Geotechnical Investigation

3841/3843 Mapleton Dr.
West Linn, Oregon

Prepared for:
Darren Gustdorf
2 October 2018

REGISTERED PROFESSIONAL
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19244PE
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SUPPORTING DATA

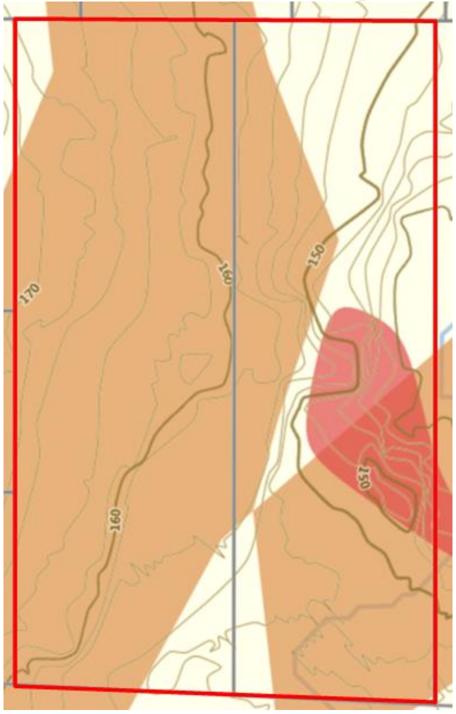
1.0 PROJECT AND SITE DESCRIPTIONS

Rapid Soil Solutions (RSS) has prepared this geotechnical report, as requested, for the proposed new six lot partition to be located in West Linn, Oregon. The subject property is located at 3841 Mapleton Dr. (State ID: 21E24BC-00500) and 3843 Mapleton Dr. (State ID: 21E24BC-00400). The site is located on the north side of Mapleton Drive approximately 600 feet east of Pacific Highway 43. The site is a rectangular shaped lot that spans approximately 230 feet along Mapleton Drive and reaches approximated 375 feet north. RSS understand that the proposed development includes the construction of six single-family residences, with associated roadways/driveways and landscaping improvements. The subject site is tucked between several lots with the street addresses of 3820-3876 Kenthorpe Way (north), 3845 Mapleton Dr. (east) and 3777-3797 Mapleton Dr. (west). The subject site is about 0.18 miles north of Mary S Young State Park, 0.11 miles east of Pacific Highway 43, 0.13 miles south of Cedar Oak Dr., 0.41 miles west of Nixon Ave., and 2.43 miles north of Interstate-205. The site can be found in the northwest quarter of Section 24, Township 2-South and Range 1-East W.M. in Clackamas County. The latitude and longitude of the site are 45.384871 and -122.636855 (45°23'05.5"N, 122°38'12.6"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 1.95-acre (84,942 square foot) subject site is situated in the Robinwood neighborhood of West Linn in incorporated Clackamas County. The site and surrounding tax lots are all zoned R-10, urban low density residential. All of the surrounding tax lots contain single-family residences. Mapleton Drive bounds the subject site to the south, with developed lots surrounding the site on the north, east, and west.



The subject site is currently vacant. The site previously contained a single-family residence within the southwestern corner of the lot (3841 Mapleton Dr.). The residence was demolished in 2017. The gravel driveway leading to the previous residence is still visible on site. Vegetation around the property includes clusters of trees, some low growing bushes and blackberry bushes. The ground surface is covered mostly by blackberry bushes and large trees on the north half of the site and grass with smaller tress on the south half of the site. During the original development of the property, it appears as though some grading work was conducted. An existing culvert runs SW-NE along the southeastern corner of the subject site.

The slopes on site gradually descend eastwards towards the Willamette River. The tax lot extends from about 170 feet in elevation along the western property line of the lot to about 140 feet in elevation along the eastern property line. The nearby Willamette River is at an approximate elevation of 10 feet above mean sea level. The project vicinity has a gentle downward slope towards the east. While on site, RSS observed that the previous building envelope was rather smooth with very gradual slope towards the eastern half of the tax lot. Any undocumented fill or debris from the demolition of the previous residence must be removed from the site prior to construction. Overall the slopes observed on site were consistent with those mapped.

