the base of the retaining wall footing and the subgrade and can be expressed as the applied vertical load multiplied by a coefficient of friction of 0.40 for concrete foundations bearing directly on the unweathered basalt bedrock or 0.36 for concrete foundations bearing directly on at least 12 inches of gravel structural fill. In addition, lateral loads may be resisted by passive earth pressures based on an equivalent fluid density of 250 pcf for footings poured "neat" against in-situ soils, or properly backfilled with structural fill. These are ultimate values. We recommend a factor of safety of 1.5 be applied to the equivalent fluid pressure, which is appropriate due to the amount of movement required to develop full passive resistance. To be clear, a safety factor has not been applied to the coefficient of friction recommended above.

We recommend that retaining walls be designed for an earth pressure determined using the Mononobe-Okabe method to mitigate future seismic forces. Our calculations were based on one-half of the Design Peak Ground Acceleration (PGA) value of 0.292g, which was obtained from Table 1 above. We have assumed that the retained soil/rock will have a minimum friction angle of 27 degrees and a total unit weight of about 130 pcf***, and that the retained backfill be sloping no steeper than 10 degrees***. For seismic loading on retaining walls, new research indicates that the seismic load is to be applied at 1/3 H of the wall instead of 2/3 H, where H is the height of the wall[3]. We recommend that a Mononobe-Okabe earthquake thrust per linear foot of ***10.6 psf*** * H^2 be applied at 1/3 H from the base of the wall, where H is the height of the wall measured in feet.

All backfill for retaining walls (within a horizontal distance equal to 75 percent of the wall height, but no greater than 6 feet) should be select granular material, such as sand or crushed rock with a maximum particle size between ¾ and 1 ½ inches, having less than 5 percent material passing the No. 200 sieve. Because of their fines content, the native soils do not meet this requirement, and it will be necessary to import material to the project for structure backfill. Silty soils can be used for the last 18 to 24 inches of backfill, thus acting as a seal to the granular backfill.

All backfill behind retaining walls (within a horizontal distance equal to 75 percent of the wall height, but no greater than 6 feet) should be moisture conditioned to within ± 2 percent of optimum moisture content, and compacted to a minimum of 92 percent of the material's maximum dry density as determined in accordance with ASTM D1557. Fill materials should be placed in layers

[3] Lew, M., et al (2010). “Seismic Earth Pressures on Deep Building Basements,” SEAOC 2010 Convention Proceedings, Indian Wells, CA.

that, when compacted, do not exceed about 8 inches. Care in the placement and compaction of fill behind retaining walls must be taken in order to ensure that undue lateral loads are not placed on the walls.

An adequate subsurface drain system will need to be designed and installed behind retaining walls to prevent hydrostatic buildup. A waterproofing system should be designed for any basement walls where moisture intrusion is not desirable.

4.0 CONSTRUCTION CONSIDERATIONS

EEI should be retained to provide observation and testing of construction activities involved in the foundation, earthwork, and related activities of this project. EEI cannot accept any responsibility for any conditions that deviate from those described in this report, nor for the performance of the foundations if not engaged to also provide construction observation for this project.

4.1 Moisture Sensitive Soils/Weather Related Concerns

During wet weather periods, increases in the moisture content of the soil can cause significant reduction in the soil strength and support capabilities. The upper soils consist of silt, which given the fines content, could become wet and may be slow to dry, thus significantly retard the progress of grading and recompaction activities. It will, therefore, be advantageous to perform construction activities during dry weather.

4.2 Drainage and Groundwater Considerations

Water should not be allowed to collect in the foundation excavations or on prepared subgrades for the floor slab during construction. Positive site drainage should be maintained throughout construction activities. Undercut or excavated areas should be sloped toward one corner to facilitate removal of any collected rainwater, groundwater, or surface runoff.

The site grading plan should be developed to provide rapid drainage of surface water away from the building areas and to inhibit infiltration of surface water around the perimeter of the building and beneath the floor slab. The grades should be sloped away from the building area. We anticipate stormwater will be disposed of towards the ravine located along the western boundary of the property.

4.3 Excavations

In Federal Register, Volume 54, No. 209 (October 1989), the United States Department of Labor, Occupational Safety and Health Administration (OSHA) amended its "Construction Standards for Excavations, 29 CFR, part 1926, Subpart P". This document and subsequent updates were issued to better insure the safety of workmen entering trenches or excavations. It is mandated by this federal regulation that excavations, whether they be utility trenches, basement excavations or footing excavations, be constructed in accordance with the new OSHA guidelines. It is our understanding that these regulations are being strictly enforced and if they are not closely followed, the owner and the contractor could be liable for substantial penalties.

