GEOTECHNICAL REPORT

RhinoOne GEOTECHNICAL

December 1, 2017

Exeltech Consulting Inc.
Karl N. Kirker, PE, SE
Senior Project Manager
921 SW Washington Street, Suite 464
Portland, Oregon 97205

RE: Geotechnical Engineering Technical Memorandum
SW Upper Hall Street Landslide Mitigation Project
1518 SW Upper Hall Street
Portland, Oregon
RhinoOne Project Number EXT-2017-001

Dear Mr. Kirker,

RhinoOne Geotechnical (ROG) is pleased to present this Geotechnical Technical Memorandum (GTM) for the Portland Bureau of Transportation (PBOT) SW Upper Hall Street Landslide Mitigation Project (T00678) in Portland, Oregon (Figure 1 – Site Location Map). This work is being completed as part of our contract dated August 2017.

The scope of work for this project is to develop remedial design for a landslide along the south shoulder of SW Upper Hall Street immediately east of house number 1518 SW Upper Hall Street in Portland, Oregon. Several remedial options such soil nail wall, sheet pile wall, Mechanically Stabilize Wall (MSE), and soldier pile wall were considered.

The purpose of this geotechnical study is to complete a geologic reconnaissance, explore subsurface conditions at the proposed project location, develop a subsurface profile, and provide geotechnical design criteria and construction recommendations for the selected retaining wall system.

Geologic Reconnaissance

Geologic field reconnaissance of the project site was completed by Peter Hughes, CEG, on October 13, 2017. Southwest Upper Hall slopes downward to the east. The south side of the roadway is a cut bank which measures about 14 feet above the road surface. The cut bank is generally covered in grass, ivy, and blackberry vines. The stairway to a house is adjacent to the west side of the landslide. Multiple deciduous trees line the top of the cut slope. Residential buildings line SW Upper Hall Street on the north side of the road.

A small landslide feature occurred on the south side of SW Upper Hall Street. The shallow slide feature occurred within the cut slope (south side) of SW Upper Hall Street. The slide measures roughly 25 feet wide by 14 feet high. One home is adjacent to the slide on the south side of SW Upper Hall Street. Multiple homes are located on the north side of SW Upper Hall Street. The house adjacent to the landslide is approximately 20 feet from the west lateral margin of the slope failure. The landslide debris was reportedly deposited on the road.

Field Exploration and Subsurface Conditions

Site geology and subsurface conditions at the project location were evaluated based on a review of geologic and hazard mapping reports, literature review, site reconnaissance, previous subsurface explorations, and explorations conducted for this study.

Geologic Mapping

The site is located in the Willamette Valley on the western flank of the Tualatin Mountains. The Tualatin Mountains form the physiographic boundary between the Portland Basin to the east and the Tualatin Basin to the west. These basins are part of the larger Puget Sound-Willamette Valley physiographic province, a tectonically active lowland situated between the Coast Range to the west and the Cascade Mountains to the east[1].

According to published geologic mapping, the site is underlain by basalt bedrock belonging to older Boring Lava flows[2]. The Boring Lava basalts are younger in age than the basement bedrock belonging to the Columbia River Basalt Group (CRBG). The CRBG consists of thick flows of basalt which have been folded and faulted from the compressional tectonics of the region.

Surficial soils in the vicinity of the landslide consist of sand, gravel, and silt. These soils are interpreted to be colluvium derived from the weathering of the Boring Lava basalts.

The elevation at the site ranges from approximately ±300 feet on SW Upper Hall Street to ±320 feet in the residential property above the head scarp of the landslide. The surface topography at the site generally slopes towards the northeast. The steep slope on which the failure occurred appears to be a road cut made for the construction of SW Upper Hall Street.

