Geotechnical Investigation

29619 SE Stone Rd.
Gresham, Oregon

Prepared for:
Marin & Daniyela Palamaryuk
12 February 2019

REGISTERED PROFESSIONAL ENGINEER
19244PE
OREGON
JULY 15, 1997
MIA C. MAHEDY
EXPIRES: 12-31-2020

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EXHIBIT
A.15

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SUPPORTING DATA

1.0 PROJECT AND SITE DESCRIPTIONS

Rapid Soil Solutions (RSS) has prepared this geotechnical report, as requested, for the proposed new single-family residence to be constructed on the tax lot currently assigned the street address of 29619 SE Stone Road in Gresham, Oregon (97080). RSS understands that a new residential dwelling will be constructed within the northeastern corner of the subject site. The new residence will be accessed via the existing driveway that is located off the western edge of SE Short Road. The subject site is bounded to the east by SE Short Road and to the south by SE Stone Road. The proposed building site is accessed via a driveway about 200 feet north of SE Stone Roads intersection with SE Short Road. The site is situated next to rural residential properties with the street addresses of 8137 SE Short Rd. (north) and 29425 SE Stone Rd. (west). The subject site is 1.36 miles north of Mt. Hood Highway (Hwy-26), 0.67 miles east of SE 82nd Ave., 0.37 miles south of SE Orient Dr., 0.5 miles west of SE Clark Rd. and 5.54 miles south of Interstate-84. The site can be found in the southeast quarter of Section 19, Township 1-South and Range 4-East W.M. in Multnomah County. The tax lot identification number is 00200 (State ID: 1S4E19DC-00200) and the alternate/R number is R994190890. The latitude and longitude of the site are 45.463107 and -122.35841 (45°27'47.1"N, 122°21'30.2"W). See Appendix A, Figure 1 for site location. Subsequent figures include additional site location information.

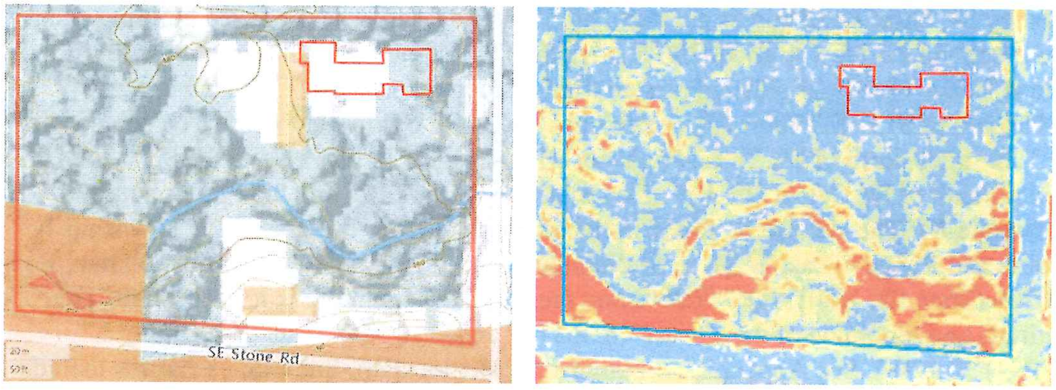
2.0 SITE CONDITIONS

2.1 Surface Conditions

This 2.66-acre subject site is situated in the Sandy River neighborhood of southeast Gresham in unincorporated Multnomah County. The site and surrounding tax lots are all zoned RUR, rural residential. The surrounding tax lots are all developed with single-family residences. Currently the subject site contains a single-family residence along the southern property line directly off of SE Stone Road. Historical records indicate that the residence was originally constructed in 1970. RSS viewed historical aerial imagery starting in 1994 as part of this investigation. RSS found that in 1994, an existing rectangular residence was located along the western edge of the proposed residence location. The residence is visible from 1994 until 2017. The residence was demolished sometime between 2017 to 2018. In late 2018, the area is clear of the previous residence and covered in grass and gravel. At the time of the site visit, RSS observed that the proposed residence area was relatively smooth and covered in grass. The proposed residence corners were staked on site following the provided site plan.

The slopes on site within the proposed construction area are relatively smooth. The highest elevation of the site is mapped as 492 feet above sea level located within the southeastern corner of the site. The slope descends northwards towards Johnson Creek which crosses the site E-W near the center of the parcel. The elevation surrounding the creek is about 478 feet. The elevation ascends northwards of the creek towards the proposed area of development. The slopes surrounding the new residence start at 480 feet in elevation and ascend to 482 feet within the northeastern corner. The slope model derived from the 5-ft DEM of Portland indicates that the majority of the northern half of the parcel contains slopes of less than 5% (blue) with a few small patches mapped as containing slopes of 5-10% (green) and 10-15%

(yellow). The southern half of the parcel containing Johnson Creek is modeled at mostly containing slopes of 10-15% (yellow), 15-20% (orange) and greater than 20% (red). Metromap classifies the entire subject site as containing slopes less than 10% (no overlay), while the southwestern corner of the tax lot is mapped as containing slopes greater than 10% (orange overlay) and one small stretch of steep slopes along the creek are mapped as greater than 25% (red overlay). The slopes observed by RSS during the field exploration were consistent with the near zero percent mapped slopes within the proposed building envelope.