The contractor is solely responsible for designing and constructing stable, temporary excavations and should shore, slope, or bench the sides of the excavations as required to maintain stability of both the excavation sides and bottom. The contractor's "responsible person", as defined in 29

CFR Part 1926, should evaluate the soil exposed in the excavations as part of the contractor's safety procedures. In no case should slope height, slope inclination, or excavation depth, including utility trench excavation depth, exceed those specified in local, state, and federal safety regulations.

We are providing this information solely as a service to our client. EEI does not assume responsibility for construction site safety or the contractor's compliance with local, state, and federal safety or other regulations.

5.0 REPORT LIMITATIONS

As is standard practice in the geotechnical industry, the conclusions contained in our report are considered preliminary because they are based on assumptions made about the soil, rock, and groundwater conditions exposed at the site during our subsurface investigation. A more complete extent of the actual subsurface conditions can only be identified when they are exposed during construction. Therefore, EEI should be retained as your consultant during construction to observe the actual conditions and to provide our final conclusions. If a different geotechnical consultant is retained to perform geotechnical inspection during construction, then they should be relied upon to provide final design conclusions and recommendations, and should assume the role of geotechnical engineer of record.

The geotechnical recommendations presented in this report are based on the available project information, and the subsurface materials described in this report. If any of the noted information is incorrect, please inform EEI in writing so that we may amend the recommendations presented in this report if appropriate and if desired by the client. EEI will not be responsible for the implementation of its recommendations when it is not notified of changes in the project.

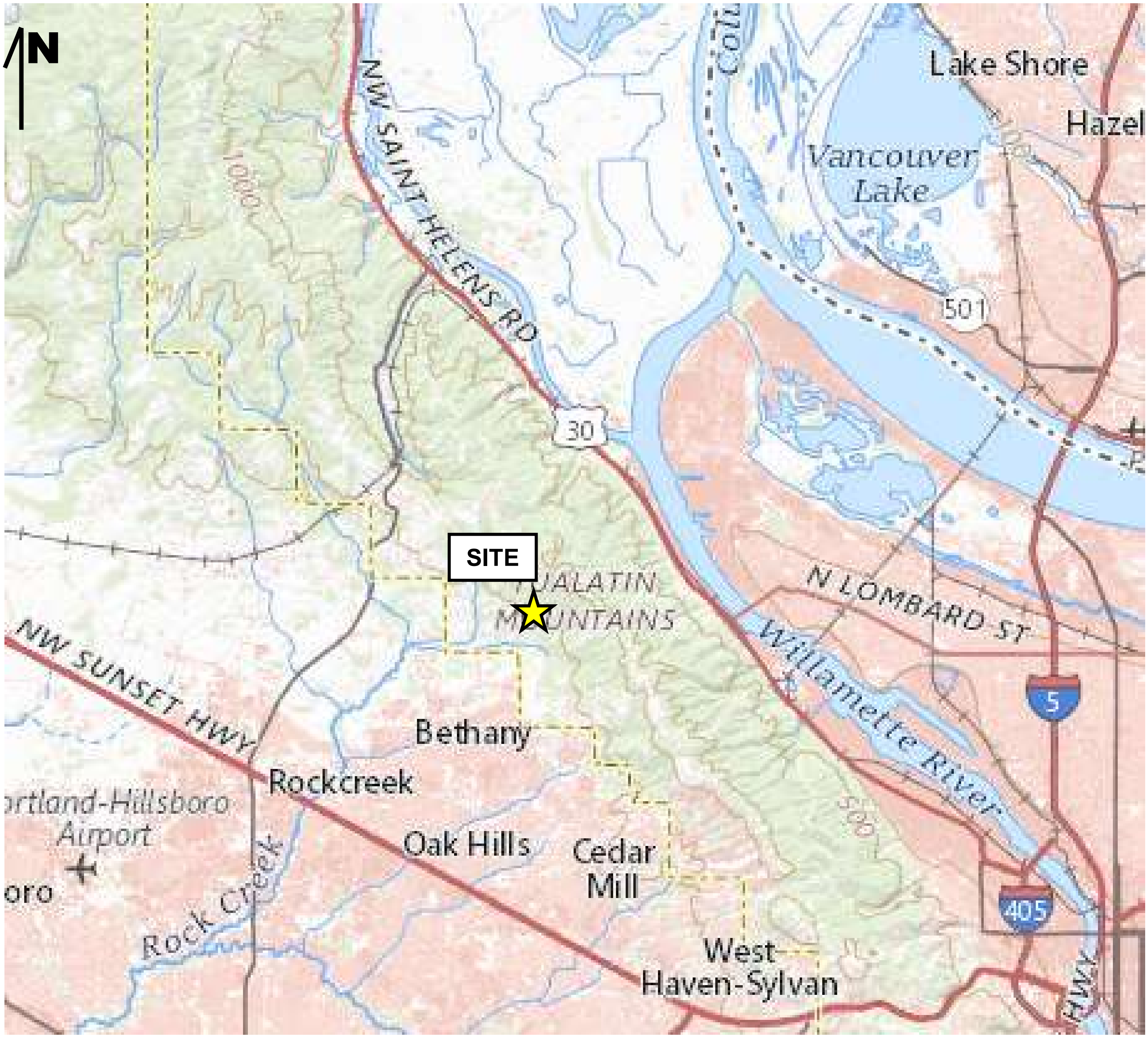
Once construction plans are finalized and a grading plan has been prepared, EEI should be retained to review those plans, and modify our existing recommendations related to the proposed construction, if determined to be necessary.