Field Explorations

The subsurface exploration program for this project consisted of drilling one (1) boring. The upper 3.5 feet of the boring were excavated using a hand operated jack hammer and vacuum truck on October 24, 2017 by Cascade Drilling, Inc. of Clackamas, Oregon. The remainder of the boring was drilled using a truck-mounted drill rig operated by Cascade Drilling, Inc. on October 25, 2017. The boring was drilled at the approximate location shown on the Site Exploration Plan (Figure 2). The drilling was performed using rock coring drilling techniques. Rock was encountered at a depth of 4.5 feet below ground surface (BGS). The boring was drilled to a depth of 25.33 feet with 21.8 feet of rock coring. A Standard Penetration Test (SPT) soil sample was obtained using a 140-pound Automatic Hammer at a depth of 3.5 feet BGS. Uncorrected blow counts from the SPT sampling are reported on the boring logs. Corrected blow counts [$(N_1)_{60}$] were used for our analysis unless otherwise noted.

The subsurface materials encountered were logged and field classified in general accordance with the Manual-Visual Classification Method (ASTM D 2488). The SPT samples were collected at desired depths and packaged in moisture-tight bags. The soil samples were reviewed in the laboratory in order to supplement field classifications. Interpreted boring log is attached.

[1] Orr, E.L. and Orr, W.N. (1999). *Geology of Oregon*. Kendall/Hunt Publishing, Iowa. Page 254.
[2] Ma, L., Madin, I.P., Duplantis, S., and Williams, K.J. (2012). ***Lidar-based Surficial Geologic Map and Database of the Greater Portland Area***, Clackamas, Columbia, Marion, Multnomah, Washington, and Yamhill Counties, Oregon, and Clarck County, Washington. Oregon Department of Geology and Mineral Industries. Open-file Report O-12-02.

Laboratory Testing

Laboratory testing was not completed as suitable samples were not recovered from the borings for laboratory testing.

Subsurface Conditions

One boring was drilled at the site. The approximate boring location is shown on Figure 2 of Appendix A. The boring log is attached at the end of this memorandum. Rock core photographs are also attached. A summary of the boring drilled is provided in Table 1 below.

Table 1 Summary of Boring

Boring Number	Approximate Ground Surface Elevation (feet)	Depth of Boring (feet)	Upper Pavement and Soils (feet)	Basalt Rock (feet)
B-1	299	25.3	4.5	4.5 – 25.3

One boring was drilled in the parking lane on the south side of the right of way for SW Upper Hall Street near the location of the landslide. The pavement section consisted of 4 inches of Asphalt Cement Concrete (ACC). Very dense sand, gravel, and cobbles with silt was encountered below the asphalt and to a depth of 4.5 feet BGS. It is likely, the upper few feet of this gravel-soil matrix is man-made fill while the lower few feet are weathered rock. However, the transition from fill to native material was difficult to determine. Medium hard to hard (R3-R4) basalt bedrock underlies the fine-grained alluvium. The basalt is slightly weathered and is moderately to intensely fractured. During drilling, circulation was lost at approximately 4 feet below ground surface. The basalt was encountered to the maximum depth of exploration (25.3-feet BGS). The rock core properties are summarized in the table below.

Table 2 Summary of Rock Coring – SW Upper Hall Street Boring B-1

Core Run	Recovery (%)	Hardness	RQD (%)	Rock Mass Designation (RMR)
1 HQ (4.5' – 11.5')	79	R3-R4	30	50 – 60 (Fair Rock)
1 HQ (11.5' – 13')	100	R3-R4	67	50 – 60 (Fair Rock)
1 HQ (13' – 16')	100	R3-R4	22	50 – 60 (Fair Rock)
1 HQ (16' – 21')	83	R3-R4	60	50 – 60 (Fair Rock)
1 HQ (21' – 25.3')	100	R3-R4	27	50 – 60 (Fair Rock)

Groundwater

Groundwater was not encountered at the time of drilling. Information provided by the US Geological Survey (USGS) *Estimated Depth to Groundwater Study of the Portland Metro Area*[3], along with a review of existing well logs in the area, indicate the groundwater table is about 165 feet BGS.

[3] US Geological Survey (USGS). *Estimated Depth to Ground Water in the Portland, Oregon Area*. Accessed from website http://or.water.usgs.gov/projs_dir/puz/ on October 26, 2017.

