HARDMAN GEOTECHNICAL SERVICES INC.
Practical, Cost-Effective Geotechnical Solutions

March 2, 2016
HGSI Project No. 16-1957

Victor Remmers / Bruce Howard
Everett Custom Homes
735 SW 158th Avenue
Beaverton, Oregon 97006

Copy: Keith Buisman / Mike Peebles, Otak

Via e-mail with hard copies mailed on request

Subject: **GEOTECHNICAL ENGINEERING REPORT**
5920 SW 48TH AVENUE SUBDIVISION
PORTLAND, OREGON

Hardman Geotechnical Services Inc. (HGSI) performed a geotechnical engineering study for the proposed residential development at 5920 SW 48th Avenue in Portland, Oregon (see Vicinity Map, Figure 1). The purpose of the geotechnical study was to explore and evaluate the surface and subsurface conditions at the site, and to provide geotechnical recommendations for foundation design and construction.

PROJECT DESCRIPTION

The site is approximately 2.31 acres in size, rectangular in shape, and is currently occupied by a residential home that is planned to be removed (Figure 2). The site slopes moderately to the south west with a heavily sloped area on the southeast edge of the property. Vegetation around the site consists of many trees and bushes.

The proposed development includes subdividing the property to create 11 lots for single family home construction. The southwest portion of the site will be undeveloped to preserve a sensitive area. A new 300-foot long extension of SW Pendleton Street will provide access to Lots 4-11; the other lots will be accessed off of the existing SW Pendleton Street. Underground utilities are also planned. We anticipate site development will consist of single family residential structures up to three stories in height.

SCOPE OF WORK AND AUTHORIZATION

The scope of work for this geotechnical study was presented in Proposal No. 16-808, dated January 5, 2016. Our field exploration program consisted of site reconnaissance, test pits, infiltration testing, geotechnical analyses, and preparation of this report. This scope of services and our *General Conditions for Geotechnical Services* were authorized by the client prior to the work being performed.

REGIONAL GEOLOGY AND SEISMIC SETTING

The subject site lies within the Portland Basin, a broad structural depression situated between the Coast Range on the west and the Cascade Range on the east. The Portland Basin is a northwest-southwest trending structural basin produced by broad regional downwarping of the area. The Portland Basin is approximately 20 miles wide and 45 miles long and is filled with consolidated and unconsolidated sedimentary rocks of late Miocene, Pliocene and Pleistocene age.

The subject site is underlain by the Quaternary age (last 1.6 million years) Willamette Formation, a catastrophic flood deposit associated with repeated glacial outburst flooding of the Willamette Valley, the last of which occurred about 10,000 years ago (Madin, 1990). Underlying the project site, these deposits consist of horizontally layered, micaceous, silt to coarse sand forming distinct beds less than 3 feet thick. Willamette Formation in the vicinity of the subject site is estimated to be approximately 30 feet thick (Madin, 1990).

At least three major fault zones capable of generating damaging earthquakes are known to exist in the region. These include the Portland Hills Fault Zone, Gales Creek-Newberg-Mt. Angel Structural Zone, and the Cascadia Subduction Zone. These potential earthquake source zones are included in the determination of seismic design values for structures, as presented in the *Seismic Design* section. None of the known faults extend beneath the site.

FIELD EXPLORATION

Subsurface conditions were explored on January 26, 2016 by excavating six test pits and preforming three infiltration tests. The infiltration tests were taken to determine the subgrade conditions for onsite disposal or stormwater.

EXPLORATORY TEST PITS

The site-specific exploration for this study consisted of exploratory test pits. Six test pits, designated TP-1 to TP-6, were excavated to depths of 4 to 9 feet, at approximate locations shown on Figure 2. It should be noted that exploration locations were determined in the field by pacing or taping distances from apparent property corners and other site features shown on the plans provided. As such, the locations of the explorations should be considered approximate.

Explorations were conducted under the full-time observation of HGSI personnel. Soil samples were classified in the field and representative portions were placed in relatively air-tight plastic bags. These soil samples were then returned to the laboratory for further examination and laboratory testing. Pertinent information including soil sample depths, stratigraphy, soil engineering characteristics, and groundwater occurrence was recorded. Soils were classified in general accordance with the Unified Soil Classification System. At the completion of the test pit logging, the explorations were backfilled with the excavated soils and tamped into place.

Summary test pit logs are attached. The stratigraphic contacts shown on the individual logs represent the approximate boundaries between soil types. The actual transitions may be more gradual. The soil and groundwater conditions depicted are only for the specific dates and locations reported, and therefore, are not necessarily representative of other locations and times.

INFILTRATION TESTING

On January 26, 2016, HGSI performed falling head infiltration tests using the stand pipe method in Test Pits TP-3, TP-4, and TP-5. Soils were pre-saturated for several hours prior to testing. Following the soil saturation, infiltration tests were conducted. The water level was measured to the nearest 0.1 inch from a fixed point. The change in water level was recorded at regular intervals over a period of several hours. Table 1 presents the results of the falling head infiltration tests.

Table 1. Summary of Infiltration Test Results

Test Pit	Depth (feet)	Soil Type	Infiltration Rate(in/hr)	Hydraulic Head Range (inches)
TP-3	4.0	Silt (ML)	0.1	36.8 – 36.8
TP-4	7.0	Silt (ML)	0.1	44.4 – 44.4
TP-5	9.0	Silt (ML)	2.0	72.0 – 68.4

SUBSURFACE CONDITIONS

The following discussion is a summary of subsurface conditions encountered in our explorations. For more detailed information regarding subsurface conditions at specific exploration locations, refer to the attached logs. Also, please note that subsurface conditions can vary between exploration locations, as discussed in the *Uncertainty and Limitations* section below.

SOIL

Results of the exploration program indicate that the site is underlain by silt belonging to the Willamette Formation. The observed conditions and soil properties are summarized below.