The Geotechnical Engineer warrants that the findings, recommendations, specifications, or professional advice contained herein have been made in accordance with generally accepted professional geotechnical engineering practices in the local area. No other warranties are implied or expressed.

This report has been prepared for the exclusive use of Kevin Spence and Amena Syed for the specific application to the proposed single family residence development to be located at 14180 Northwest Germantown Road, Portland, Multnomah County, Oregon. EEI does not authorize the use of the advice herein nor the reliance upon the report by third parties without prior written authorization by EEI.

APPENDICES

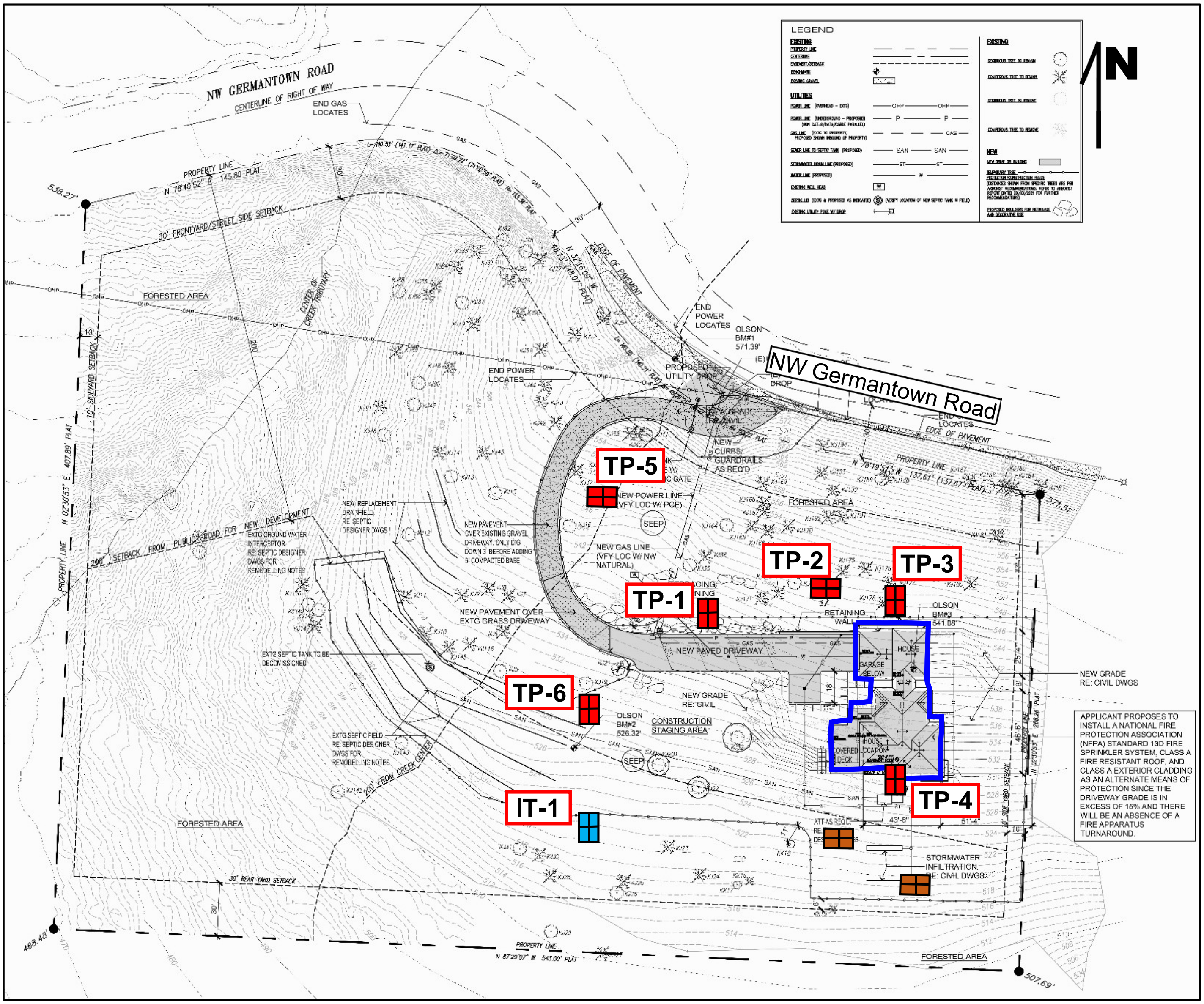
APPENDIX A – SITE LOCATION PLAN



Map Source: https://viewer.nationalmap.gov/advanced-viewer/

Earth Engineers, Inc.	Spence-Syed Residence 14180 Northwest Germantown Road Portland, Multnomah County, Oregon	**Report No.** **21-156-1-*R1***
		August 31, 2021 **(*Revised December 3, 2021*)**

APPENDIX B – SITE EXPLORATION PLAN



Base drawing source: Site Plan, A1.0, by Karina E. Adams Architect, dated November 10, 2021

Legend

- = Approximate Test Pit Locations
- = Approximate Infiltration Test Pit Location
- = Approximate Septic Test Pit Locations
- = Approximate Proposed Home Boundary

Earth Engineers, Inc.	**Spence-Syed Residence** **14180 Northwest Germantown Road** **Portland, Multnomah County, Oregon**	**Report No.** **21-156-1-*R1*** **August 31, 2021** ***Revised December 3, 2021***

Appendix C: Test Pit TP-1

Earth Engineers, Inc.

Client: Kevin Spence & Amena Syed
Project: Spence and Syed New SFR
Site Address: 14180 NW Germantown Rd
Portland, Oregon