Discussion

A small landslide feature occurred on the south side of SW Upper Hall Street. The shallow slide feature occurred within the cut slope (south side) of SW Upper Hall Street. The slide measures roughly 25 feet wide by 14 feet high. One home is adjacent to the slide on the south side of SW Upper Hall Street. Multiple homes are located on the north side of SW Upper Hall Street. The house adjacent to the landslide is approximately 20 feet from the west lateral margin of the slope failure. The landslide debris was reportedly deposited on the road. The area within the landslide and immediate vicinity was surveyed by PBOT. ExcelTech has developed sections at every 15 feet along the proposed wall alignment.

Based on our observations, a review of the survey data, and discussion with you, we recommend the landslide mitigation measures extend from approximate Station 00+00 to Station 00+45 for a 45 feet long wall. The actual location will be determined in a meeting on the site at a later date. The bottom of the wall should be measured from the base of the slope (road level). Assuming the wall will be backfilled at a slope of 3H:1V to blend with the existing slopes of the top, the maximum wall height is 10 to 13 feet.

Several wall options were considered. These options are briefly discussed in the paragraphs below.

MSE Wall/Cast-in-Place Concrete Wall

MSE or cast-in-place concrete walls can be used for landslide mitigation. Large excavations will be required to construct such walls to accommodate the wall footings and the grids. Temporary cuts required to construct such a wall at a slope of 1H:1V are not feasible due to limited ROW (approximately 10 feet from edge of the road). Temporary shoring can be constructed to build a vertical excavation. However temporary or permanent easements may be required for these walls which are not desirable. Therefore, these options are not considered any further.

Soil Nail Wall/Anchored Soldier Pile Wall/Anchored Mesh Walls

These walls will require permanent easements for the installation of nails/anchors. We understand PBOT does not desire to have permanent easements. Therefore, these walls are not considered any further.

Sheet Pile Wall

A sheet pile wall can be constructed for landslide mitigation. Installation of the sheets is not feasible due to the shallow rock formation. Therefore, these walls are not considered any further.

Cantilever Soldier Pile Wall

Due to the limited height of walls (13 feet or less), ease of installation, and the limit of the ROW, the soldier pile wall is a feasible option. The following paragraphs discuss design parameters and construction requirements for a soldier-pile wall.

Cantilever Soldier Pile Retaining Wall

A soldier pile wall can be constructed as the new retaining wall. Wood lagging can be used for temporary support as the cut is being made for the portions of wall under existing slope line i.e. near the base of the slope. We understand the permanent facing will either be cast-in-place concrete or precast concrete panels. This method has the advantage of top-down construction with the slope supported during the excavation of the cut. In addition, this method does not require permanent easement for the height of the cuts proposed can be handled using cantilever wall sections. This method will be more expensive than the soil nail wall but has a lower risk.

Based on our discussions with you, the soldier pile wall is the preferred method for the construction of this wall. Design recommendations for the soldier pile wall are provided in the section below.

Soldier Pile Wall Design Recommendations

The portions between the existing slope and the pile wall will be backfilled. A review of the sections indicates the backfill behind the wall will have slopes of 3H:1V or flatter. Our recommended pressure diagram for the design of a cantilever soldier pile wall is provided on Figure 3. The wall should be designed for an active equivalent fluid unit weight (EFW) of 47.5 pounds per cubic foot (pcf) above the base of cut. The 1,000 year adjusted PGA at this site is 0.33g. Using this value and the Mononobe-Okabe method, we calculated an additional seismic pressure on the order of $14H^2$ applied at 0.6H from the base of the wall for the design of the wall.

For passive pressure, ignore the top 2 feet of material. Use a passive EFW of 400 pcf from 2 to 5 feet and 600 pcf at depths greater than 5 feet. The EFW should be applied to 2 times the diameter from 2 to 5 feet and 3 times the diameter below that. Since these piles are upslope, we do not anticipate any traffic surcharge. However, we recommend a fence be placed on top of this wall to arrest any loose material which may fall downslope and to prevent vandalism. The soldier piles should be embedded a minimum of 15 feet below the road base. We also recommend a concrete ditch be constructed at the top of this wall to collect and route surface water from upslope.