> ***Topsoil:*** In all test pits the ground surface was directly underlain by topsoil consisting of dark brown, moderately organic silt with fine roots throughout. Topsoil thickness in test pits was about 1 to 2 feet.
>
> ***Silt:*** Beneath the topsoil in all test pits we encountered very stiff, brown mottled with orange and gray, slightly moist, Silt. These soils are interpreted as belonging to the Willamette Formation and extended to the termination of test pits TP-3 through TP-6
>
> ***Clay:*** Below the silt unit in test pits TP-1 and TP-2 we encountered light brown, moist, clay; also interpreted as Willamette Formation materials. These soils extended to the termination of each test pit.

GROUNDWATER

At the time of our explorations, groundwater was not encountered beneath the site. Based on experience and interpolation of a nearby depth to ground water map (Snyder, 2008) we anticipate ground water to be present at a depth of about 40 feet below the existing ground surface at the site. In our experience, it is not uncommon to encounter thin perched groundwater zones within the Willamette Formation in this area, particularly during the wet season.

The groundwater conditions reported above are for the specific date and locations indicated, and therefore may not necessarily be indicative of other times and/or locations. Furthermore, it is anticipated that groundwater conditions will vary depending on the season, local subsurface conditions, changes in land use and other factors.

Conclusions and Recommendations

Results of this study indicate that the proposed development is geotechnically feasible, provided that the recommendations of this report are incorporated into the design and construction phases of the project. Spread foundations are feasible for support of the proposed structures. Additional discussion and recommendations are presented below regarding site preparation, engineered fill, fill slope keying and benching, wet weather earthwork, spread footing foundations, below grade structural retaining walls, perimeter footing drains, seismic design, stormwater infiltration systems, excavating conditions and utility trench backfill, typical pavement sections, and erosion control considerations.

Site Preparation and Undocumented Fill Removal

Proposed areas to receive engineered fill should first be cleared of vegetation and any loose debris or undocumented fill, and debris from clearing should be removed from the site. Organic-rich topsoil should be stripped to the relatively inorganic native soils. We recommend an initial stripping depth of 12 inches, to remove the upper high-organic portion of the topsoil. At that elevation, subgrade conditions should be evaluated to determine if additional measures such as deeper mixing and/or root picking will be needed. Deeper stripping may be needed in areas that have abundant root balls, been tilled in the past, or areas of localized fill deposits, etc. The final depth of stripping removal may vary depending on local subsurface conditions and the contractor's methods, and should be determined on the basis of a site inspection after the initial stripping has been performed.

Stripped organic soil should be stockpiled only in designated areas or removed from the site and stripping operations should be observed and documented by HGSI. Any existing subsurface structures (tile drains, old utility lines, septic leach fields, etc.) beneath structures and pavements should be removed and the excavations backfilled with engineered fill.

During dry weather conditions, once stripping is approved, the area should be ripped or tilled to a depth of 12 inches, moisture conditioned, and compacted in-place prior to the placement of engineered fill or crushed aggregate base for pavement. Exposed subgrade soils should be evaluated by HGSI. For large areas, this evaluation is normally performed by proof-rolling the exposed subgrade with a fully loaded scraper or dump truck. For smaller areas where access is restricted, and during wet weather, the subgrade should be evaluated by probing the soil with a steel probe.

Soft/loose soils, and any undocumented fill identified during site preparation should be compacted to a firm and unyielding condition or over-excavated and replaced with engineered fill, as described below. The depth of overexcavation, if required, should be evaluated by HGSI at the time of construction.

Engineered Fill

On-site native soils are anticipated to be suitable for use as engineered fill during dry weather, provided they are adequately moisture conditioned prior to compacting. Imported fill material should be reviewed by the geotechnical engineer prior to being imported to the site. Oversize material greater than 6 inches in size should not be used within 3 feet of foundation footings, and material greater than 12 inches in diameter should not be used in engineered fill.

Engineered fill should be compacted in horizontal lifts not exceeding 12 inches using heavy vibratory compaction equipment. We recommend that engineered fill be compacted to at least 95% of the maximum dry density determined by Standard Proctor (ASTM D698) or equivalent. We anticipate that aeration of native soil will be necessary for compaction operations.

Proper test frequency and earthwork documentation usually requires daily observation and testing during stripping, rough grading, and placement of engineered fill. Field density testing should conform to ASTM

D2922 and D3017, or D1556. Engineered fill should be periodically observed and tested by HGSI. Typically, one density test is performed for at least every 2 vertical feet of fill placed or every 500 yd^3, whichever requires more testing. Because testing is performed on an on-call basis, we recommend that the earthwork contractor be held contractually responsible for test scheduling and frequency.

FILL EMBANKMENT SLOPES

We recommend that fill slopes for the project be planned no steeper than 2H:1V and be constructed in accordance with the Fill Slope Detail, Figure 3. For fill slopes constructed at 2H:1V or flatter, and comprised of engineered fill placed and compacted as recommended herein, we anticipate that adequate factors of safety against global failure will be maintained.

Prior to placing compacted fill against the existing natural slopes, all loose undocumented fill, topsoil, and soft soils must first be removed. Adequate benching must be maintained. Fill slope keyways should be constructed with a minimum depth of 2 feet and minimum width of H/3 (10 feet minimum), where H equals the vertical height between the base and top of the fill slope. Both benches and keyways should be roughly horizontal in the down slope direction. A subdrain should be incorporated in the fill slope keyway, and HGSI should observe the keyway excavations prior to the placement of fill.

Measures should be taken to prevent surficial instability and/or erosion of embankment material. This can be accomplished by conscientious compaction of the embankment fills all the way out to the slope face, by maintaining adequate drainage, and planting the slope face as soon as possible after construction. To achieve the specified relative compaction at the slope face, it may be necessary to overbuild the slopes several feet, and then trim back to design finish grade. In our experience, compaction of slope faces by "track-walking" is generally ineffective and is therefore not recommended.