2.2 Regional Geology

Current geologic literature[1,2,3] classifies the slopes underlying the subject site as Pleistocene aged Missoula floods deposits. These deposits were transported into the Portland Basin by dozens of gigantic floods that intermittently inundated the basin at the end of the last ice age. These floods deposits form a thick blanket of unconsolidated materials that covers much of the lowlands in the Portland Basin, and obscures most of the older sedimentary deposits left behind by ancient rivers that meandered across the basin as it formed.

Geologic History

The subject site is situated generally in a central area within the Portland Basin, along the course of the Columbia River. 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 concurrent with its development.

Between about 21,000 to 12,000 years ago, dozens of gigantic floods periodically burst through the ice damn that retained Glacial Lake Missoula, bringing sediment-laden floodwaters into the Portland Basin. These floodwaters emerged from the Gorge at Crown Point Gap at velocities up to 60 miles per hour and plunged down into the broad lowlands. During each flooding event, the wall of water 400-500 feet high descended on the basin, souring many areas down to bedrock and burying others beneath a thick layer of gravels, sand and silt. Dramatic scour features and giant bars can be seen within the Portland 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 http://www.oregongeology.org/sub/ogdc/index.htm

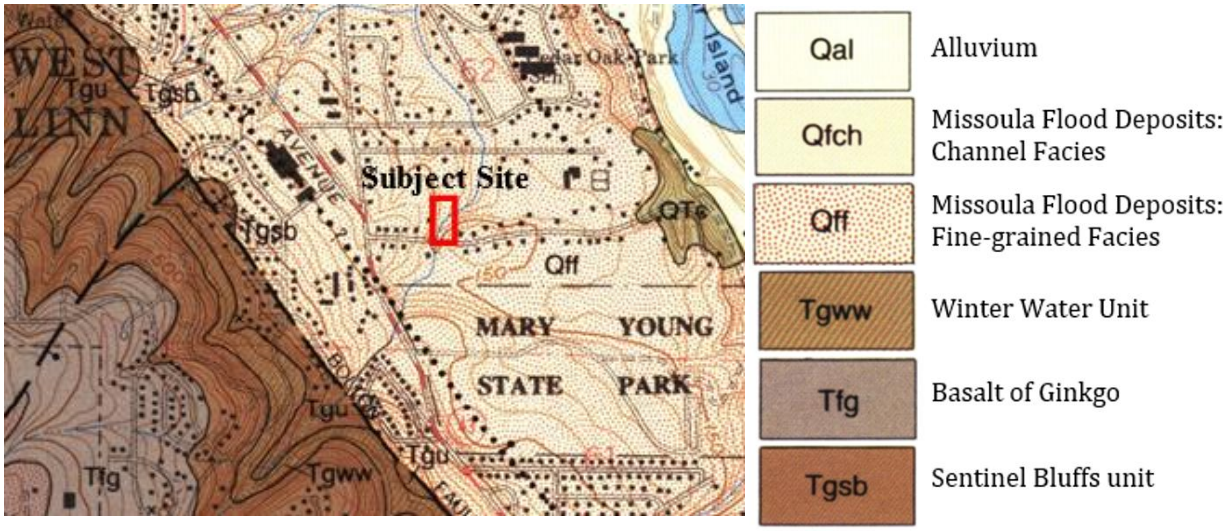
3 Beeson, M.H., Tolan, T.L., and Madin, I.P., 1989, *Geologic map of the Lake Oswego quadrangle, Clackamas, Multnomah, and Washington counties, Oregon*: Oregon Department of Geology and Mineral Industries, Geological Map Series 59, scale 1:24,000.

and demonstrate the great influence the floodwaters had on shaping the Quaternary geomorphology of the region. As the floodwaters hit the hydraulically restrictive Kalama Gap along the Columbia North of Portland, only two thirds of the floodwaters escaped the basin, the rest of the waters ponded in the Portland basin as well as the Tualatin and Willamette basins. The ponded waters dropped a large amount of fine-grained sediments across all of these basins.

Site Geology

Mapping conducted in the local region has divided the unconsolidated Missoula Flood Deposits into categories based on grain size. The subject site is classified as containing surficial deposits that fall within the fine-grained fraction of the Missoula Flood deposits, but the mapped contact with the course grained fraction is merely a third of a mile north of the subject site. The contact between the two grain-size defined facies can be gradational and/or interfingering.