Location of Exploration: See Appendix B
Logged By: J. Martin

Report Number: 21-156-1-R1
Excavation Contractor: Dan Fischer Excavating
Excavation Method: Backhoe with 2-foot toothed bucket
Excavation Equipment: CASE 580 Extendahoe
Approximate Ground Surface Elevation (ft msl): 538'
Date of Exploration: August 2, 2021

Depth (ft)	Water Level	Lithology: Lithologic Symbol	Lithology: Geologic Description of Soil and Rock Strata	Sampling Data: Sample Number	Digging Effort	Drive Probe Blows Per 6 Inches	Pocket Pen. (tsf)	Moisture Content (%)	% Passing #200 Sieve	Liquid Limit	Plastic Limit	Remarks
0			Topsoil - brown silt with roots, dry, soft becoming very stiff with depth		Easy	10						
						29						
1			Silt (ML) - brown, dry, very stiff	GRAB 1	Mod	26	4.5	10	67			
						20						
2			mottled with rootlets, decomposed course sand	GRAB 2		25	3.5	18	87			
						29						
3						26	4.25					
			mottled with gray clay			34						
4			boulder									
				GRAB 3	Hard	55		18				
			mottled with decomposed course sand									
5												
			broken rock									
6												
			dark brown, mottled with decomposed course sand, moist	GRAB 4				17				
7												
					Hard							
8			bottom of test pit due to digging refusal on rock									

9 10 11 12 13 14 15

Notes: Test pit terminated at a depth of approximately 8-feet bgs. Drive probe terminated at a depth of approximately 4.5-feet bgs. Groundwater was not encountered at the time of our exploration. Test pit loosely backfilled with excavated soil on 8/2/2021. Approximate elevation based off of Preliminary Drawings for Syed/Spence Residence, sheet A1.0, by Karina E. Adams, Architect, dated 6/23/2021. Mod = Moderate

Appendix C: Test Pit TP-2

Sheet 1 of 1

Earth Engineers, Inc.