WET WEATHER EARTHWORK

The on-site soils are moisture sensitive and may be difficult to handle or traverse with construction equipment during periods of wet weather. Earthwork is typically most economical when performed under dry weather conditions. Earthwork performed during the wet-weather season will probably require expensive measures such as cement treatment or imported granular material to compact fill to the recommended engineering specifications. If earthwork is to be performed or fill is to be placed in wet weather or under wet conditions when soil moisture content is difficult to control, the following recommendations should be incorporated into the contract specifications.

- Earthwork should be performed in small areas to minimize exposure to wet weather. Excavation or the removal of unsuitable soils should be followed promptly by the placement and compaction of clean engineered fill. The size and type of construction equipment used may have to be limited to prevent soil disturbance. Under some circumstances, it may be necessary to excavate soils with a track-mounted excavator to minimize subgrade disturbance caused by equipment traffic;
- The ground surface within the construction area should be graded to promote run-off of surface water and to prevent the ponding of water;
- Material used as engineered fill should consist of clean, granular soil containing less than about 7 percent fines. The fines should be non-plastic. Alternatively, cement treatment of on-site soils may be performed to facilitate wet weather placement;
- The ground surface within the construction area should be sealed by a smooth drum vibratory roller, or equivalent, and under no circumstances should be left uncompacted and exposed to moisture. Soils which become too wet for compaction should be removed and replaced with clean granular materials;

- Excavation and placement of fill should be observed by the geotechnical engineer to verify that all unsuitable materials are removed and suitable compaction and site drainage is achieved; and
- Bales of straw and/or geotextile silt fences should be strategically located to control erosion.

If cement or lime treatment is used to facilitate wet weather construction, HGSI should be contacted to provide additional recommendations and field monitoring.

Spread Footing Foundations

Shallow, conventional isolated or continuous spread footings may be used to support the proposed structures, provided they are founded on competent native soils, or compacted engineered fill placed directly upon the competent native soils. We recommend a maximum allowable bearing pressure of 2,000 pounds per square foot (psf) for designing spread footings bearing on undisturbed native soils or engineered fill. The recommended maximum allowable bearing pressure may be increased by a factor of 1.33 for short term transient conditions such as wind and seismic loading. All footings should be founded at least 18 inches below the lowest adjacent finished grade. Minimum footing widths should be determined by the project engineer/architect in accordance with applicable design codes.

Assuming construction is accomplished as recommended herein, and for the foundation loads anticipated, we estimate total settlement of spread foundations of less than about 1 inch and differential settlement between two adjacent load-bearing components supported on competent soil of less than about ½ inch. We anticipate that the majority of the estimated settlement will occur during construction, as loads are applied.

Wind, earthquakes, and unbalanced earth loads will subject the proposed structure to lateral forces. Lateral forces on a structure will be resisted by a combination of sliding resistance of its base or footing on the underlying soil and passive earth pressure against the buried portions of the structure. For use in design, a coefficient of friction of 0. 5 may be assumed along the interface between the base of the footing and subgrade soils. Passive earth pressure for buried portions of structures may be calculated using an equivalent fluid weight of 360 pounds per cubic foot (pcf), assuming footings are cast against dense, natural soils or engineered fill. The recommended coefficient of friction and passive earth pressure values do not include a safety factor. The upper 12 inches of soil should be neglected in passive pressure computations unless it is protected by pavement or slabs on grade.

Footing excavations should be trimmed neat and the bottom of the excavation should be carefully prepared. Loose, wet or otherwise softened soil should be removed from the footing excavation prior to placing reinforcing steel bars. HGSI should observe foundation excavations prior to placing crushed rock, to verify that adequate bearing soils have been reached. HGSI should monitor crushed rock placement beneath foundations and perform density tests to verify compliance with the engineered fill density specification.

Below-Grade Structural Retaining Walls

Lateral earth pressures against below-grade retaining walls will depend upon the inclination of any adjacent slopes, type of backfill, degree of wall restraint, method of backfill placement, degree of backfill compaction, drainage provisions, and magnitude and location of any adjacent surcharge loads. At-rest soil pressure is exerted on a retaining wall when it is restrained against rotation. In contrast, active soil pressure will be exerted on a wall if its top is allowed to rotate or yield a distance of roughly 0.001 times its height or greater. If the subject retaining walls will be free to rotate at the top, they should be designed for an active earth pressure equivalent to that generated by a fluid weighing 35 pcf for level backfill against the wall. For restrained walls, an at-reset equivalent fluid pressure of 54 pcf should be used in design, again assuming level backfill against the wall. These values assume that the recommended drainage provisions are incorporated, and hydrostatic pressures are not allowed to develop against the wall.

During a seismic event, lateral earth pressures acting on below-grade structural walls will increase by an incremental amount that corresponds to the earthquake loading. Based on the Mononobe-Okabe equation and peak horizontal accelerations appropriate for the site location, seismic loading should be modeled using the active or at-rest earth pressures recommended above, plus an incremental rectangular-shaped seismic load of magnitude 5H, where H is the total height of the wall.

We assume relatively level ground surface below the base of the walls. As such, we recommend passive earth pressure of 360 pcf for use in design, assuming wall footings are cast against competent native soils or engineered fill. If the ground surface slopes down and away from the base of any of the walls, a lower passive earth pressure should be used and HGSI should be contacted for additional recommendations.

A coefficient of friction of 0.5 may be assumed along the interface between the base of the wall footing and subgrade soils. The recommended coefficient of friction and passive earth pressure values do not include a safety factor, and an appropriate safety factor should be included in design. The upper 12 inches of soil should be neglected in passive pressure computations unless it is protected by pavement or slabs on grade.

The above recommendations for lateral earth pressures assume that the backfill behind the subsurface walls will consist of properly compacted structural fill, and no adjacent surcharge loading. If the walls will be subjected to the influence of surcharge loading within a horizontal distance equal to or less than the height of the wall, the walls should be designed for the additional horizontal pressure. For uniform surcharge pressures, a uniformly distributed lateral pressure of 0.3 times the surcharge pressure should be added.