Seismic Design Recommendations

The seismic design criteria for retaining walls for this project are governed by Oregon's Department of Transportation (ODOT) Geotechnical Design Manual (GDM) dated December 2016. We have assumed the new retaining wall will be classified as a "Highway Retaining Wall" for the purpose of seismic design. The ODOT GDM (2016) states "Highway Retaining Walls should be designed for 1000-year return period ground motions." Additionally, under this level of shaking, the Highway Retaining Wall must be able to withstand seismic forces and displacements (including displacements due to liquefaction) without failure of any part of the Highway Retaining Wall.

A soil profile type "C" can be used for the seismic design of the retaining wall based on the subsurface conditions encountered. The 1000-year event equates to a seven-percent (7%) probability of exceedance in a 75 year return period. Recommended seismic design parameters are presented in the table below.

Table 3 1000-year Earthquake Return Interval Seismic Design Parameters

	Short Period	1 Second
Maximum Credible Earthquake Spectral Acceleration	S_s = 0.5949 g	S_1 = 0.2170g
Mapped Peak Ground Acceleration	PGA = 0.2715 g	
Site Class	C	
Site Coefficients	F_{PGA} = 1.200	
	F_a = 1.2621	F_v = 1.500
Design Spectral Response Acceleration Parameters	S_{DS} = 0.7507 g	S_{D1} = 0.3255 g
Design Spectral Peak Ground Acceleration	A_S = 0.3258 g	

As discussed above these accelerations will add an additional seismic pressure on the order of $14H^2$ applied at 0.6H from the base of the wall for the design of the wall.

Global Slope Stability Analysis

We completed a global stability analysis for the slopes with the soldier pile wall. The slope stability model was developed based on our field exploration, geologic reconnaissance and survey cross-sections. The soils were modelled as shown in the table below. The pile wall was modeled with infinite strength. We also assumed groundwater was at a depth of 5 feet below the existing ground surface. The soil strength properties for stability analysis are tabulated below.

Table 4 Recommended Soil Parameters

Soil	Average Depth at Pile Location (feet)	Total Unit Weight (pcf)	Angle of Internal Friction (degree)	Undrained Cohesion (psf)
Poorly Graded Gravel	0 – 5	120	34	0
Weathered Bedrock	>5	120	38	1,000

The computer program SLIDE Version 6.00 was used for our slope stability analysis. Analyses were performed for both the static and seismic loading conditions. Seismic slope stability was evaluated using the pseudo-static method. The horizontal seismic coefficient used in the seismic analysis was equal to 2/3rd of the maximum site-adjusted peak ground acceleration for the design seismic event (1,000 year return period), which is 0.33g for the site.

Using the slope profile and properties discussed above, the stabilized slope with cantilever pile wall is calculated to have a factor of safety (FS) of greater than 1.5 under static conditions and 1.1 under seismic conditions. These factors of safety assume that the piles extend a minimum of 15 feet below the base of the cut.

Limitations

This technical memorandum has been prepared exclusively for the Project design team for the above-mentioned project, in accordance with generally accepted geotechnical engineering practice. No other warranty, express or implied, is made.

The geotechnical strength parameters described herein are based on subsurface conditions available from field explorations as documented in this report. Simplified assumptions are made for data interpretation. These data indicate subsurface conditions only at specific locations and times, and only to the depths penetrated. The data do not necessarily reflect variations which may exist between such locations. If variations in subsurface condition from those described are noted during supplemental exploration or construction, the recommendations in this technical memorandum must be reevaluated.