The fine-grained deposits of the Missoula Floods are described as an unconsolidated light-brown to light-gray silt, clay and fine to medium sand. The sediments are deposited in a series of distinct layers, a few inches to a few feet thick, each of which represents a single flood. The finer sediments are predominantly quartz and feldspar and also contain white mica. The coarser sediments can be comprised of Columbia River Basalt fragments. Poorly defined beds of 1- to 3-feet thickness are observed in outcrops, and complex layering has been recorded in boreholes. These deposited have been interested as slack-water sediments settling form the slowing floodwaters. In some areas of this unit, it can include sediments compositionally similar to loess. Soil development commonly introduces significant clay and iron oxides into the upper 6-10 feet of the deposit.



2.3 Field Exploration and Subsurface Conditions

2.3.1 Field Explorations

Four (4) test pits were excavated. The location of the test pits are shown on Figure 3 in Appendix A. An EIT, engineer-in-training, observed the excavation of the test pits and logged the subsurface materials. A registered professional engineer reviewed the results. Logs detailing materials encountered are in the appendix. 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 Aloha silt loam (3-6% slopes). This unit forms on terraces from stratified glaciolacustrine deposits. The Aloha silt loam is classified as somewhat poorly drained and generally has a water table depth of about 18 to 24 inches. The typical profile of Aloha silt loam is silt loam (H1: 0”-8”, H2: 8”-51”, H3: 51”-80”).

2.3.2 Subsurface Conditions

The soil conditions were medium stiff at about 6 inches and then stiff SILT to a depth of 8 feet. Moisture contents ranged from 13.2% to 31.0%.

2.3.3 Groundwater

Groundwater was encountered in TP#1 at 7 feet. Groundwater was not encountered in TP#2 thru TP#4. It is likely that during the winter months, static water levels rise to within a few feet of the ground surface.

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 6-12 inches. This depth may be locally variable and should be confirmed by a geotechnical engineer or their representative at the time of construction. The debris resulting from the demolition of the previous residence and any abandoned utilities must be removed from the site and may not be used as backfill. All tree stumps and roots greater than 1/2 inch in diameter must be removed from any building, slab or pavement subgrade areas. ***Please allow 24hours notice to call for foundation inspections.***

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

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

Footings placed on engineered fill or firm native sub-grade should be designed for an allowable bearing capacity of *2,000 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	2,000psf
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 45.384871 and a Long of -122.636855, soil site class D.

	Short Period	**1 Second**
Maximum Credible Earthquake Spectral Acceleration	$S_S = 0.962$ g	$S_1 = 0.412$ g
Adjusted Spectral Acceleration	$S_{MS} = 1.073$ g	$S_{M1} = 0.655$ g
Design Spectral Response Acceleration Perimeters	$S_{DS} = 0.715$ g	$S_{D1}= 0.436$ g

3.4 Geohazard Review

The Oregon HazVu: Statewide Geohazard Viewer[5] and Metromap[6] were reviewed on 20 September 2018 to investigate mapped geological hazards. This review indicates that the subject site is situated outside the preliminary 100-year floodplain, as mapped by FEMA. The expected earthquake-shaking hazard is classified as 'severe'. The site contains a mapped liquefaction hazard classification of 'high'. The nearest mapped fault classified

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

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

as active by DOGAMI is the NW-SE oriented Lake Oswego Fault passing roughly 0.26-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 0.2 miles southwest of the subject site along the descending slopes of Hidden Springs Road. The landslide hazard at the subject site is classified as 'moderate' landslide susceptibility.

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 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, 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 SILT to the depth explored (8.0 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 house 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

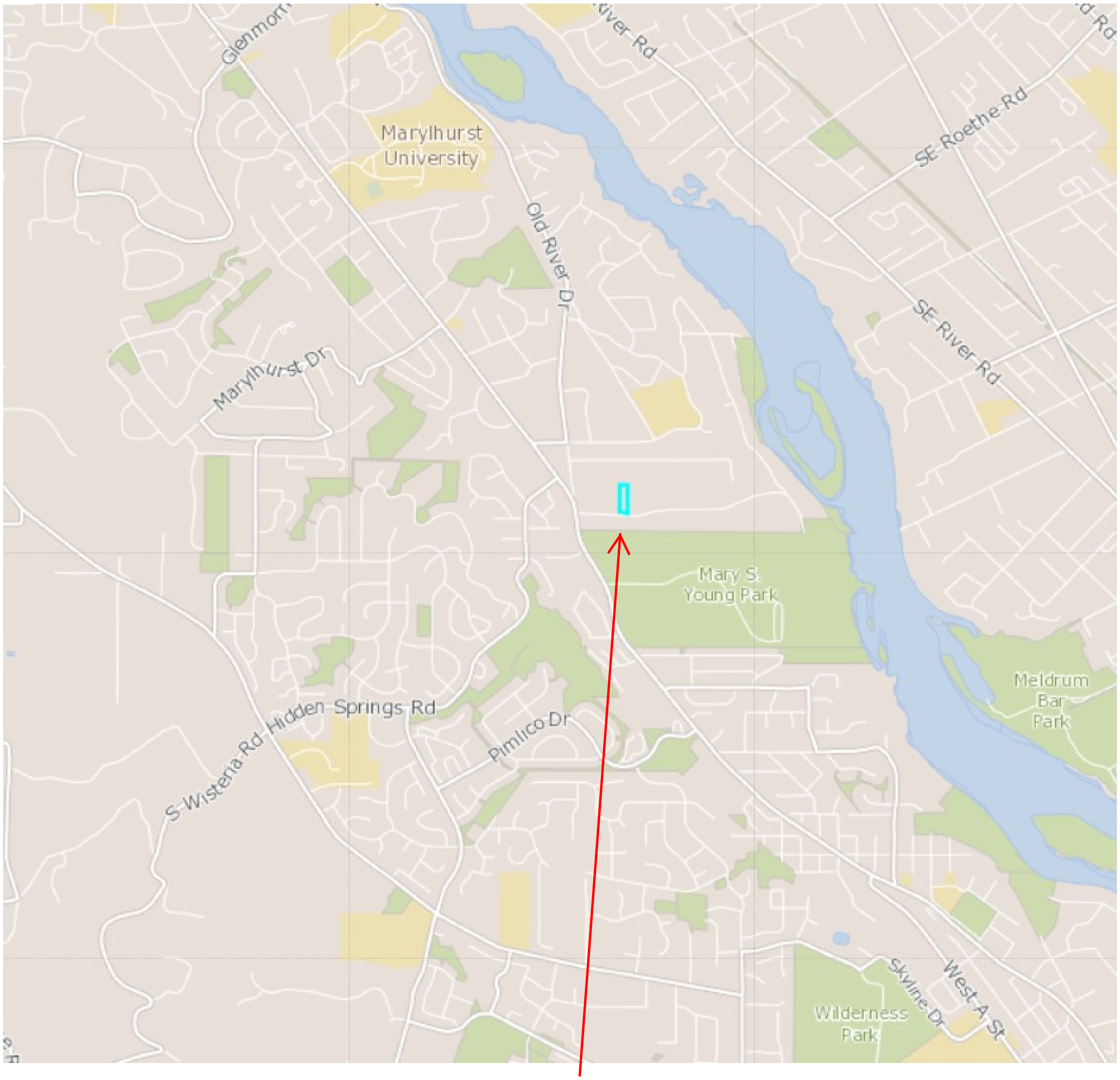


Figure 1 –Site Location

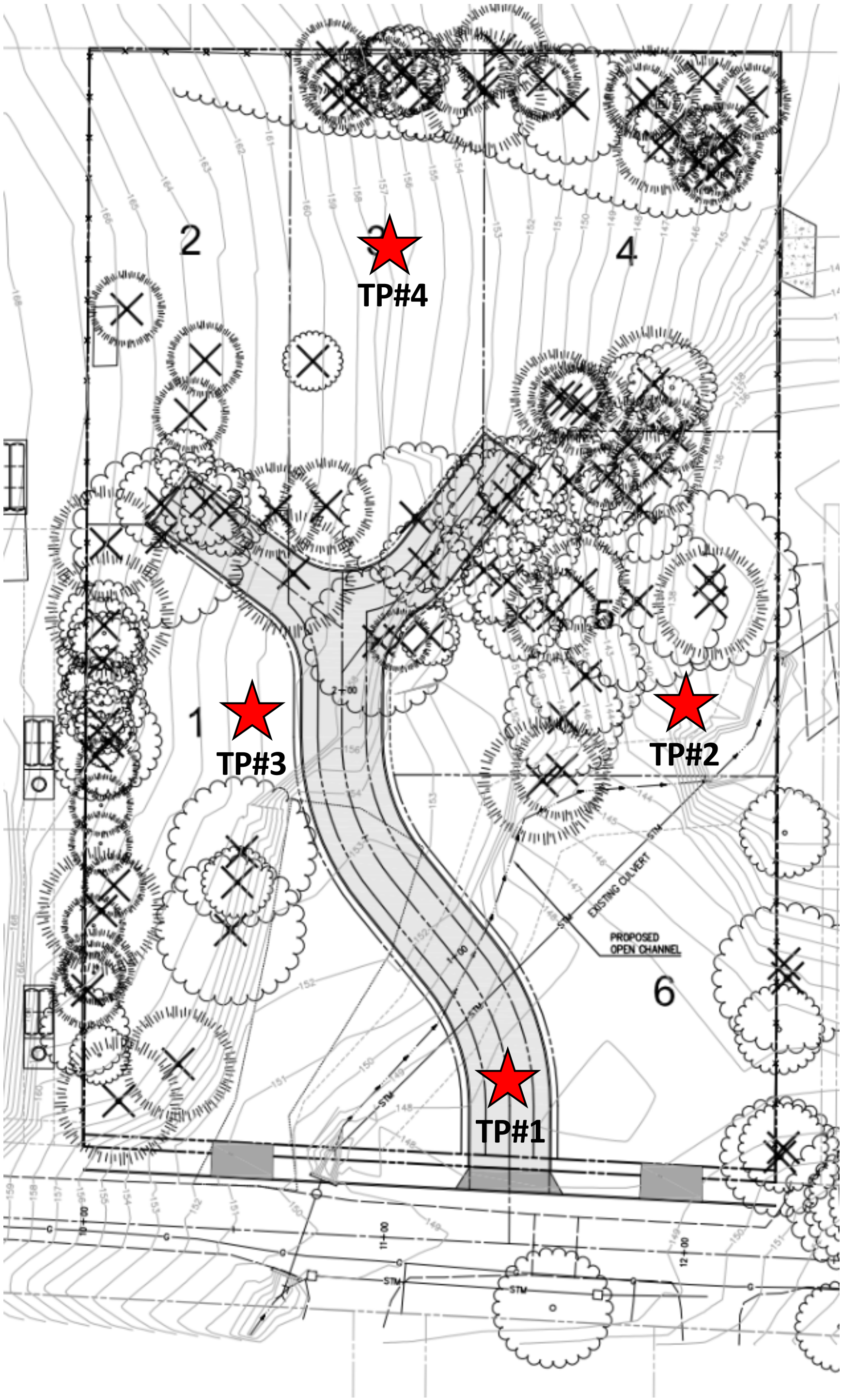
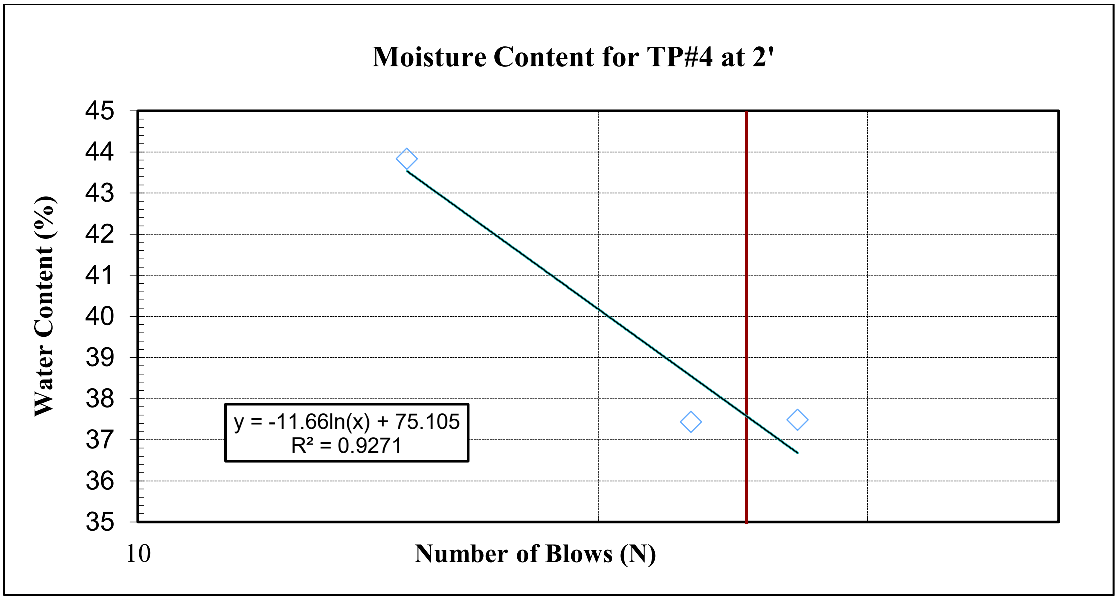
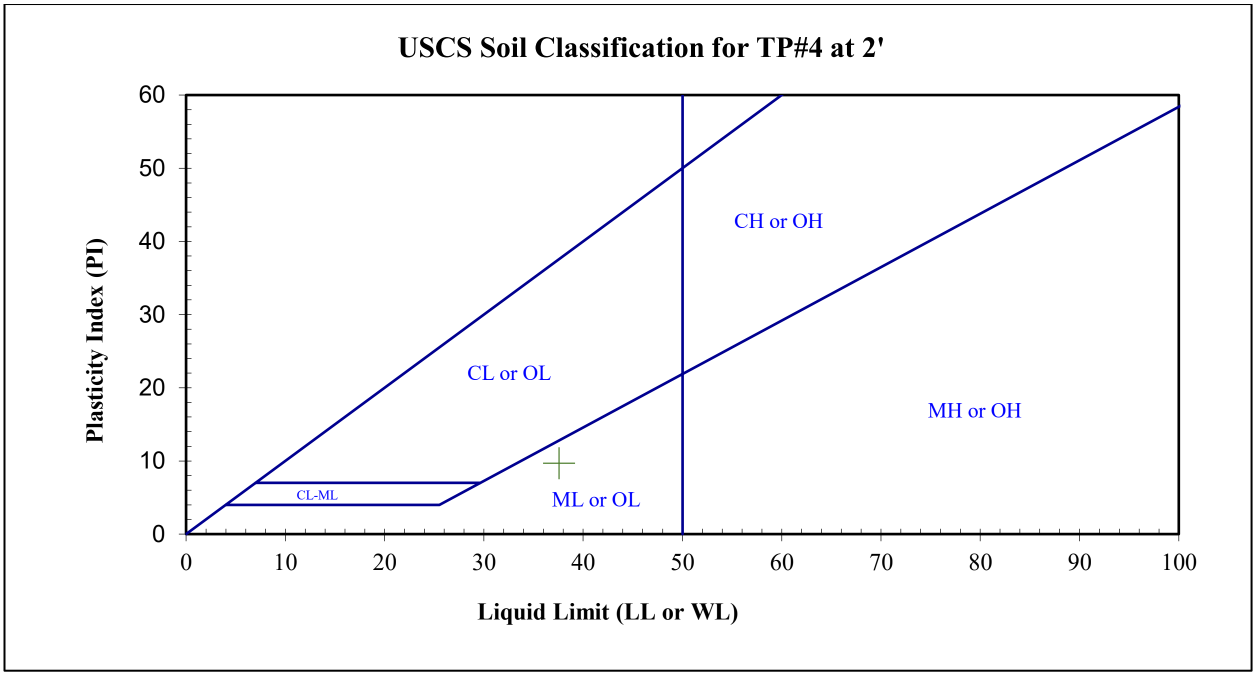