The recommended equivalent fluid densities assume a free-draining condition behind the walls so that hydrostatic pressures do not build up. This can be accomplished by placing a 12-inch wide zone of crushed drain rock containing less than 5 percent fines against the walls. A 3-inch minimum diameter perforated, plastic drain pipe should be installed at the base of the walls and connected to a sump to remove water from the crushed drain rock zone. The drain pipe should be wrapped in filter fabric (Mirafi 140N or other as approved by the geotechnical engineer) to minimize clogging. The above drainage measures are intended to remove water from behind the wall to prevent hydrostatic pressures from building up. Additional drainage measures may be specified by the project architect or structural engineer, for damp-proofing or other reasons.

HGSI should be contacted during construction to verify subgrade strength in wall keyway excavations, to verify that backslope soils are in accordance with our assumptions, and to take density tests on the wall backfill materials.

PERIMETER FOOTING DRAINS

To minimize the fluctuation of soil moisture content near structural foundations, we recommend that the structures be constructed with perimeter footing drains. The outside edge of all perimeter footings should be provided with a drainage system consisting of 3-inch minimum diameter perforated plastic pipe embedded in a minimum of 1 ft^3 per lineal foot of clean, crushed drain rock. The drain pipe and surrounding drain rock should be wrapped in non-woven geotextile (Mirafi 140N, or approved equivalent) to minimize the potential for clogging and/or ground loss due to piping. Water collected from the footing drains should be directed into the local storm drain system or other suitable outlet. A minimum 0.5 percent fall should be maintained throughout the drain and non-perforated pipe outlet. The footing drains should include clean-outs to allow periodic maintenance and inspection.

Construction should include typical measures for controlling subsurface water beneath the homes, including positive crawlspace drainage to an adequate low-point drain exiting the foundation, visqueen covering the exposed ground in the crawlspace, and crawlspace ventilation (foundation vents). The homebuyers should be informed and educated that some slow flowing water in the crawlspaces is considered normal and not necessarily detrimental to the home given these other design elements incorporated into its construction. Appropriate design professionals should be consulted regarding crawlspace ventilation, building material selection and mold prevention issues, which are outside HGSI's area of expertise.

Down spouts and roof drains should collect roof water in a system separate from the footing drains in order to reduce the potential for clogging. Roof drain water should be directed to an appropriate discharge point well away from structural foundations. Grades should be sloped downward and away from buildings to reduce the potential for ponded water near structures.

SEISMIC DESIGN

Structures should be designed to resist earthquake loading in accordance with the methodology described in the 2014 Oregon Residential Specialty Code (ORSC). We recommend Site Class D be used for design per the Building Code, Table 1613.5.2. Design values determined for the site using the USGS (United States Geological Survey) *Seismic Design Tool* utility are summarized below in Table 2.

Table 2. Recommended Earthquake Ground Motion Parameters (2014 ORSC)

Parameter	Value
Location (Lat, Long), degrees	45.481, -122.727
Mapped Spectral Acceleration Values (MCE, Site Class B):	
Short Period, S_S	0.998 g
Soil Factors for Site Class D:	
F_a	1.101
$SD_S = 2/3 \times F_a \times S_S$	0.732 g
Seismic Design Category (2014 ORSC Table R301.2.2.1.1)	D_1 $0.50g < SD_S < 0.83g$

Soil liquefaction is a phenomenon wherein saturated soil deposits temporarily lose strength and behave as a liquid in response to earthquake shaking. Soil liquefaction is generally limited to loose, granular soils located below the water table. Following development, on-site soils will consist predominantly of engineered fill or stiff native soils above the water table, which are not considered susceptible to liquefaction. Therefore, it is our opinion that special design or construction measures are not required to mitigate the effects of liquefaction.

STORMWATER INFILTRATION SYSTEMS

Based on results of the infiltration testing, near-surface soils on site exhibit very low to low infiltration rates, as summarized on Table 1. Groundwater was not encountered in test pits advanced to a maximum depth of 9 feet. No indications of seasonal high groundwater were observed.

The designer of the stormwater system should select an appropriate infiltration value based on our test results. The infiltration rates do not incorporate a factor of safety. For the design infiltration rate, the system designer should incorporate an appropriate factor of safety against slowing of the rate over time due to biological and sediment clogging.

Infiltration test methods and procedures attempt to simulate the as-built conditions of the planned disposal system. However, due to natural variations in soil properties, actual infiltration rates may vary from the measured and/or recommended design rates. All systems should be constructed such that potential overflow is discharged in a controlled manner away from structures, and all systems should include an adequate factor of safety. Infiltration rates presented in this report should not be applied to inappropriate or complex hydrological models such as a closed basin without extensive further studies. Evaluating environmental implications of stormwater disposal at this site are beyond the scope of this study.

Excavating Conditions and Utility Trenches

We anticipate that on-site soils can be excavated using conventional heavy equipment such as scrapers and trackhoes to depths of at least 10 feet. Maintenance of safe working conditions, including temporary excavation stability, is the responsibility of the contractor. Actual slope inclinations at the time of construction should be determined based on safety requirements and actual soil and groundwater conditions. All temporary cuts in excess of 4 feet in height should be sloped in accordance with U.S. Occupational Safety and Health Administration (OSHA) regulations (29 CFR Part 1926), or be shored. The existing native soils classify as Type B Soil and temporary excavation side slope inclinations as steep as 1H:1V may be assumed for planning purposes. This cut slope inclination is applicable to excavations above the water table only.

Perched groundwater is likely to be encountered during wet weather season and should be anticipated in excavations and utility trenches.

Vibrations created by traffic and construction equipment may cause some caving and raveling of excavation walls. In such an event, lateral support for the excavation walls should be provided by the contractor to prevent loss of ground support and possible distress to existing or previously constructed structural improvements.