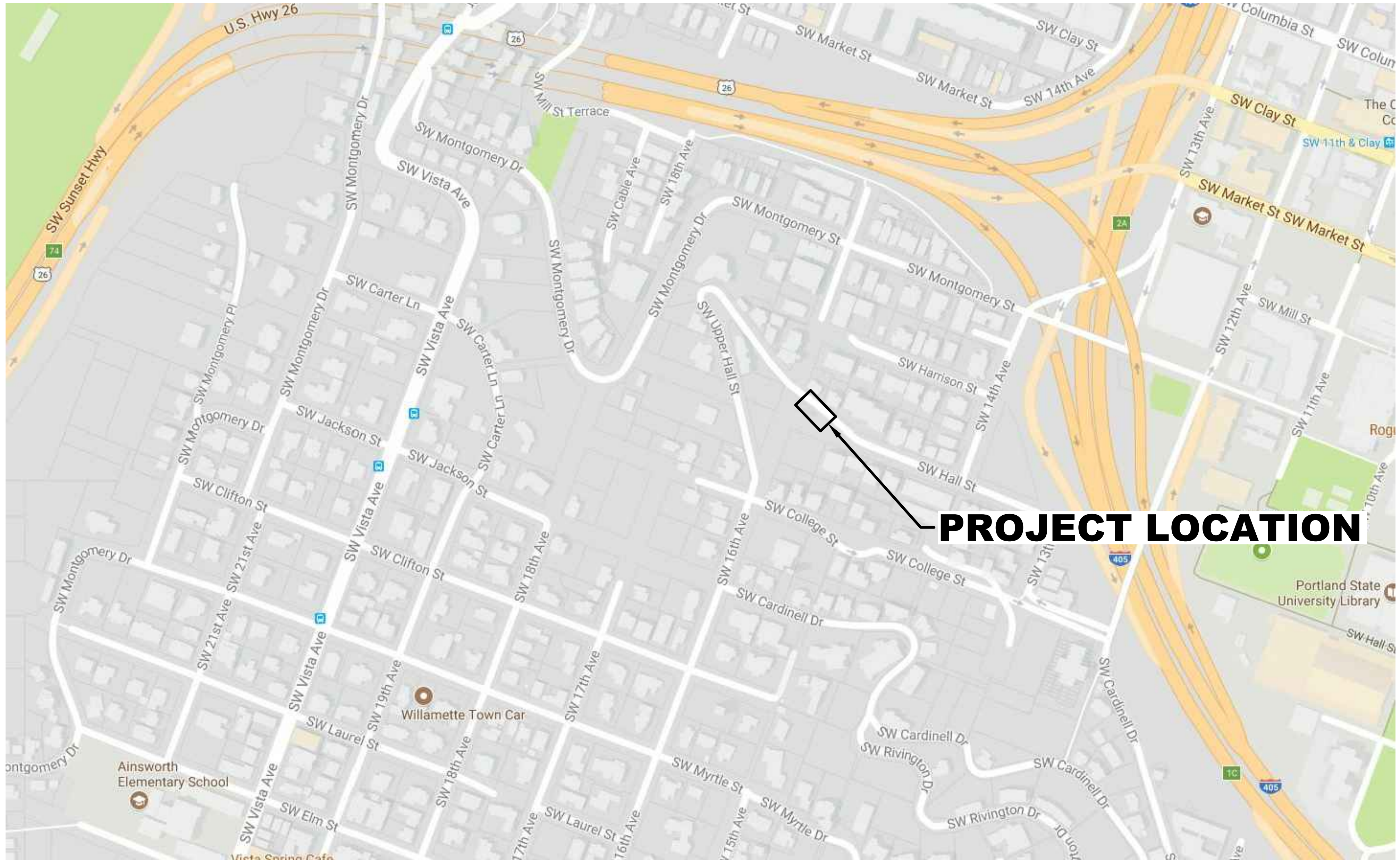
Rhino One hopes this submittal meets your requirements at this time. Please call us if you need further information.