Client: Kevin Spence & Amena Syed
Project: Spence and Syed New SFR
Site Address: 14180 NW Germantown Rd
Portland, Oregon
Location of Exploration: See Appendix B
Logged By: J. Martin

Report Number: 21-156-1-R1
Excavation Contractor: Dan Fischer Excavating
Excavation Method: Backhoe with 2-foot toothed bucket
Excavation Equipment: CASE 580 Extendahoe
Approximate Ground Surface Elevation (ft msl): 546'
Date of Exploration: August 2, 2021

Depth (ft)	Water Level	Lithology: Lithologic Symbol	Lithology: Geologic Description of Soil and Rock Strata	Sampling Data: Sample Number	Digging Effort	Drive Probe Blows Per 6 Inches	Pocket Pen. (tsf)	Moisture Content (%)	% Passing #200 Sieve	Liquid Limit	Plastic Limit	Remarks
0			Topsoil - brown silt with roots, dry, soft becoming hard with depth		Easy	26						
						50						drive probe refusal at 3-inches after 50-blows
1			Silt (ML) - brown, dry, hard		Hard							
2			plastic silt, dark brown, with rootlets, decompsed course sand	GRAB 1	Hard		4.5	34	89			
3			broken rock				4.5					
			bottom of test pit due to digging refusal on rock									

Notes: Test pit terminated at a depth of approximately 3.25-feet bgs. Drive probe terminated at a depth of 9-inches bgs. Groundwater was not encountered at the time of our exploration. Test pit loosely backfilled with excavated soil on 8/2/2021. Approximate elevation based off of Preliminary Drawings for Syed/Spence Residence, sheet A1.0, by Karina E. Adams, Architect, dated 6/23/2021.

Appendix C: Test Pit TP-3

Earth Engineers, Inc.

Client: Kevin Spence & Amena Syed
Project: Spence and Syed New SFR
Site Address: 14180 NW Germantown Rd
Portland, Oregon
Location of Exploration: See Appendix B
Logged By: J. Martin

Report Number: 21-156-1-R1
Excavation Contractor: Dan Fischer Excavating
Excavation Method: Backhoe with 2-foot toothed bucket
Excavation Equipment: CASE 580 Extendahoe
Approximate Ground Surface Elevation (ft msl): 539'
Date of Exploration: August 2, 2021

Depth (ft)	Water Level	Lithology: Lithologic Symbol	Lithology: Geologic Description of Soil and Rock Strata	Sampling Data: Sample Number	Digging Effort	Drive Probe Blows Per 6 Inches (0 20 40 60)	Pocket Pen. (tsf)	Moisture Content (%)	% Passing #200 Sieve	Liquid Limit	Plastic Limit	Remarks
0			Topsoil - brown silt with roots, dry, soft becoming stiff with depth		Easy	7						
						10						
1						13						
			Silt (ML) - brown, dry, very stiff			26	1.5					
2			mottled, dark brown with rust									
					Hard	33						
3			mottled, with decomposed course sand to fine gravel	GRAB 1		43		16				
						56						
			brown to tan									
4			brown with decomposed course sand to fine gravel rounded boulder	GRAB 2				19	69			
5			bottom of test pit due to digging refusal on rock									
6												
7												
8												
9												
10												
11												
12												
13												
14												
15												

Notes: Test pit terminated at a depth of approximately 5-feet bgs. Drive probe terminated at a depth of approximately 3.5-feet bgs. Groundwater was not encountered at the time of our exploration. Test pit loosely backfilled with excavated soil on 8/2/2021. Approximate elevation based off of Preliminary Drawings for Syed/Spence Residence, sheet A1.0, by Karina E. Adams, Architect, dated 6/23/2021.

Earth Engineers, Inc.

Appendix C: Test Pit TP-4

Client: Kevin Spence & Amena Syed
Project: Spence and Syed New SFR
Site Address: 14180 NW Germantown Rd
Portland, Oregon
Location of Exploration: See Appendix B
Logged By: J. Martin

Report Number: 21-156-1-R1
Excavation Contractor: Dan Fischer Excavating
Excavation Method: Backhoe with 2-foot toothed bucket
Excavation Equipment: CASE 580 Extendahoe
Approximate Ground Surface Elevation (ft msl): 528'
Date of Exploration: August 2, 2021

Depth (ft)	Water Level	Lithologic Symbol	Geologic Description of Soil and Rock Strata	Sample Number	Digging Effort	Drive Probe Blows Per 6 Inches	Pocket Pen. (tsf)	Moisture Content (%)	% Passing #200 Sieve	Liquid Limit	Plastic Limit	Remarks
0			**Topsoil** - brown silt with roots, dry, soft becoming hard with depth		Easy	10						
						30						
1			**Silt (ML)** - brown, dry, hard			33						
			mottled, decomposed course sand, dry	GRAB 1	Mod	26		18	91			
2						20						
			mottled with rust color			25						
3					Hard	28	4.5					
			moist			21						
4			mottled brown, decomposed course sand with plastic silt	GRAB 2		29	4.5	19	88			
						36						
5						25						
						20						
6			mottled brown, decomposed course sand, moist	GRAB 3	Hard	18		20	86			
						19						
7						16						
						17						
8					Mod	20						
						31						
9						33						
						28						
10					Mod	28						
						33						
11			mottled brown and gray, weathered fine gravel and course sand, moist weathered gravel	GRAB 4	Hard	42		33	10			
						44						
12						60						
13												
					Hard							
14												
			fractured/weathered, gravel to course sand	GRAB 4				28	56			
15			bottom of test pit due to digging refusal on rock									