We recommend that structural trench backfill be compacted to at least 95% of the maximum dry density obtained by Standard Proctor (ASTM D698) or equivalent. Initial backfill lift thicknesses for a ¾"-0 crushed aggregate base may need to be as great as 4 feet to reduce the risk of flattening underlying flexible pipe. Subsequent lift thickness should not exceed 1 foot. If imported granular fill material is used, then the lifts for large vibrating plate-compaction equipment (e.g. hoe compactor attachments) may be up to 2 feet, provided that proper compaction is being achieved and each lift is tested. Use of large vibrating compaction equipment should be carefully monitored near existing structures and improvements due to the potential for vibration-induced damage.

Adequate density testing should be performed during construction to verify that the recommended relative compaction is achieved. Typically, at least one density test is taken for every 4 vertical feet of backfill on each 200-lineal-foot section of trench.

Typical Pavement Section

For the on-site local residential street, we recommended the following minimum pavement section for dry weather construction conditions (Table 3).

Table 3. Recommended Minimum Dry Weather Pavement Sections

Material Layer	Minimum Thickness (inches) Streets	Compaction Standard
Asphaltic Concrete (AC)	3	92% of Rice Density (top lift) 91% of Rice Density (lower lifts) AASHTO T-209
Crushed Aggregate Base ¾"-0 (leveling course)	2	95% of Modified Proctor ASTM D1557
Crushed Aggregate Base 1½"-0	8	95% of Modified Proctor ASTM D1557
Recommended Subgrade	12	95% of Standard Proctor or approved native

In new pavement areas, the native soil subgrade should be ripped or tilled to a minimum depth of 12 inches, moisture conditioned, and recompacted in-place to at least 95 percent of ASTM D698 (Standard Proctor) or equivalent. In order to verify subgrade strength, we recommend proof-rolling directly on subgrade with a loaded dump truck during dry weather and on top of base course in wet weather. Soft areas that pump, rut, or weave should be stabilized prior to paving.

For planning purposes, we recommend a wet weather section with a minimum subgrade deepening of 6 inches to accommodate a working subbase of additional 1½"-0 crushed rock. Geotextile fabric, Mirafi 500x or equivalent, should be placed on subgrade soils prior to placement of base rock.

In some instances it may be preferable to use Special Treated Base (STB) in combination with overexcavation and increasing the thickness of the rock section. HGSI should be consulted for additional recommendations regarding use of STB in wet weather pavement sections if it is desired to pursue this alternative. Also, cement treating of the roadway subbase is considered sufficient wet weather treatment without an increase in base rock thickness.

The above recommendations are subject to field verification. HGSI should be on-site during construction to verify subgrade strength and to take density tests on the engineered fill, base rock and asphaltic pavement materials.

EROSION CONTROL CONSIDERATIONS

During our field exploration program, we did not observe soil types that would be considered highly susceptible to erosion. Erosion at the site during construction can be minimized by implementing the project erosion control plan, which should include judicious use of bio-bags, silt fences, or other appropriate technology. Straw bales are not allowed in the City of Portland for temporary erosion control. Where used, erosion control devices should be in place and remain in place throughout site preparation and construction. Areas of exposed soil requiring immediate and/or temporary protection against exposure should be covered with either mulch or erosion control netting/blankets.

UNCERTAINTIES AND LIMITATIONS

We have prepared this report for the owner and his/her consultants for use in design of this project only. The conclusions and interpretations presented in this report should not be construed as a warranty of the subsurface conditions. Experience has shown that soil and groundwater conditions can vary significantly over small distances. Inconsistent conditions can occur between explorations that may not be detected by a geotechnical study. If, during future site operations, subsurface conditions are encountered which vary appreciably from those described herein, HGSI should be notified for review of the recommendations of this report, and revision of such if necessary.

Sufficient geotechnical monitoring, testing and consultation should be provided during construction to confirm that the conditions encountered are consistent with those indicated by explorations. Recommendations for design changes will be provided should conditions revealed during construction differ from those anticipated, and to verify that the geotechnical aspects of construction comply with the contract plans and specifications.

Within the limitations of scope, schedule and budget, HGSI executed these services in accordance with generally accepted professional principles and practices in the field of geotechnical engineering at the time the report was prepared. No warranty, expressed or implied, is made. The scope of our work did not include environmental assessments or evaluations regarding the presence or absence of wetlands or hazardous or toxic substances in the soil, surface water, or groundwater at this site.

We appreciate this opportunity to be of service.

Sincerely,

HARDMAN GEOTECHNICAL SERVICES INC.

REGISTERED PROFESSIONAL ENGINEER
64,422PE
OREGON
SEPTEMBER 26, 2000
SCOTT L. HARDMAN

3-2-16

EXPIRES: 06-30-2017

Scott L. Hardman, P.E., G.E.
Principal Geotechnical Engineer

Attachments: References
Figure 1 – Vicinity Map
Figure 2 – Site Plan
Figure 3 – Fill Slope Detail
Logs of Test Pits TP-1 through TP-6

REFERENCES

Beeson, M.H., Tolan, T.L., and Madin, I.P., 1991, Geologic map of the Portland Quadrangle, Multnomah, and Washington Counties, Oregon: Oregon Department of Geology and Mineral Industries Geological Map Series GMS-75, scale 1:24,000.

Madin, I.P., 1990, Earthquake hazard geology maps of the Portland metropolitan area, Oregon: Oregon Department of Geology and Mineral Industries Open-File Report 0-90-2, scale 1:24,000, 22 p.

Snyder, D.T., 2008, Estimated Depth to Ground Water and Configuration of the Water Table in the Portland, Oregon Area: U.S. Geological Survey Scientific Investigations Report 2008–5059, 41 p., 3 plates.