Figure 2 – Testing Locations

Rapid Soil Solution Lab Results							
Project Name:	**Trillium Creek**		**Sample Date:**		**9/19/2018**		
Moisture Content Test							
Sample number	**TP#1**	**TP#1**	**TP#1**	**TP#2**	**TP#3**	**TP#4**	**TP#4**
Date & Time in oven	9/19/18 2:00 PM	9/19/18 2:00 PM	9/19/18 2:00 PM	9/19/18 2:00 PM	9/19/18 2:00 PM	9/19/18 2:00 PM	9/19/18 2:00 PM
Date & Time out of oven	9/20/18 2:00 PM	9/20/18 2:00 PM	9/20/18 2:00 PM	9/20/18 2:00 PM	9/20/18 2:00 PM	9/20/18 2:00 PM	9/20/18 2:00 PM
Depth (ft)	2	4	8	4	6	2	6
Tare No.	6	7	8	9	10	11	12
Tare Mass	232	230	232	230	230	230	231
Tare plus sample moist	1386	1125	1360	1323	932	709	1208
Tare plus sample dry	1251	913	1104	1098	789	618	999
Mass of water (g)	135	212	256	225	143	91	209
Mass of soil (g)	1019	683	872	868	559	388	768
Water Content (%)	**13.2**	**31.0**	**29.4**	**25.9**	**25.6**	**23.5**	**27.2**

Atterburg Limit Test					
Sample Number:	**TP#4**	**Depth (feet):**		**2**	
	Liquid Limit			**Plastic Limit**	
Tare No.	**D#2.1**	**D#2.2**	**D#2.3**	**R#2.1**	**R#2.2**
Tare Mass (g)	39.73	39.58	39.49	39.86	38.97
Tare Plus Wet Soil (g)	91.7	90.21	98.96	50.69	50.18
Tare Plus Dry Soil (g)	77.53	74.78	82.76	48.33	47.73
Mass of Water (g)	14.17	15.43	16.2	2.36	2.45
Mass of Soil (g)	37.8	35.2	43.27	8.47	8.76
Water Content (g)	37.49	43.84	37.44	27.86	27.97
No. Blows	27	15	23	N/A	N/A

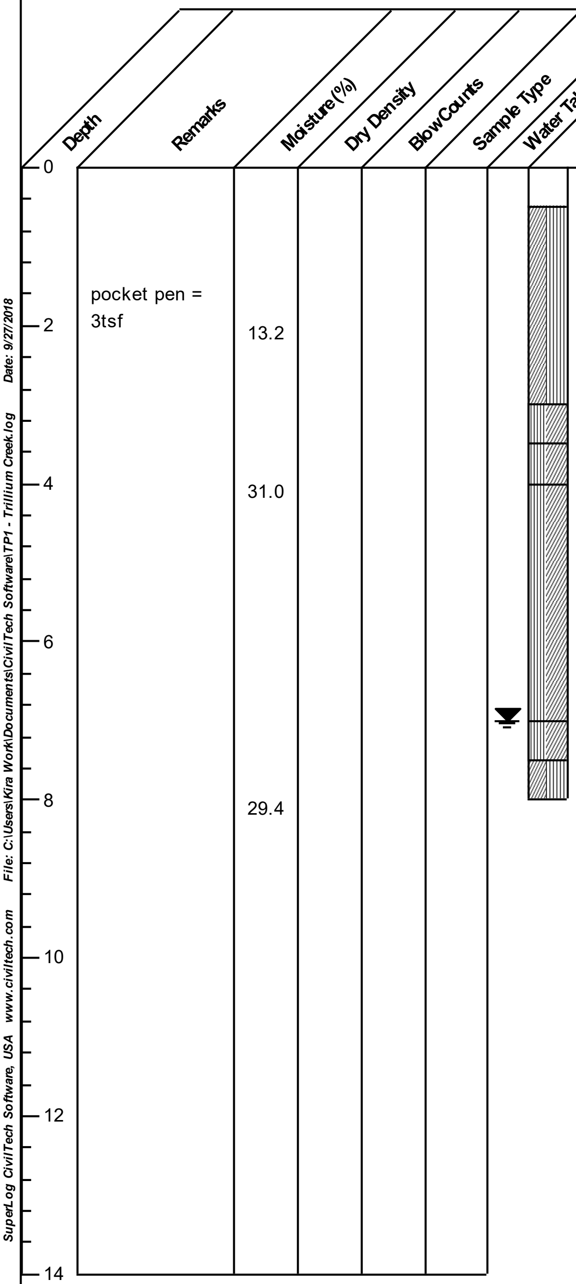




Atterburg Results for TP#4 at 2’	
Liquid Limit (%)	**37.6**
Plastic Limit (%)	**27.9**
Plasticity Index (%)	**9.7**
USCS Classification	**ML; Low Plasticity Silt**

TP#1

Surface Elevation: 152
Boring Date: 9/19/18
Boring Location: West Linn, OR
Drilling Method: Excavator



TP	Top Soil with grass roots
CL-ML	Dry, Tan to medium brown, fine grained, very stiff, clayey-SILT, trace gravels
ML-CL	Damp/moist, medium brown, medium stiff, silty CLAY, debris and organics present
ML-CL	Damp/moist, blue/grey, fine grained, medium stiff, silty CLAY
ML-CL	Blue/grey, damp/moist, fine grained, medium stiff, silty CLAY
ML-CL	Water table
CL-ML	Dark brown, fine grained, medium stiff, clayey-SILT, sand and gravels present, trace cobbles
	Test pit completed at depth of 8 feet

SuperLog CivilTech Software, USA www.civiltech.com File: C:\Users\Kira Work\Documents\CivilTech Software\TP1 - Trillium Creek.log Date: 9/27/2018

LOG OF BORING

CIVILTECH SOFTWARE

Trillium Creek
Darren Gustdorf

Plate 1

TP #2

Surface Elevation: 144
Boring Date: 9/19/2018
Boring Location: West Linn
Drilling Method: Excavator

Depth	Remarks	Moisture(%)	Dry Density	BlowCounts	Sample Type	Water Table		
0					GWT not encountered		TP	Topsoil with grass roots
							ML	Dry, light brown/tan, fine coarse grain, sandy-SILT, trace gravels/cobbles
1								
							ML	Dry, medium brown, redox features, fine coarse grain sandy-SILT, trace gravels/cobbles
	Pocket pen = 4.5 tsf							
2								
3								
							ML	Damp/moist, medium-dark brown, sandy-SILT, pockets of grey mottling, some carbonized organics present
	Pocket pen = 4.5 tsf							
4		25.9					ML	Test pit completed at depth of 4 feet
5								
6								
7								

LOG OF BORING

CIVILTECH SOFTWARE	**Trillium Creek** **Darren Gustdorf**	**Plate 2**

TP #3

Surface Elevation: 162
Boring Date: 9/19/18
Boring Location: West Linn
Drilling Method: Excavator

Depth	Remarks	Moisture(%)	Dry Density	BlowCounts	Sample Type	Water Table		
0						GWT not encountered	TP	Topsoil with tree roots
							ML	Dry, grey/brown, fine coarse grain SILT, with roots present
1								
							ML	Dry, tan/medium brown, fine coarse grain clayey SILT, debris and organics present
2	Pocket pen = 3.5 tsf							
							ML	Damp/moist, medium brown/tan, clayey-SILT
3								
	Pocekt pen = 3 tsf							
4							ML	Damp/moist, medium brown, fine grain sandy SILT
5								
6		25.6						Test pit completed at depth of 6 feet
7								

LOG OF BORING

CIVILTECH SOFTWARE	**Trillium Creek** **Darren Gustdorf**	**Plate 3**

TP #4

Surface Elevation: 154
Boring Date: 9/19/18
Boring Location: West Linn
Drilling Method: Excavator

Depth	Remarks	Moisture(%)	Dry Density	BlowCounts	Sample Type	Water Table		
0						GWT not encountered	TP	Topsoil
1							ML	Dry, grey-brown, redox features, fine grained SILT
							ML	Dry, medium-brown, fine grained sandy SILT
2	Pocket Pen = 3.5 tsf, Liquid Limit = 37.6, Plastic Limit = 27.9, Plasticity Index = 9.7	23.5					ML	Dry, medium-brown, fine grained sandy SILT
							ML	Damp/moist, dense, medium-brown, fine grained sandy SILT
3								
4								
5								
6		27.2						Test pit completed at depth of 6 feet
7								

SuperLog CivilTech Software, USA www.civiltech.com File: C:\Users\Kira Work\Documents\CivilTech Software\TP1 - Trillium Creek.log Date: 9/27/2018

LOG OF BORING

CIVILTECH SOFTWARE	**Trillium Creek** **Darren Gustdorf**	**Plate 4**