Notes: Test pit terminated at a depth of approximately 15-feet bgs. Drive probe terminated at a depth of approximately 12.25-feet bgs. Groundwater was not encountered at the time of our exploration. Test pit loosely backfilled with excavated soil on 8/2/2021. Approximate elevation based off of Preliminary Drawings for Syed/Spence Residence, sheet A1.0, by Karina E. Adams, Architect, dated 6/23/2021. Mod = Moderate

Appendix C: Test Pit TP-5

Earth Engineers, Inc.

Sheet 1 of 1

Client: Kevin Spence & Amena Syed	Report Number: 21-156-1-R1
Project: Spence and Syed New SFR	Excavation Contractor: Dan Fischer Excavating
Site Address: 14180 NW Germantown Rd	Excavation Method: Backhoe with 2-foot toothed bucket
Portland, Oregon	Excavation Equipment: CASE 580 Extendahoe
Location of Exploration: See Appendix B	Approximate Ground Surface Elevation (ft msl): 547'
Logged By: J. Martin	Date of Exploration: August 2, 2021

Depth (ft)	Water Level	Lithologic Symbol	Geologic Description of Soil and Rock Strata	Sample Number	Digging Effort	Drive Probe Blows Per 6 Inches (0 20 40 60)	Pocket Pen. (tsf)	Moisture Content (%)	% Passing #200 Sieve	Liquid Limit	Plastic Limit	Remarks
0			Topsoil - brown silt with roots, dry, soft		Easy							
1			Silt (ML) - brown, dry, hard									
			light gray, dry									
			dark brown, mottled plastic silt, dry	GRAB 1	Hard			17				
2							4					
3												
4			dark brown, dry, hard									
5			dark brown, mottled, decomposed course sand, dry	GRAB 2				17				
			decomposed rock									
6			bottom of test pit due to digging refusal on rock									
7												
8												
9												
10												
11												
12												
13												
14												
15												

Notes: Test pit terminated at a depth of approximately 6 feet bgs. Drive probe testing not attempted at this location. Groundwater was not encountered at the time of our exploration. Test pit loosely backfilled with excavated soil on 8/2/2021. Approximate elevation based off of Preliminary Drawings for Syed/Spence Residence, sheet A1.0, by Karina E. Adams, Architect, dated 6/23/2021.

Appendix C: Test Pit TP-6

Earth Engineers, Inc.

Sheet 1 of 1

Client: Kevin Spence & Amena Syed
Project: Spence and Syed New SFR
Site Address: 14180 NW Germantown Rd
Portland, Oregon
Location of Exploration: See Appendix B
Logged By: J. Martin

Report Number: 21-156-1-R1
Excavation Contractor: Dan Fischer Excavating
Excavation Method: Backhoe with 2-foot toothed bucket
Excavation Equipment: CASE 580 Extendahoe
Approximate Ground Surface Elevation (ft msl): 532'
Date of Exploration: August 2, 2021

Depth (ft)	Water Level	Lithologic Symbol	Geologic Description of Soil and Rock Strata	Sample Number	Digging Effort	Drive Probe Blows Per 6 Inches (0 20 40 60)	Pocket Pen. (tsf)	Moisture Content (%)	% Passing #200 Sieve	Liquid Limit	Plastic Limit	Remarks
0			Topsoil - brown silt with roots, dry, soft		Easy							
1			Silt (ML) - brown, dry, hard									
					Mod							
2			dark brown									
			dark brown plastic silt, mottled, decomposed course sand, dry	GRAB 1				14				
3			mottled, rust color with some clay									
					Hard							
4			dark brown plastic silt, mottled, decomposed course sand, dry	GRAB 2				13				
5												
6					Hard							
7												
8												
9			brown, mottled, decomposed course sand, dry	GRAB 3				19				
10												
11			decomposed rock									
				GRAB 4				15				
12			bottom of test pit due to digging refusal on rock									
13												
14												
15												