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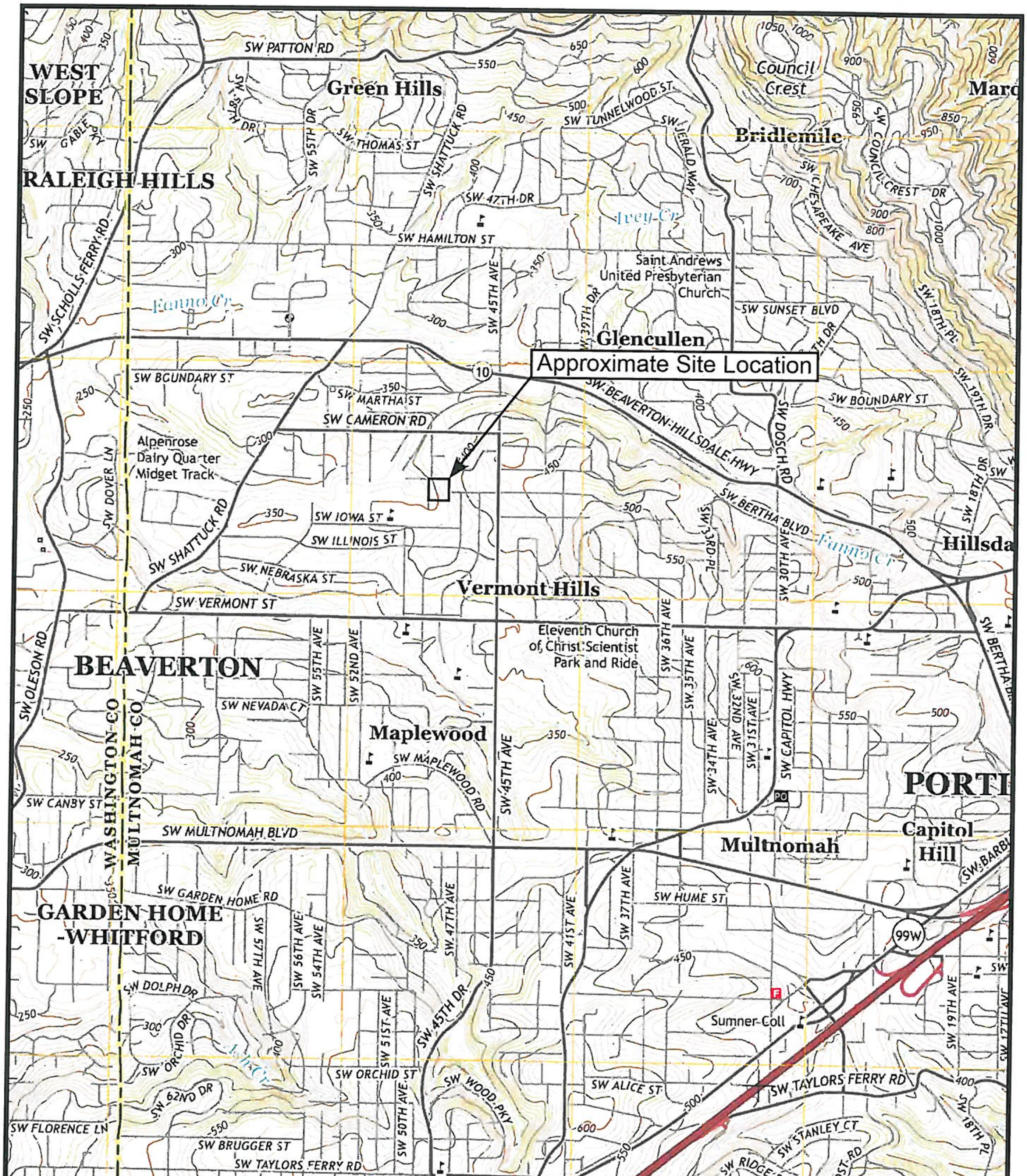
VICINITY MAP



Project: 5920 SW 48th Ave Portland, Oreogn	Project No. 16-1957	FIGURE 1

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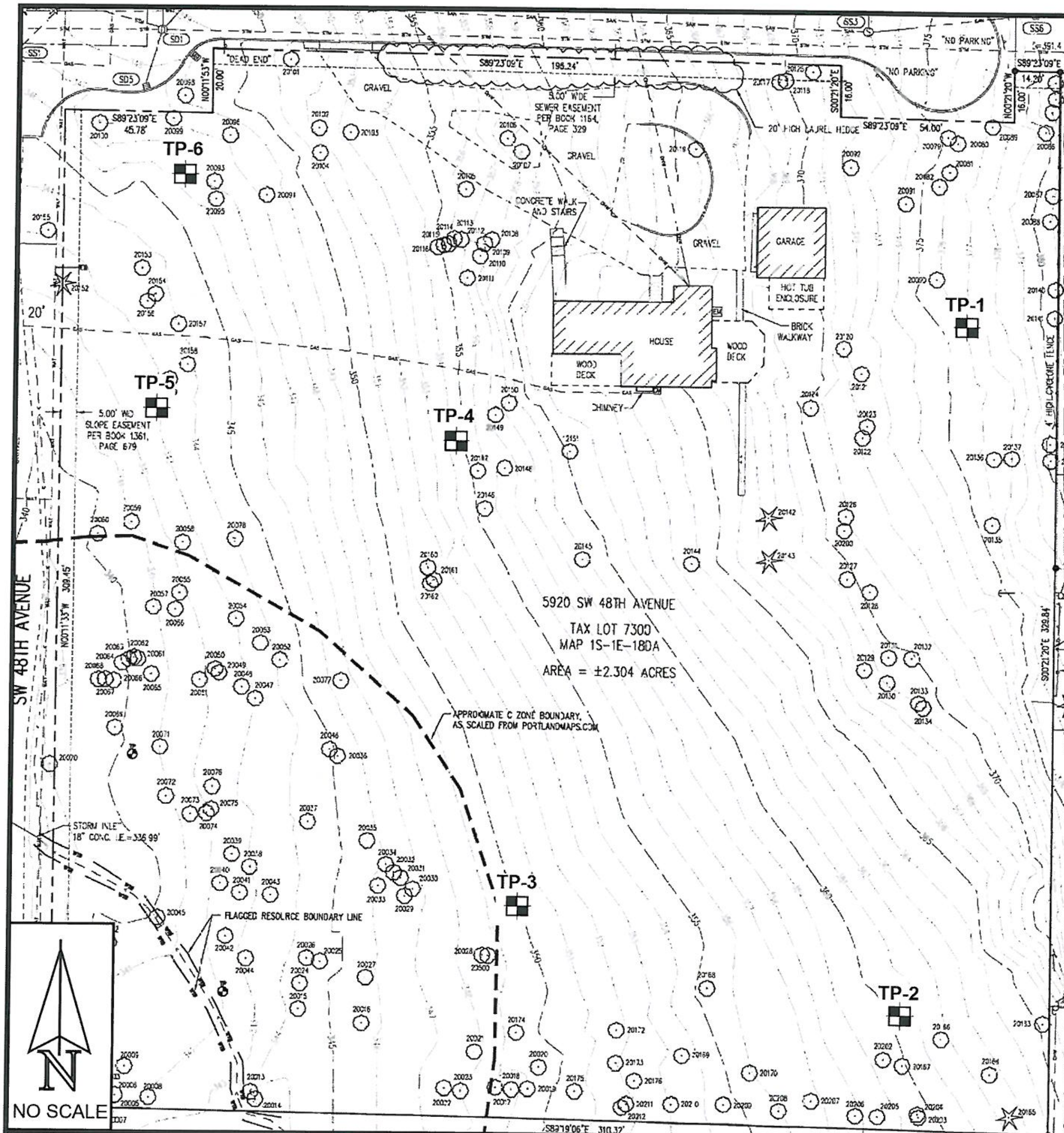
SITE AND EXPLORATION PLAN