2.2 Regional Geology

Current geologic literature[1,2,3] classifies the slopes underlying the subject site as part of the Pliocene to Pleistocene aged Boring Volcanic flows overlain by a thick surficial deposit of wind-blown loess and colluvium.

Geologic History

The subject site is situated generally in the eastern edge of the Portland Basin. The Portland Basin is part of the series of topographic and structural depressions that constitute the Puget-Willamette forearc trough of the Cascadia subduction system. It is a relatively low-relief valley, characterized by broad, flat, lowlands surrounded by prominent uplands controlled primarily by structural features (faulting and folding) in the underlying bedrock. The tectonic compressional stress that is associated with the subduction zone, and associated mountain building to both the east and west of the foearc trough, both initiated basin development and produced a prolonged enlargement of the structural feature. This basin contains a thick accumulation of material that preserves a complex record of deposition and erosion (aggradation and incision) produced by the lakes and rivers that that flowed through the basin

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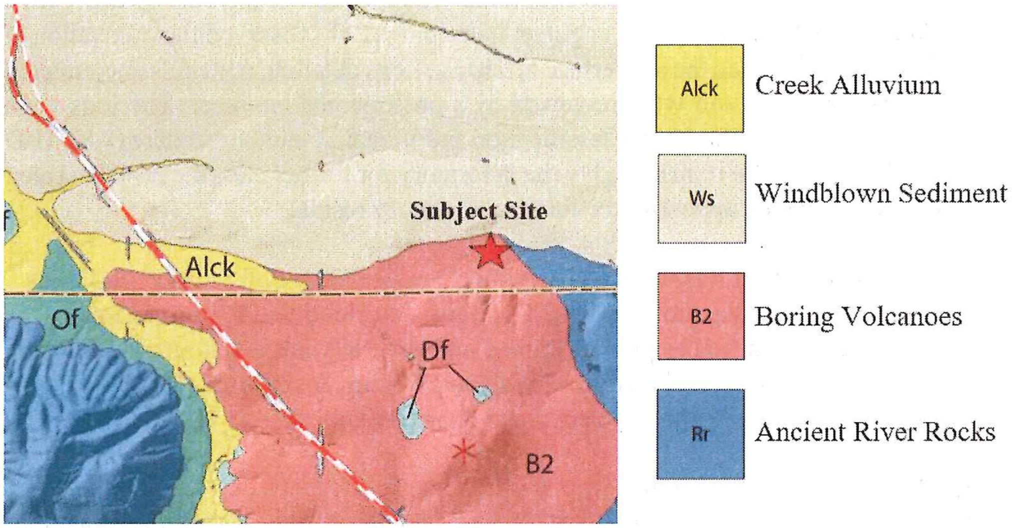
[1] Ma, L., Madin, I.P., Duplantis, S., and Williams, K.J., 2012, *Lidar-based surficial geologic map and database of the greater Portland, Oregon, area, Clackamas, Columbia, Marion, Multnomah, Washington, and Yamhill Counties, Oregon, and Clark County, Washington*: Oregon Department of Geology and Mineral Industries, Open-File Report 0-2012-02, scale 1:8,000.

[2] Treasher, R.C., 1942, *Geologic map of the Portland area*: Oregon Department of Geology and Mineral Industries, Quadrangle Map 9, scale 1:96,000.

[3] Trimble, D.E., 1957, *Geology of the Portland quadrangle, Oregon-Washington*: U.S. Geological Survey, Geologic Quadrangle Map GQ-104, scale 1:62,500.

concurrent with its development.

The Portland Basin is dotted with dozens of young extinct volcanoes known as the Boring volcanic field. These Boring eruptive events formed isolated hills and hill clusters that rise up to 200 meters above the surrounding landscape. Many of the hills are monogenetic volcanoes that erupted west of the Cascade arc axis beginning in latest Pliocene time. Boring centers consist of cinder cones and associated lava flows, small shields and lava cones. Typical eruptive events were brief, during which the volcano built up a cinder cone, which may have then filled with a lava lake that eventually broke out of the crater and flowed across the surrounding landscape. One such vent is situated just south of the subject site.



Site Geology

As a whole the boring volcanic deposits are typically described as light-grey to grey, diktytaxitic, olivine-phyric (less commonly plagioclase-phyric) basalt and basaltic andesite. The igneous flows erupted from a series of local vents across the local basins. The cones are generally comprised of interstratified cinders and lava. The lava flows typically display block to columnar jointing and, if preserved, vesicular flow tops. The Boring cinders and lavas weather very rapidly in the wet Portland climate. Cinder cones become masses of red and yellow clay, and lava flows decompose into large rounded core-stones surrounded by sticky red clay. These deposits tend to be poorly exposed due to variable degrees of deep wreathing and/or mantling by loess.