Sincerely,

Christina Hemberry, PE
Staff Geotechnical Engineer

Rajiv Ali, PE, GE (OR)
Geotechnical Engineer

Attachments
Figure 1: Site Location Map
Figure 2: Site Exploration Plan and Foundation Data
Figure 3: Recommended Pressure Diagram for Wall Design
Boring Log: B-1
Rock Core Photograph



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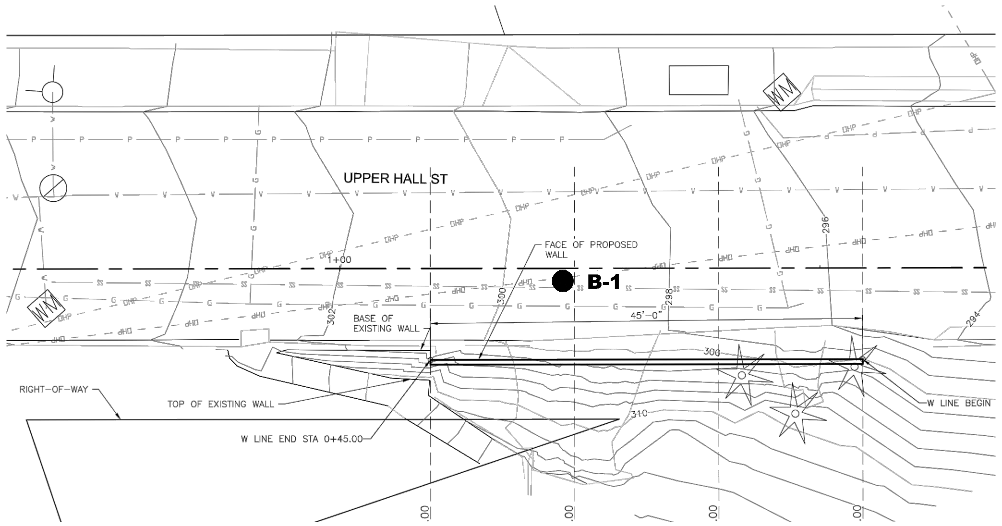
UPPER HALL STREET LANDSLIDE MITIGATION

1518 SW UPPER HALL STREET
PORTLAND, OREGON PBOT PROJECT T00678

FIGURE 1 - SITE LOCATION MAP

PROJECT
EXT-2017-001

DATE
NOV 2017



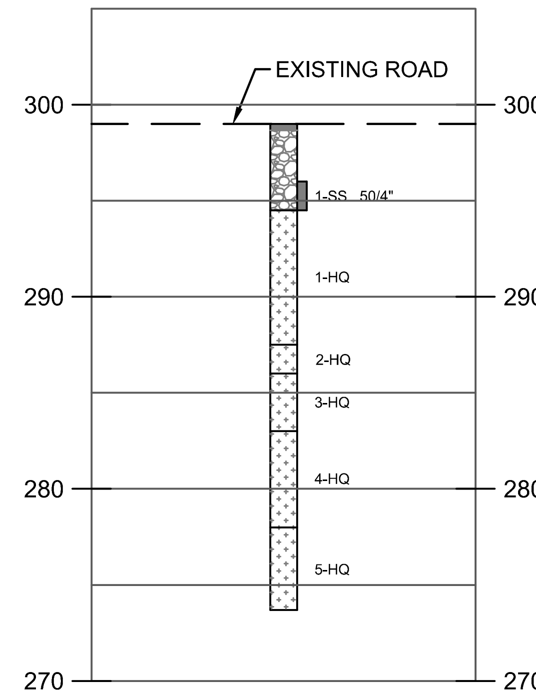
SITE PLAN

SCALE: 1" = 20'

BORING B-1

10/25/2017

EL. ±299 FT AMSL



BORING PROFILE AT ROAD

Hor. Scale: 1"= 10'

Vert. Scale: 1"= 10'

GENERAL NOTE

1. GEOTECHNICAL DATA SHOWN ON THIS DRAWING ARE A CONSOLIDATION OF INFORMATION AND/OR REVISION IN TERMINOLOGY FROM THE SOIL BORING LOGS. THE SOIL BORING LOGS USED IN COMPILING THIS DRAWING ARE AVAILABLE UPON REQUEST. CONTRACTOR SHALL REFER TO GEOTECHNICAL REPORTS AND SOIL BORING LOGS AND INFORMATION CONTAINED THEREIN.

LEGEND

1-HQ CORE SAMPLE

RQD ROCK QUALITY DESIGNATION

q = UNCONFINED COMPRESSIVE STRENGTH

EXPLANATION OF MATERIALS

ASPHALT CEMENT CONCRETE (AC)

POORLY GRADED GRAVEL

BASALT; MEDIUM HARD TO HARD (R3-R4)

STANDARD PENETRATION TEST - BLOW COUNT (N) USING 140# HAMMER FALLING 30"

DAMES AND MOORE SAMPLER - BLOW COUNT (N) USING 140# HAMMER FALLING 30"

BORING B-1

CORE RUN	RECOVER (%)	HARDNESS	RQD (%)	qu (PSI)
1 - HQ	79	R3-R4	30	
2 - HQ	100	R3-R4	67	
3 - HQ	100	R3-R4	22	
4 - HQ	83	R3-R4	60	
5 - HQ	100	R3-R4	27	

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UPPER HALL STREET LANDSLIDE MITIGATION

1518 SW UPPER HALL STREET

PORTLAND, OREGON PBOT PROJECT T00678

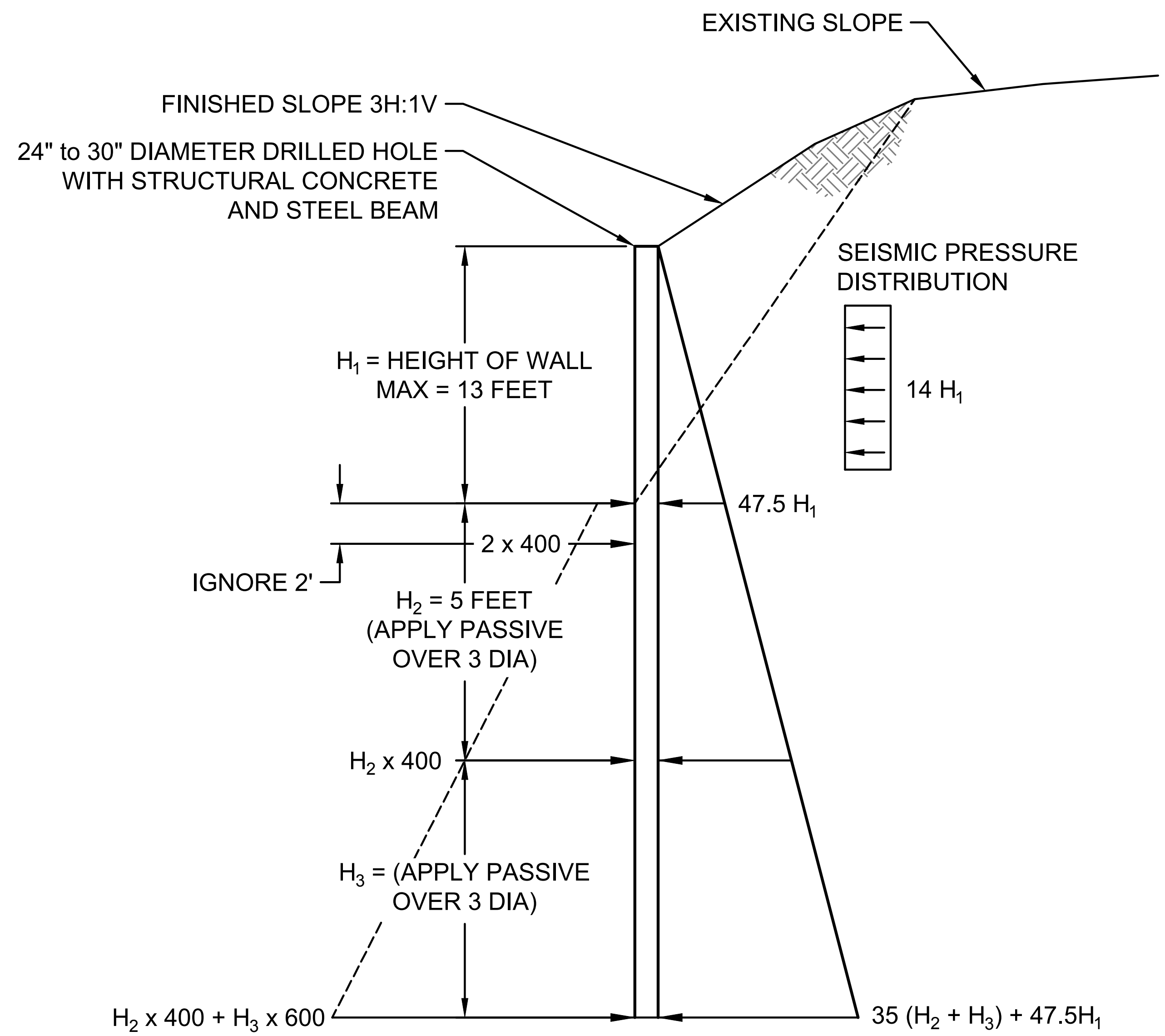
FIGURE 2 - SITE EXPLORATION PLAN AND FOUNDATION DATA SHEET

PROJECT

EXT-2017-001

DATE

NOV 2017



PILE SUPPORTED WALL

NOTES:

1. MINIMUM EMBEDMENT 15 FEET BELOW STREET. (10' EMBEDMENT IN ROCK)
2. USE 24" to 30" DIAMETER DRILLED HOLES WITH STRUCTURAL CONCRETE.
3. NO UNSUPPORTED CUTS GREATER THAN 3 FEET.

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UPPER HALL STREET LANDSLIDE MITIGATION

1518 SW UPPER HALL STREET

PORTLAND, OREGON PBOT PROJECT T00678

FIGURE 3 - PRESSURE DIAGRAM FOR RETAINING WALL

PROJECT	DATE
EXT-2017-001	NOV 2017

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SW Upper Hall Street Landslide Mitigatio
1518 SW Upper Hall Street, Portland, Ore
PBOT Project T00678

Boring Number: B-1

Project: SW Upper Hall Street Landslide Mitigatio	Driller: Cascade Drilling Inc.
Project Number: EXT-2017-001	Date: October 25, 2017
Drilling Method: HQ Rock Core	Elevation: Approx. 299 feet AMSL
Diameter: 3 inch; Water Level: Not Encountered	Logged by: Christina H

Sample No.	Sample Type	Recovery (%)	RQD (%)	Blow Count per 6 inches	Blows/Foot (N)	Water Table
SS1		60		21-50 for 4"	50	GWT not encountered
C1		79	30			
C2		100	67			
C3		100	22			
C4		83	60			
C5		100	27			

Depth (ft BGS)	Graphic Log	Materials Description	Moisture (%)	Remarks
0		Asphalt Cement Concrete (4-inch)		
	GP-GC	Very dense, brown and black, gravelly COBBLE with sand; dry to damp, gravel is angular		No drill fluid circulation below 4 feet BGS
5	BR	Hard (R4), grey, BASALT, slightly weathered, very intensely fractured, rough, irregular and oxidized fractures		
	BR	Becomes intensely to slightly fractured, rough fractures, approximately 30% of fractures are partly healed, very thin infill		
10				
	BR	Becomes intensely fractured, random, rough fractures, very thin to thin infill		
15				
	BR	Becomes slightly fractured, mostly subvertical, rough fractures, some 10 to 20% rough fractures, very thin infill Becomes medium hard to hard (R3-R4), 20% vesicles		
20				
	BR	Becomes intensely to slightly fractured, irregular and rough fracture surfaces		
25				
		Boring terminated at: 25.3 (feet BGS); backfilled with bentonite chips and capped with asphalt cement concrete		
30				

Photograph 1: HQ Rock Core Box #1

Photograph 2: HQ Rock Core Box #2

Photograph 3: HQ Rock Core Box #3