Legend

TP-6 Test Pit Designation and Approximate Location

Base map provided by:
Northwest Surveying Inc.

Project: 5920 SW 48th Ave Portland, Oregon	Project No. 16-1957	FIGURE 2

HARDMAN GEOTECHNICAL SERVICES INC.
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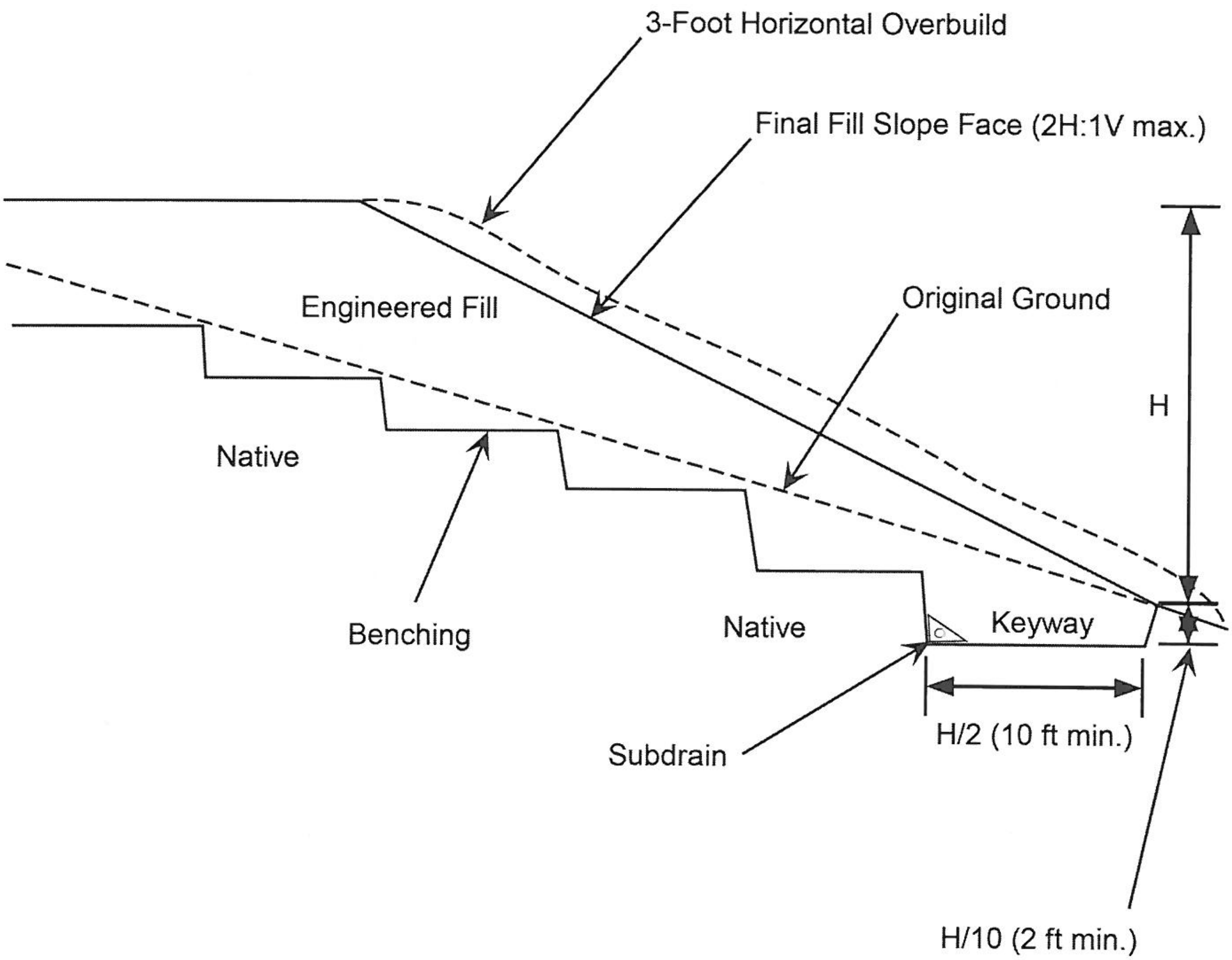
FILL SLOPE DETAIL

TYPICAL KEYWAY, BENCHING & FILL SLOPE DESIGN



Recommended subdrain is minimum 3-inch-diameter ADS Heavy Duty grade (or equivalent), perforated plastic pipe enveloped in a minimum of 3 cubic feet per lineal foot of 2" to 1/2" open-graded gravel drain rock wrapped with geotextile filter fabric (Mirafi 140N or equivalent).