Throughout the mapped area, the bedrock deposits are draped in a thick layer of loess and/or colluvium. The wind-blown loess deposits are comprised of quartzo-feldspathic silt. The colluvium is composed of loess deposits mixed with clay and sand derived from weathered basalt, and sometimes includes angular pebble- to boulder-sized clasts of basalt and/or weathered basalt. Near the tops of slopes, the surficial material is primarily composed of loess-derived silt while near the bottom of slopes the surficial deposit can become increasingly derived from basalt and weathered basalt. On ridge crests the loess layer is generally stable, but as slopes steepen and descend into the nearby canyons, the loess typically moves slowly downhill under the influence of gravity, water, and numerous small landslides. This blanket of moving and mixing colluvium includes fragments of the underlying bedrock. When exposed in road cuts or excavations, the loess is featureless and tan. When saturated by heavy rainfall or runoff, the loess and loess-colluvium mix is very susceptible to landslides and can liquefy to flow down slopes.

2.3 Field Exploration and Subsurface Conditions

2.3.1 Field Explorations

Two (2) hand auger borings were excavated. The location of the borings are shown on Figure 2 in Appendix A. An engineer in training (EIT) observed the excavation of the borings and logged the subsurface materials. Logs detailing materials encountered are in the appendix and were reviewed by a professional engineer. The logs were created using the Unified Soil Classification and Visual Manual Procedure (ASTM-D 2488). Samples were transported to the laboratory for further classification in sealed bags. Please see the appendix for further laboratory results.

The USDA National Resource Conservation Service Web Soil Survey[4] classifies the soils on site as Wapato silt loam. This forms on flood plains from recent alluvium. It is classified as poorly drained and generally has a water table depth of 0 to 12 inches. The typical profile of Wapato silt loam is silt loam (H1: 0"-18", H2: 18"-45") and very gravelly sandy clay loam (H3: 45"-60").

2.3.2 Subsurface Conditions

The soil conditions were stiff silty-CLAY to a depth of 4 feet. Moisture content ranged from 36.0 to 36.5%.

2.3.3 Groundwater

Groundwater was encountered at 3.5 feet below the ground surface in HA#2.

3.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

3.1 Foundation Design

The building foundations may be installed on either engineered fill or firm native sub-grade that is found at a depth of about 5 to 7 inches. This depth may be locally variable and should be confirmed by a geotechnical engineer or their representative at the time of construction. ***Please allow 24hours notice to call for foundation inspections.***

Continuous wall and isolated spread footings should be at least 16 and 24 inches wide,

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[4] http://websoilsurvey.sc.egov.usda.gov/App/WebSoilSurvey.aspx

respectively. The bottom of exterior footings should be at least 16 inches below the lowest adjacent exterior grade. The bottom of interior footings should be at least 12 inches below the base of the floor slab.

Footings placed on engineered fill or firm native sub-grade should be designed for an allowable bearing capacity of *1,500 pounds per square foot* (**psf**). The recommended allowable bearing pressure can be doubled for short-term loads such as those resulting from wind or seismic forces.

Based on our analysis the total post-construction settlement is calculated to be less than 1 inch, with differential settlement of less than 0.5 inch over a 50-foot span for maximum column, perimeter footing loads of less than 100 kips and 6.0 kips per linear foot.

Lateral loads on footings can be resisted by passive earth pressure on the sides of the structures and by friction at the base of the footings. An allowable lateral bearing pressure of 150 *pounds per cubic foot* (**psf/f**) below grade may be used. Adjacent floor slabs, pavements or the upper 12-inch depth of adjacent, unpaved areas should not be considered when calculating passive resistance.

If construction is undertaken during wet weather, we recommend a thin layer of compacted, crushed rock be placed over the footing sub-grades to help protect them from disturbance due to the elements and foot traffic.

If construction is undertaken during periods of rain, then I recommend a 2-inch (or greater) layer of compacted, crushed rock be placed over the native soil. The clayey soil is moisture sensitive. Meaning when dry it is firm and non-yielding but exposed to season rains it will lose its strength and need to be excavated and replaced with rock. See section 4.1.2 for wet weather conditions.

3.2 Retaining Walls and Embedded Walls

Default lateral soil load for the design of basement and retaining walls supporting level backfill shall be 35 psf/ft for laterally unrestrained retaining walls and 60 psf/ft for laterally restrained retaining walls.

For embedded building walls, a superimposed seismic lateral force should be calculated based on a dynamic force of $5H^2$ pounds per lineal foot of wall, where H is the height of the wall in feet and applied at 1/3 H from the base of the wall. The wall footings should be designed in accordance with the guidelines provided in the "Foundation Design" section of this report. These design parameters have been provided assuming that back-of-wall drains will be installed to prevent buildup of hydrostatic pressures behind all walls.

The backfill material placed behind the walls and extending a horizontal distance equal to at least half of the height of the retaining wall should consist of granular retaining wall backfill as specified in the "Structural Fill" section of this report. The wall backfill should be compacted to a minimum of 95 percent of the maximum dry density, as determined by ASTM D698. However, backfill located within a horizontal distance of 3 feet from the retaining walls should only be compacted to approximately 92 percent of the maximum dry density, as determined by ASTM D698. Backfill placed within 3 feet of the wall should be compacted in lifts less than 6 inches thick using hand-operated tamping equipment (e.g., jumping jack or vibratory plate compactors). If flat work (e.g., sidewalks or pavements) will be placed atop the wall backfill, we recommend that the upper 2 feet of material be compacted to 95 percent of the maximum dry density, as determined by ASTM D698.

A minimum 12-inch-wide zone of drain rock, extending from the base of the wall to within 6 inches of finished grade, should be placed against the back of all retaining walls. Perforated collector pipes should be embedded at the base of the drain rock. The drain rock should meet the requirements provided in the "Structural Fill" section of this report. The perforated collector pipes should discharge at an appropriate location away from the base of the wall. The discharge pipe(s) should not be tied directly into storm water drain systems, unless measures are taken to prevent backflow into the wall's drainage system. Settlements of up to 1 percent of the wall height commonly occur immediately adjacent to the wall as the wall rotates and develops active lateral earth pressures.

Engineering values summary

Bearing capacity soil	1,500psf
Bearing capacity rock	2,500psf
Coefficient of friction soil	0.30
Coefficient of friction rock	0.45
Active pressure	40pcf
Passive pressure	300pcf

A safety factor of 1.5 is included in the above values.

3.3 Seismic Design Criteria

We understand that the seismic design criteria for this project is based on the 2012/15 IBC, Section 1615 and the USGS web site using a Lat of 45.463107 and a Long of -122.35841, soil site class D.

	Short Period	**1 Second**
Maximum Credible Earthquake Spectral Acceleration	Ss = 0.843 g	S1 = 0.359 g
Adjusted Spectral Acceleration	Sms = 0.980 g	Sm1 = 0.604 g
Design Spectral Response Acceleration Perimeters	Sds = 0.654 g	Sd1= 0.403 g

3.4 Geohazard Review

The Oregon HazVu: Statewide Geohazard Viewer[5] and Metromap[6] were reviewed on 5

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[5] http://www.oregongeology.org/hazvu/

[6] http://gis.oregonmetro.gov/metromap/

February 2019 to investigate mapped geological hazards. This review indicates that the subject site is situated within the effective 100-year floodplain of Johnson Creek, as mapped by FEMA. However the house is placed outside this area. The expected earthquake-shaking hazard is classified as 'very strong' and contains a mapped liquefaction hazard classification of 'high'. The nearest mapped fault classified as active by DOGAMI is the SW-NE oriented Boring Fault passing roughly 1.7-miles southwest of the subject site. There are no landslides mapped on or adjacent to the subject site. The nearest mapped landslide is located about 0.15 miles southeast of the subject site. There are two mapped debris flows on the southern half of the subject site. The debris flows are mapped as relatively historic deposits which are presumed to have failed sometime over 150 years ago. Debris flows occur during periods of heavy rain, which creates cone-shaped fans of mud, rock and debris where steep streams reach flatter ground. The landslide hazard at the subject site is classified as 'low' landslide susceptibility.

No indications of significant slope instability were observed on the subject site. No tension cracks, sharp scarps, slump blocks, rotated blocks or other indications of recent instability were observed. No slump blocks or sag ponds were observed. No unusual vegetation was observed on slopes that could indicate season springs.

In our opinion, the proposed future development, following the provided recommendations, will not increase the risk of unstable slopes on or adjacent to the project site. Adequate drainage and appropriate grading will be required to prevent water-related issues in the wet season. Surface backfill for foundation walls should be sloped away to lessen the likelihood of saturated foundation subgrades. Gutter and foundation drains should be discharged in a manner and at sufficient distance from the foundations to prevent saturation and erosion damage.

3.5 Infiltration Testing

Rapid Soil Solutions (RSS) attempted one (1) infiltration test at a depth of 4 feet. At a depth of 3.5 feet below the ground surface, RSS found shallow groundwater. RSS does not recommend onsite infiltration for the site due to the shallow groundwater. RSS recommends the new residence use downspout splash blocks. With one every 700 sq.ft. of roof area for storm water disposal.

4.0 CONSTRUCTION RECOMMENDATIONS

4.1 Site Preparation

On this site only disturb the area in which can be covered with rock during the day. The moisture sensitive silty-CLAY soil when exposed to wet weather becomes soft and yielding. See wet weather conditions below.

4.1.1 Proof Rolling

Following stripping and prior to placing aggregate base course, pavement the exposed sub-grade should be evaluated by proof rolling. The sub-grade should be proof rolled to identify soft, loose, or unsuitable areas. Please give 24 hour notice to observe the proof rolling. Soft or loose zones identified during the field evaluation should be compacted to an unyielding condition or be excavated and replaced with structural fill, as discussed in the *Structural Fill* section of this report.

4.1.2 Wet Weather Conditions

The near-surface soils will be difficult during or after extended wet periods or when the moisture content of the surface soil is more than a few percentage points above optimum. Soils that have been disturbed during site preparation activities, or soft or loose zones identified during probing or proof rolling, should be removed and replaced with compacted structural fill. Track-mounted excavating equipment will be required during wet weather. The imported granular material should be placed in one lift over the prepared, undisturbed sub-grade and compacted using a smooth drum, non-vibratory roller. Additionally, a geo-textile fabric should be placed as a barrier between the sub-grade and imported granular material in areas of repeated traffic.

4.2 Excavation

Subsurface conditions of accessible cleared areas of the project site show predominately silty-CLAY to the depth explored (4.0 feet). Excavations in the upper soils may be readily accomplished with conventional earthwork equipment with smooth faced bucket.

4.3 Structural Fills

Fills should be placed over sub-grade prepared in compliance with Section 4.1 of this report. Material used, as structural fill should be free of organic matter or other unsuitable materials and should meet specifications provided in OSSC, depending upon the application. A discussion of these materials is in the following sections.

4.3.1 Native Soils

Laboratory testing indicates that the moisture content of the near-surface is greater than the optimum moisture content of the soil required for satisfactory compaction. This is depending on the weather conditions at the time of excavation. See section 4.3.2 for imported granular fill.

4.3.2 Imported Granular Fill

The imported granular material must be reasonably well graded to between coarse and fine material and have less than 5% by weight passing the US Standard No.200 Sieve. Imported granular material should be placed in lifts 8 to12 inches and be compacted to at least 95% of the maximum dry density, as determined by ASTM D 698. Where imported granular material is placed over wet or soft soil sub-grades, we recommend that a geo-textile serve as a barrier between the sub-grade and imported granular material.

4.4 Drainage Considerations

The Contractor shall be made responsible for temporary drainage of surface water and groundwater as necessary to prevent standing water and/or erosion at the working surface. We recommend removing only the foliage necessary for construction to help minimize erosion. Slope the ground surface around the structures to create a minimum gradient of 2% away from the building foundations for a distance of at least 5 feet. Surface water should be directed away from all buildings into drainage swales or into a storm drainage system. Foundation drains are required.

5.0 CONSTRUCTION OBSERVATIONS

Satisfactory pavement and earthwork performance depends on the quality of construction. Sufficient monitoring of the activities of the contractor is a key part of determining that the work is completed in accordance with the construction drawings and specifications. I recommend that a geotechnical engineer observe general excavation, stripping, fill placement, and sub-grades in addition to base. Subsurface conditions observed during construction should be compared with those encountered during the subsurface explorations. Recognition of changed conditions requires experience. Therefore, qualified personnel should visit the site with sufficient frequency to detect whether subsurface conditions change significantly from those anticipated.

6.0 LIMITATIONS

This report has been prepared for the exclusive use of the addressee, and their architects and engineers for aiding in the design and construction of the proposed development. It is the addressee's responsibility to provide this report to the appropriate design professionals, building officials, and contractors to ensure correct implementation of the recommendations.

The opinions, comments and conclusions presented in this report were based upon information derived from our literature review, field investigation, and laboratory testing. Conditions between, or beyond, our exploratory borings may vary from those encountered. Unanticipated soil conditions and seasonal soil moisture variations are commonly encountered and cannot be fully determined by merely taking soil samples or soil borings. Such variations may result in changes to our recommendations and may require that additional expenditures be made to attain a properly constructed project. Therefore, some contingency fund is recommended to accommodate such potential extra costs.

If there is a substantial lapse of time between the submission of this report and the start of work at the site; if conditions have changed due to natural causes or construction operations at, or adjacent to, the site; or, if the basic project scheme is significantly modified from that assumed, it is recommended this report be reviewed to determine the applicability of the conclusions and recommendations.

The work has been conducted in general conformance with the standard of care in the field of geotechnical engineering currently in practice in the Pacific Northwest for projects of this nature and magnitude. No warranty, express or implied, exists on the information presented in this report. By utilizing the design recommendations within this report, the addressee acknowledges and accepts the risks and limitations of development at the site, as outlined within the report.

APPENDIX

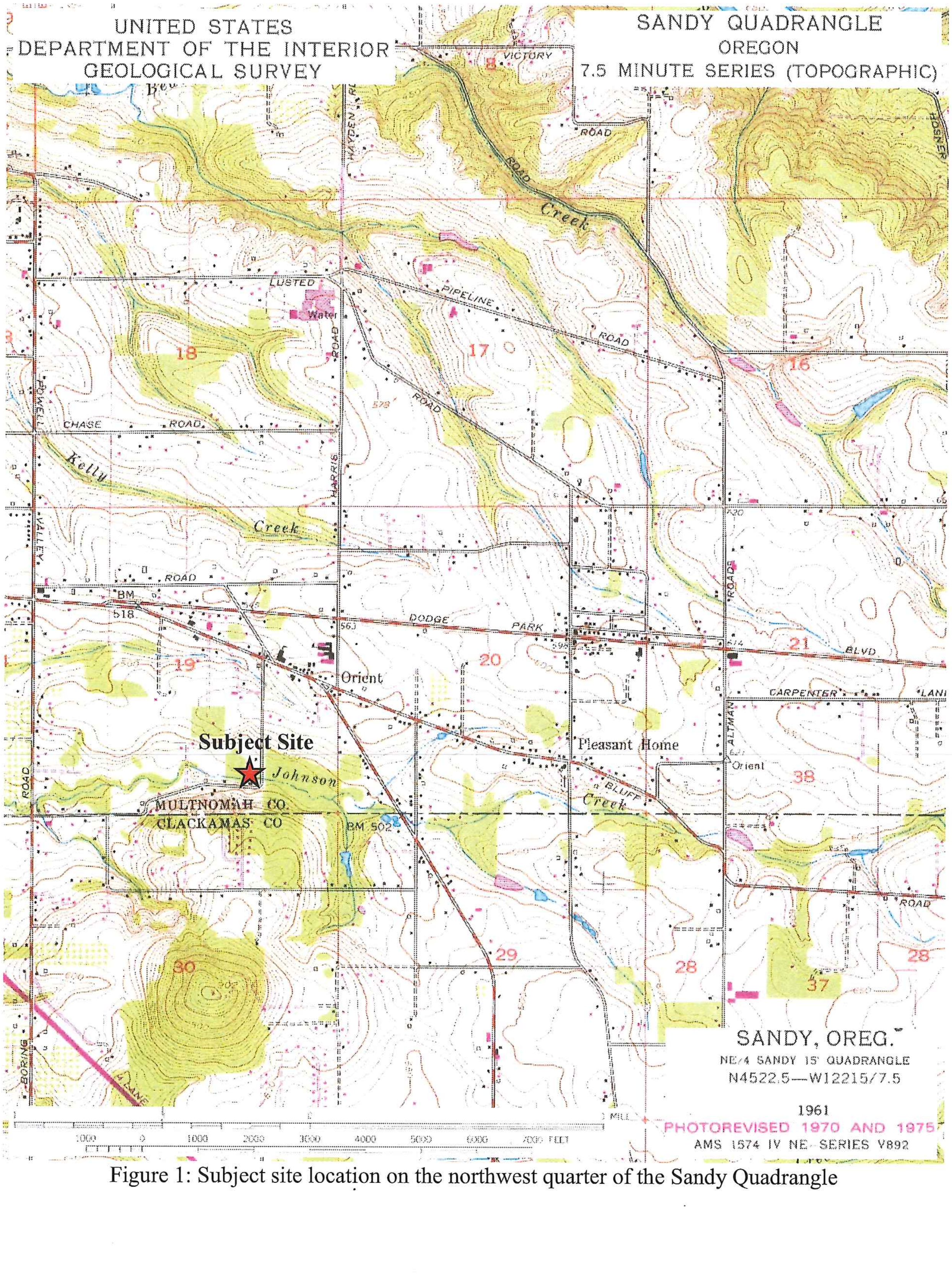


Figure 1: Subject site location on the northwest quarter of the Sandy Quadrangle

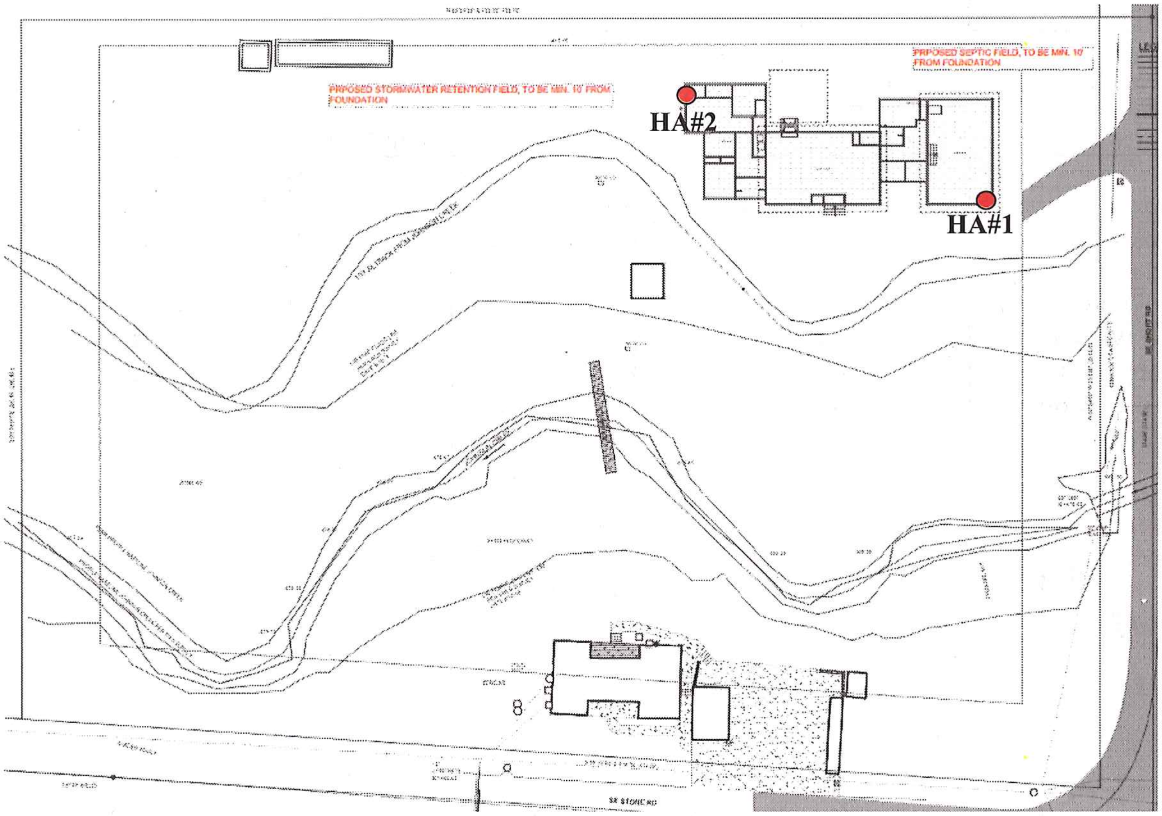


Figure 2: Testing Locations

Lab Results

Project Name: 29619 SE Stone Rd., Gresham, OR

Sample Date 2/5/2019

Moisture

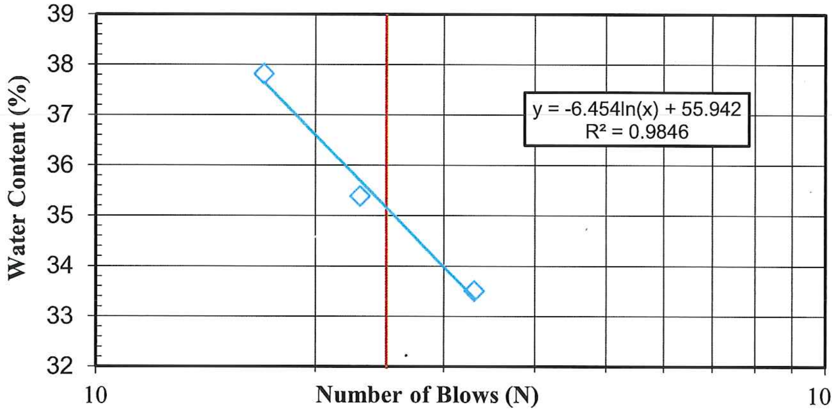
	Sample number	HA#1	HA#2			
1	Date and time in oven	2/5/19 2:00 PM	2/5/19 2:00 PM			
2	Date and time out of ovei	2/7/19 10:15 AM	2/7/19 10:15 AM			
3	Depth (ft)	2	4			
4	Tare No.	2	3			
5	Tare Mass	233	234			
6	Tare plus sample moist	603	1105			
7	Tare plus sample dry	505	872			
8	Mass of water (g)	98	233			
9	Mass of soil (g)	272	638			
10	Water Content (%)	36.0	36.5			

Atterberg Limit Test

Sample Number: HA#1

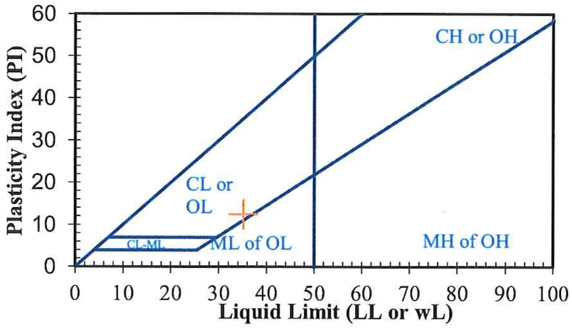
Depth: 2'

		Liquid Limit			Plastic Limit	
		1	2	3	1	2
1	Tare No.	D#1.1	D#1.2	D#1.3	R#1.1	R#1.2
2	Tare Mass (g)	39.12	39.42	40.08	39.33	40.08
3	Tare Plus Wet Soil (g)	80.71	77.94	77.32	49.66	50.93
4	Tare Plus Dry Soil (g)	70.27	67.87	67.1	47.75	48.94
5	Mass of Water (g)	10.44	10.07	10.22	1.91	1.99
6	Mass of Soil (g)	31.15	28.45	27.02	8.42	8.86
7	Water Content (g)	33.52	35.40	37.82	22.68	22.46
8	No. Blows	33	23	17		



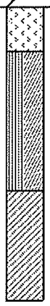
Liquid Limit (%)	**35.2**
Plastic Limit (%)	**22.6**
Plasticity Index (%)	**12.6**
USCS Classification	**CL**

Rapid Soil Solutions INC



HA#1

Surface Elevation: 482
Boring Date: 2/5/19
Boring Location: Gresham, OR
Drilling Method: Hand Auger



Depth	Remarks	Moisture (%)	Dry Density	Blow Counts	Sample Type	Water Table		
0						GWT not encountered	TP	Topsoil with grass roots
							ML-CL	Damp, dark to medium brown, fine grained, stiff, silty-CLAY
1							CL	Moist, medium brown with orange-tan redox. staining, fine grained, stiff, silty-CLAY
2	PI=12.6; LL=35.2	36.0						Boring completed at depth of 2ft
3								
4								
5								
6								
7								

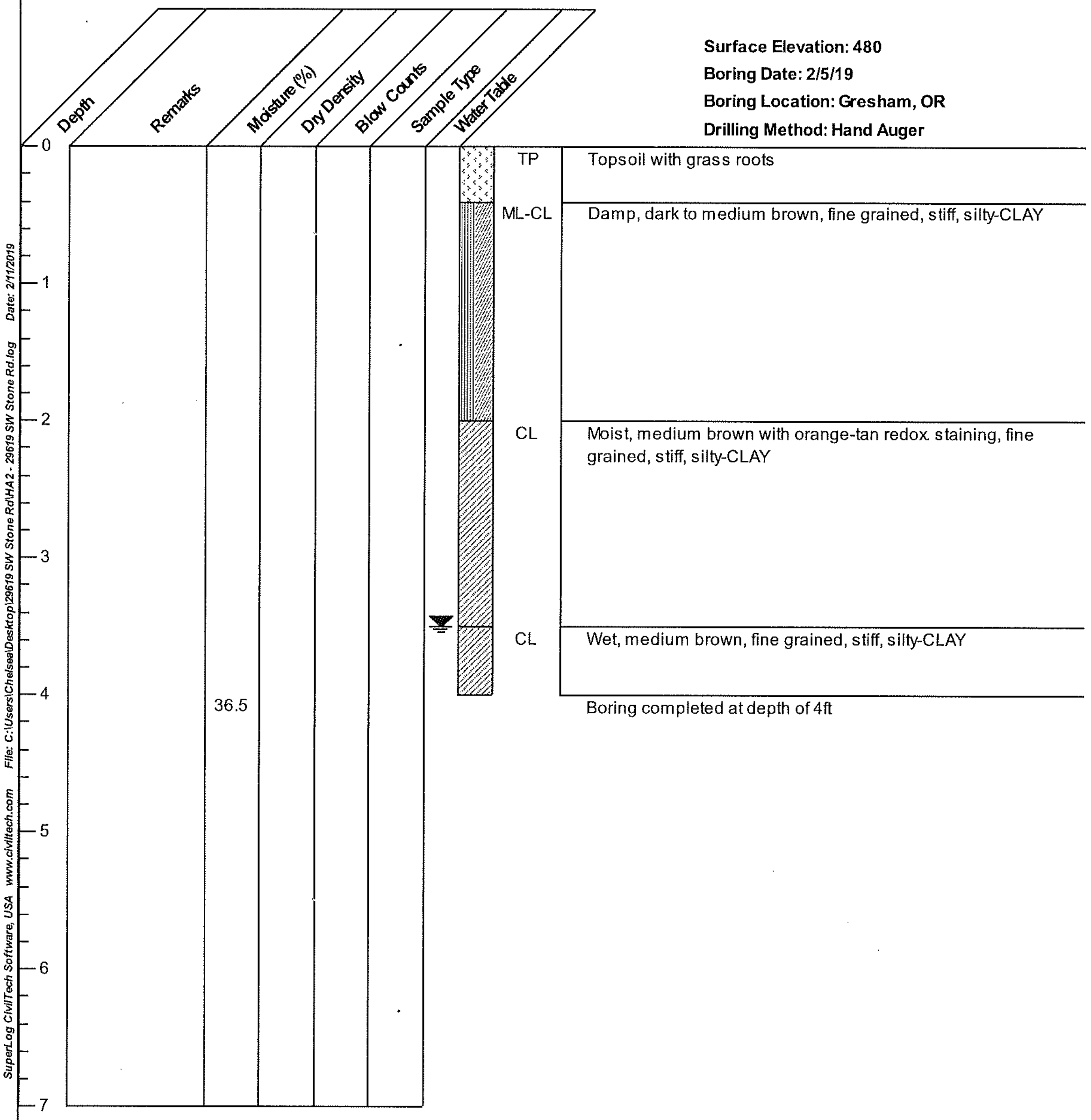
SuperLog CivilTech Software, USA www.civiltech.com File: C:\Users\Chelsea\Desktop\29619 SW Stone Rd\HA1 - 29619 SW Stone Rd.log Date: 2/11/2019

LOG OF BORING

Rapid Soil Solutions	**29619 SE Stone Rd.** **Marin & Daniyela Palamaryuk**	**Plate 1**

HA#2

Surface Elevation: 480
Boring Date: 2/5/19
Boring Location: Gresham, OR
Drilling Method: Hand Auger



TP	Topsoil with grass roots
ML-CL	Damp, dark to medium brown, fine grained, stiff, silty-CLAY
CL	Moist, medium brown with orange-tan redox. staining, fine grained, stiff, silty-CLAY
CL	Wet, medium brown, fine grained, stiff, silty-CLAY

Boring completed at depth of 4ft

SuperLog CivilTech Software, USA www.civiltech.com File: C:\Users\Chelsea\Desktop\29619 SW Stone Rd\HA2 - 29619 SW Stone Rd.log Date: 2/11/2019

LOG OF BORING

Rapid Soil Solutions	29619 SE Stone Rd. Marin & Daniyela Palamaryuk	Plate 1