Notes: Test pit terminated at a depth of approximately 12 feet bgs. Drive probe testing not attempted at this location. Groundwater was not encountered at the time of our exploration. Test pit loosely backfilled with excavated soil on 8/2/2021. Approximate elevation based off of Preliminary Drawings for Syed/Spence Residence, sheet A1.0, by Karina E. Adams, Architect, dated 6/23/2021. Mod = Moderate

APPENDIX D: SOIL CLASSIFICATION LEGEND

APPARENT CONSISTENCY OF COHESIVE SOILS (PECK, HANSON & THORNBURN 1974, AASHTO 1988)				
Descriptor	**SPT N_{60} (blows/foot)***	**Pocket Penetrometer, Qp (tsf)**	**Torvane (tsf)**	**Field Approximation**
Very Soft	< 2	< 0.25	< 0.12	Easily penetrated several inches by fist
Soft	2 – 4	0.25 – 0.50	0.12 – 0.25	Easily penetrated several inches by thumb
Medium Stiff	5 – 8	0.50 – 1.0	0.25 – 0.50	Penetrated several inches by thumb w/moderate effort
Stiff	9 – 15	1.0 – 2.0	0.50 – 1.0	Readily indented by thumbnail
Very Stiff	16 – 30	2.0 – 4.0	1.0 – 2.0	Indented by thumb but penetrated only with great effort
Hard	> 30	> 4.0	> 2.0	Indented by thumbnail with difficulty

* Using SPT N_{60} is considered a crude approximation for cohesive soils.

APPARENT DENSITY OF COHESIONLESS SOILS (AASHTO 1988)	
Descriptor	**SPT N_{60} Value (blows/foot)**
Very Loose	0 – 4
Loose	5 – 10
Medium Dense	11 – 30
Dense	31 – 50
Very Dense	> 50

MOISTURE (ASTM D2488-06)	
Descriptor	**Criteria**
Dry	Absence of moisture, dusty, dry to the touch, well below optimum moisture content (per ASTM D698 or D1557)
Moist	Damp but no visible water
Wet	Visible free water, usually soil is below water table, well above optimum moisture content (per ASTM D698 or D1557)

PERCENT OR PROPORTION OF SOILS (ASTM D2488-06)	
Descriptor	**Criteria**
Trace	Particles are present but estimated < 5%
Few	5 – 10%
Little	15 – 25%
Some	30 – 45%
Mostly	50 – 100%
Percentages are estimated to nearest 5% in the field. Use "about" unless percentages are based on laboratory testing.	

SOIL PARTICLE SIZE (ASTM D2488-06)	
Descriptor	**Size**
Boulder	> 12 inches
Cobble	3 to 12 inches
Gravel - Coarse Fine	¾ inch to 3 inches No. 4 sieve to ¾ inch
Sand - Coarse Medium Fine	No. 10 to No. 4 sieve (4.75mm) No. 40 to No. 10 sieve (2mm) No. 200 to No. 40 sieve (.425mm)
Silt and Clay ("fines")	Passing No. 200 sieve (0.075mm)

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D2488)				
Major Division			**Group Symbol**	**Description**
Coarse Grained Soils (more than 50% retained on #200 sieve)	**Gravel** (50% or more retained on No. 4 sieve)	Clean Gravel	GW	Well-graded gravels and gravel-sand mixtures, little or no fines
			GP	Poorly graded gravels and gravel-sand mixtures, little or no fines
		Gravel with fines	GM	Silty gravels and gravel-sand-silt mixtures
			GC	Clayey gravels and gravel-sand-clay mixtures
	Sand (> 50% passing No. 4 sieve)	Clean sand	SW	Well-graded sands and gravelly sands, little or no fines
			SP	Poorly-graded sands and gravelly sands, little or no fines
		Sand with fines	SM	Silty sands and sand-silt mixtures
			SC	Clayey sands and sand-clay mixtures
Fine Grained Soils (50% or more passing #200 sieve)	**Silt and Clay** (liquid limit < 50)		ML	Inorganic silts, rock flour and clayey silts
			CL	Inorganic clays of low-medium plasticity, gravelly, sandy & lean clays
			OL	Organic silts and organic silty clays of low plasticity
	Silt and Clay (liquid limit > 50)		MH	Inorganic silts and clayey silts
			CH	Inorganic clays or high plasticity, fat clays
			OH	Organic clays of medium to high plasticity
Highly Organic Soils			PT	Peat, muck and other highly organic soils

Earth Engineers, Inc.

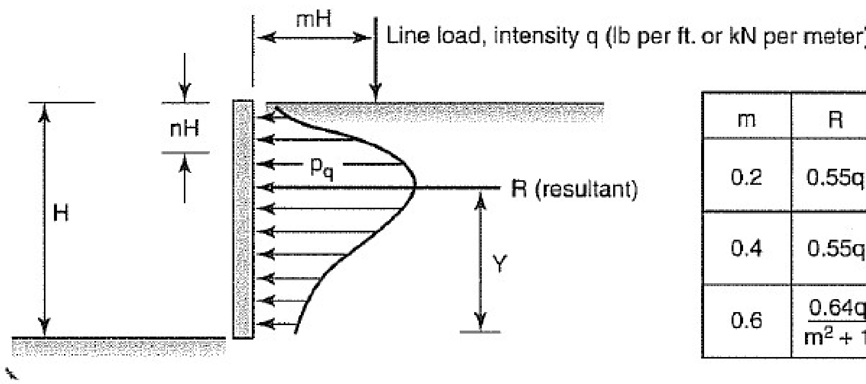
GRAPHIC SYMBOL LEGEND	
GRAB	Grab sample
SPT	Standard Penetration Test (2" OD), ASTM D1586
ST	Shelby Tube, ASTM D1587 (pushed)
DM	Dames and Moore ring sampler (3.25" OD and 140-pound hammer)
CORE	Rock coring

APPENDIX E

SURCHARGE-INDUCED LATERAL EARTH PRESSURES FOR WALL DESIGN

APPENDIX E: SURCHARGE-INDUCED LATERAL EARTH PRESSURES FOR WALL DESIGN

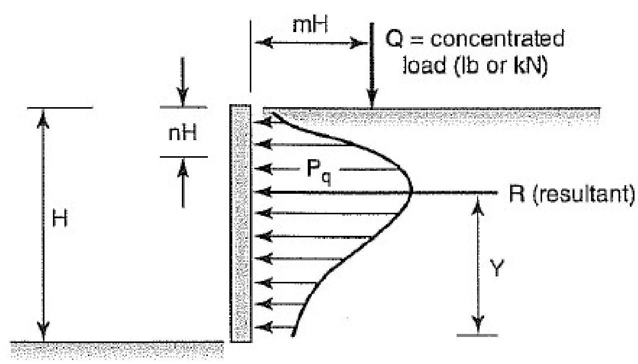
LINE LOAD (applicable for retaining walls not exceeding 20 feet in height):



m	R	Y
0.2	0.55q	0.60 H
0.4	0.55q	0.58 H
0.6	$\frac{0.64q}{m^2+1}$	0.52 H

Figure 16-28 Pressure distribution against vertical wall resulting from line load of intensity *q*.

CONCENTRATED POINT LOAD (applicable for retaining walls not exceeding 20 feet in height):



m	R	Y
0.2	$0.78\,\frac{Q}{H}$	0.59 H
0.4	$0.78\,\frac{Q}{H}$	0.59 H
0.6	$0.48\,\frac{Q}{H}$	0.48 H

Figure 16-27 Pressure distribution against vertical wall resulting from point load, *Q*.

AREAL LOAD:

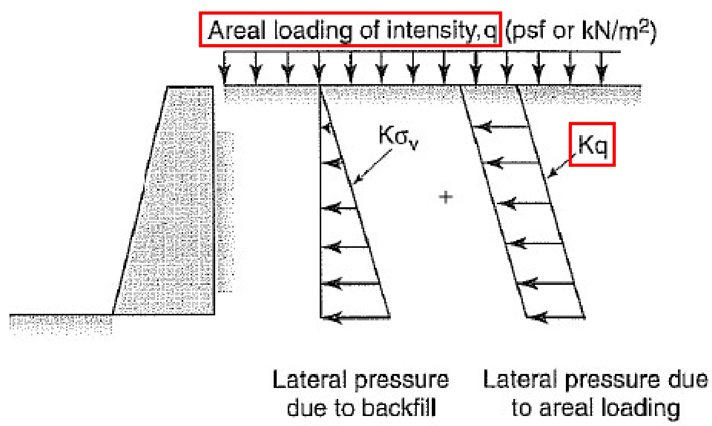
Figure 16-26 Influence of areal loading on wall pressures.

use K=0.4 for active condition (i.e. top of wall allowed to deflect laterally)

use K=0.9 for at-rest condition (i.e. top of wall not allowed to deflect laterally)

Resultant, R = K * q * H

Where H = wall height (feet)



Source of Figures: McCarthy, D.F., 1998, "Essentials of Soil Mechanics and foundations, Basic Geotechnics, Fifth Edition."