Project: 5920 SW 48th Ave Portland, Oregon	Project No. 16-1957	FIGURE 3

LOG OF BACKHOE / EXCAVATOR TEST PIT

Project: 5920 SW 48th Ave Portland, Oregon	Project No. 16-1957	Test Pit No. TP - 1

Depth (ft)	Pocket Penetrometer (tons/ft²)	Sample Interval	Sample Designation	Moisture Content (%)	Groundwater	Material Description
0–2						Soft to medium stiff, Silt, dark brown, moist, many roots (top soil)
1	0.5					
2	1.25					Very stiff to hard, Silt, brown mottled with orange and gray, slightly moist
3	3.5					
4	>4					
5						Silt, brown, slightly moist
6						
7						
8						Clay, light brown, moist
9						Test pit terminated at 9 feet No groundwater or seepage encountered
10						
11						
12						
13						
14						
15						
16						
17						

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LEGEND

S-1 — Soil Sample Depth Interval and Designation

▽ — Water Level at Time of Excavation

Date Excavated: 1-26-16

Logged By: IDM

Surface Elevation:

LOG OF BACKHOE / EXCAVATOR TEST PIT

Project: 5920 SW 48th Ave Portland, Oregon	Project No. 16-1957	Test Pit No. TP - 2

Depth (ft)	Pocket Penetrometer (tons/ft²)	Sample Interval	Sample Designation	Moisture Content (%)	Groundwater	Material Description
0						Soft to medium stiff, Silt, dark brown, moist, many roots (top soil)
1	0.5					
2	1.25					Very stiff to hard, Silt, brown mottled with orange and gray, slightly moist
3	4.0					
4	3.5					
5						Silt, brown, slightly moist
6						
7						
8						Clay, light brown, moist
9						Test pit terminated at 9 feet Slight seepage was encountered at 2 feet
10						
11						
12						
13						
14						
15						
16						
17						

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S-1 — Soil Sample Depth Interval and Designation

▽ — Water Level at Time of Excavation

Date Excavated: 1-26-16
Logged By: IDM
Surface Elevation:

LOG OF BACKHOE / EXCAVATOR TEST PIT

Project: 5920 SW 48th Ave Portland, Oregon	Project No. 16-1957	Test Pit No. TP - 3

Depth (ft)	Pocket Penetrometer (tons/ft²)	Sample Interval	Sample Designation	Moisture Content (%)	Groundwater	Material Description
0–2						Soft to medium stiff, Silt, dark brown, moist, many roots (top soil)
1	0.25					
2	1.0					Very stiff, Silt, brown mottled with orange and gray, slightly moist
3	2.5					
4	3.25					Test pit terminated at 4 feet Moderate seepage was encountered at 2 feet
5						
6						
7						
8						
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S-1 — Soil Sample Depth Interval and Designation

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Date Excavated: 1-26-16

Logged By: IDM

Surface Elevation:

LOG OF BACKHOE / EXCAVATOR TEST PIT

Project: 5920 SW 48th Ave Portland, Oregon	Project No. 16-1957	Test Pit No. TP - 4

Depth (ft)	Pocket Penetrometer (tons/ft²)	Sample Interval	Sample Designation	Moisture Content (%)	Groundwater	Material Description
						Very soft to soft, Silt, dark brown, very moist, many roots (top soil)
1	0.0					
2	0.5					Stiff to very stiff, Silt, brown mottled with orange and gray, slightly moist
3	2.0					
4	2.0					
5						
6						
7						Test pit terminated at 7 feet Heavy seepage was encountered at 2 feet
8						
9						
10						
11						
12						
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14						
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16						
17						

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S-1 — Soil Sample Depth Interval and Designation

▽ — Water Level at Time of Excavation

Date Excavated: 1-26-16
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Surface Elevation:

LOG OF BACKHOE / EXCAVATOR TEST PIT

Project: 5920 SW 48th Ave Portland, Oregon	Project No. 16-1957	Test Pit No. TP - 5

Depth (ft)	Pocket Penetrometer (tons/ft²)	Sample Interval	Sample Designation	Moisture Content (%)	Groundwater	Material Description
0						Soft to medium stiff, Silt, dark brown, moist, many roots (top soil)
1	0.5					
2	0.75					Stiff to hard, Silt, brown mottled with orange and gray, slightly moist
3	2.0					
4	>4					
5						
6						
7						
8						
9						Test pit terminated at 9 feet Slight seepage was encountered at 2 feet
10						
11						
12						
13						
14						
15						
16						
17						

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S-1 — Soil Sample Depth Interval and Designation

▽ — Water Level at Time of Excavation

Date Excavated: 1-26-16

Logged By: IDM

Surface Elevation:

LOG OF BACKHOE / EXCAVATOR TEST PIT

Project: 5920 SW 48th Ave Portland, Oregon	Project No. 16-1957	Test Pit No. TP - 6

Depth (ft)	Pocket Penetrometer (tons/ft²)	Sample Interval	Sample Designation	Moisture Content (%)	Groundwater	Material Description
0						Medium stiff to stiff, Silt, dark brown, moist, many roots (top soil)
1	0.5					
2	2.25					Very stiff, Silt, brown mottled with orange and gray, slightly moist
3	3.75					
4	3.75					
5						
6						
7						
8						Test pit terminated at 8 feet No groundwater or seepage encountered
9						
10						
11						
12						
13						
14						
15						
16						
17						

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S-1 — Soil Sample Depth Interval and Designation

Water Level at Time of Excavation

Date Excavated: 1-26-16
Logged By: IDM
Surface Elevation: