Development Services

From Concept to Construction

Phone: 503-823-7300 Email: bds@portlandoregon.gov 1900 SW 4th Ave, Portland, OR 97201
More Contact Info (http://www.portlandoregon.gov//bds/article/519984)

APPEAL SUMMARY

Status: Hold for Additional Information

Appeal ID: 19032	**Project Address:** 2834 SE Gladstone St
Hearing Date: 2/20/19	**Appellant Name:** Westin Glass
Case No.: B-007	**Appellant Phone:** 5033587235
Appeal Type: Building	**Plans Examiner/Inspector:** Maureen McCafferty
Project Type: commercial	**Stories:** 2 **Occupancy:** R-3, M **Construction Type:** V-B
Building/Business Name: Jolene's First Cousin	**Fire Sprinklers:** Yes - Throughout
Appeal Involves: Erection of a new structure	**LUR or Permit Application No.:** 17-237371-CO, 17-237384-CO
Plan Submitted Option: pdf [File 1] [File 2]	**Proposed use:**

APPEAL INFORMATION SHEET

Appeal item 1

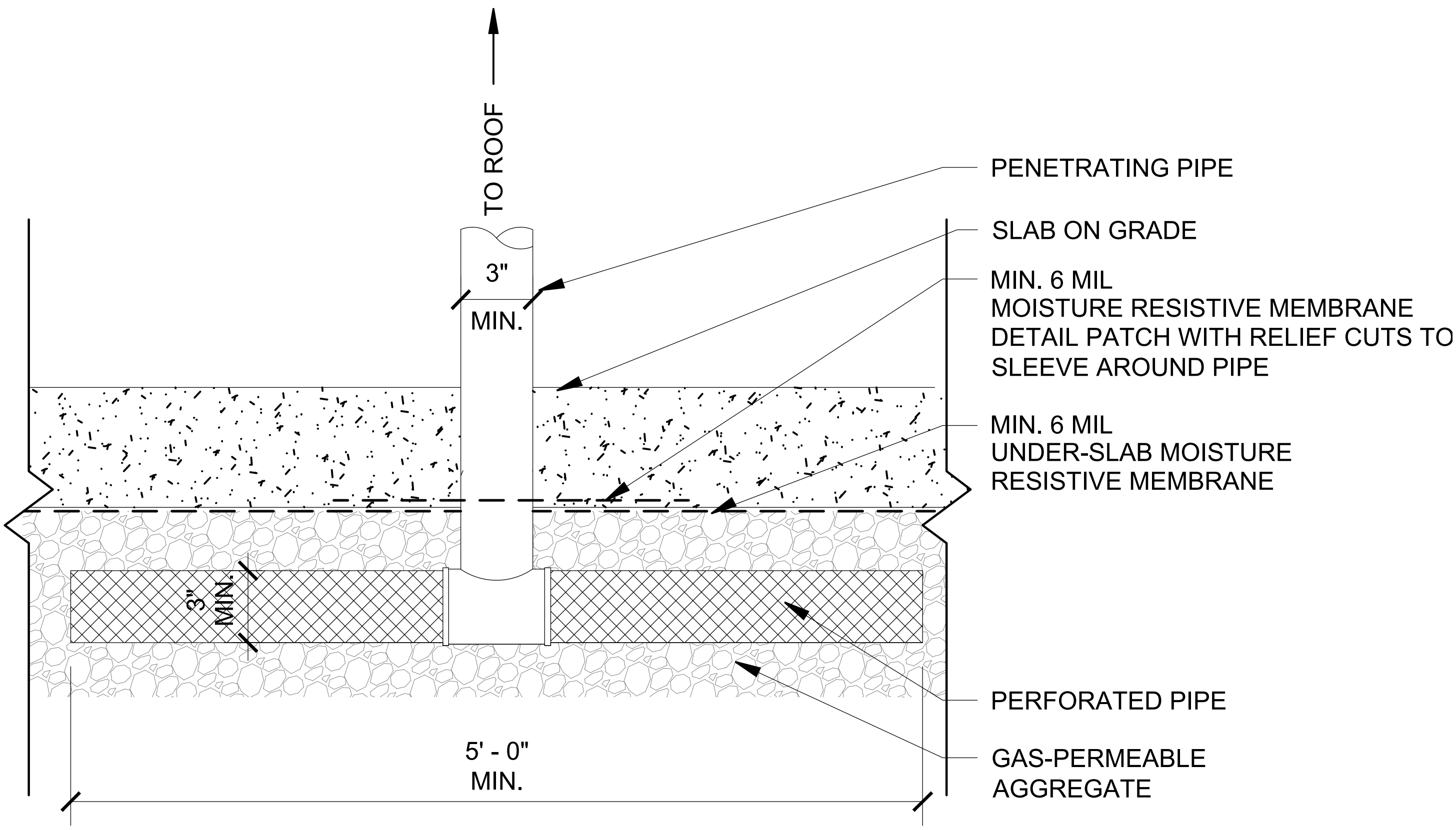
Code Section	1812.3.2
Requires	OSSC Section 1812.3.2 Subfloor preparation. A layer of gas-permeable material shall be placed under all concrete slabs...that directly contact the ground and are within the walls of the living spaces of the building to facilitate future installation of a subslab depressurization system, if needed. The gas-permeable layer shall consist of one of the following: A uniform layer of clean aggregate, a minimum of 4 inches thick. The aggregate shall consist of material that will pass through a 2-inch sieve and be retained by a 1/4-inch sieve.
Proposed Design	Due to a misunderstanding, the General Contractor used a "road base" 3/4" minus aggregate as the subslab layer, which does not meet the description of a "clean aggregate" as required by the code. In the area immediately surrounding the radon Subslab Soil Exhaust System Ducts (SSESDs), he did use code-compliant "clean aggregate." This area extends approximately one foot from the SSESDs on all sides, and for the full depth of the subslab aggregate. (See attached photos.) Radon testing performed in August of 2017 in the basement of the single-family home that previously occupied the site showed very low levels of radon, approximately half of the Environmental Protection Agency’s recommended action level of 4.0 pCi/L. (Radon survey attached.) Geotech evaluation of the site shows silt at a depth of 4.5, and sand at a depth of 10ft. (Geotech report attached.)
Reason for alternative	Applicant believes that, given the low radon levels and existing soil makeup of the site, the subslab aggregate and SSESDs as installed, along with a soil-gas-retarder compliant with 1812.3.3, and proper sealing of entry routes compliant with 1812.3.4, will provide sufficient evacuation of any

small amount of radon that may be present on the site, and provide an acceptable level of protection from radon to the residents of the new building.

Since the subslab aggregate used does not qualify as gas-permeable, it would be safe to assume that any radon gas percolating through the soil below would be forced to migrate laterally through the silty layer until it reaches the permeable area surrounding the SSESD, at which point it would be evacuated through the SSESD to the open air above the building's roof.

APPEAL DECISION

Use of 3/4 inch minus road base gravel in lieu of gas-permeable clean aggregate: Hold for additional information. Proposal does not provide equivalent Life Safety protection.
Appellant may contact John Butler (503 823-7339) with questions.



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RADON SSESD DETAIL

1 1/2" = 1'-0" @ 22 X 34, 3/4"=1'-0" @ 11 X 17

EcoTech environmental services
PO Box 83096 Portland, OR 97283
Phone (503) 493-1040
Fax (503) 493-1042
www.ecotechllc.com

Professional Radon Measurement Report for 2834 SE Gladstone St, Portland, OR 97202

This is a report for radon measuring at the above location. The purpose of the measurement was to determine if the concentration of indoor radon might pose a risk to future occupants, specifically if the concentration exceeded 4.0 pCi/L in the indoor air. The measurement was conducted from 8/1/17 through 8/3/17.

A "Continuous Radon Measurement" device (CRM) was deployed, per EPA protocols for real estate transactions. The measuring device was placed in the location indicated below. Tamper detection efforts were used to detect possible disturbance of the device during measuring. The radon concentrations are noted in the table below. The measuring device was processed by EcoTech per NEHA standards.

Summary and Recommendations

The average radon concentration does not exceed the Environmental Protection Agency's recommended action level of 4.0 pCi/L. Oregon law requires disclosure of radon concentrations to prospective buyers. Should you decide to sell your home at this time, you may advertise that it was professionally inspected for radon.

Please call if we may be of further assistance or if you have any questions regarding this test and/or its results.

Respectfully submitted by: Victoria Williams, EcoTech LLC on August 7, 2017

Detailed test results are attached

CCB# OR 155915, WA ECOTEL*963LJ
Certified NEHA Radon Service Provider

Continuous Radon Monitor

Model Number: 1028

Calibration Date: 01/12/2017

Monitor Time: 8/3/2017 14:14

Serial Number: 91917064

CF: 2.76

Inspector Information

EcoTech LLC
7302 N Richmond Ave
Portland
OR-97203
Phone Number: 503-493-1040
License Number: On File

Customer Information

2834 SE Gladstone St
Portland
OR-97202

Site & Condition

Wind: NA

Year Built: NA

SqFt: NA

Atmospheric Condition: NA

Structure Type: NA

Monitor Location: Basement

Test Summary

Start Time: 08/01/2017 11:57

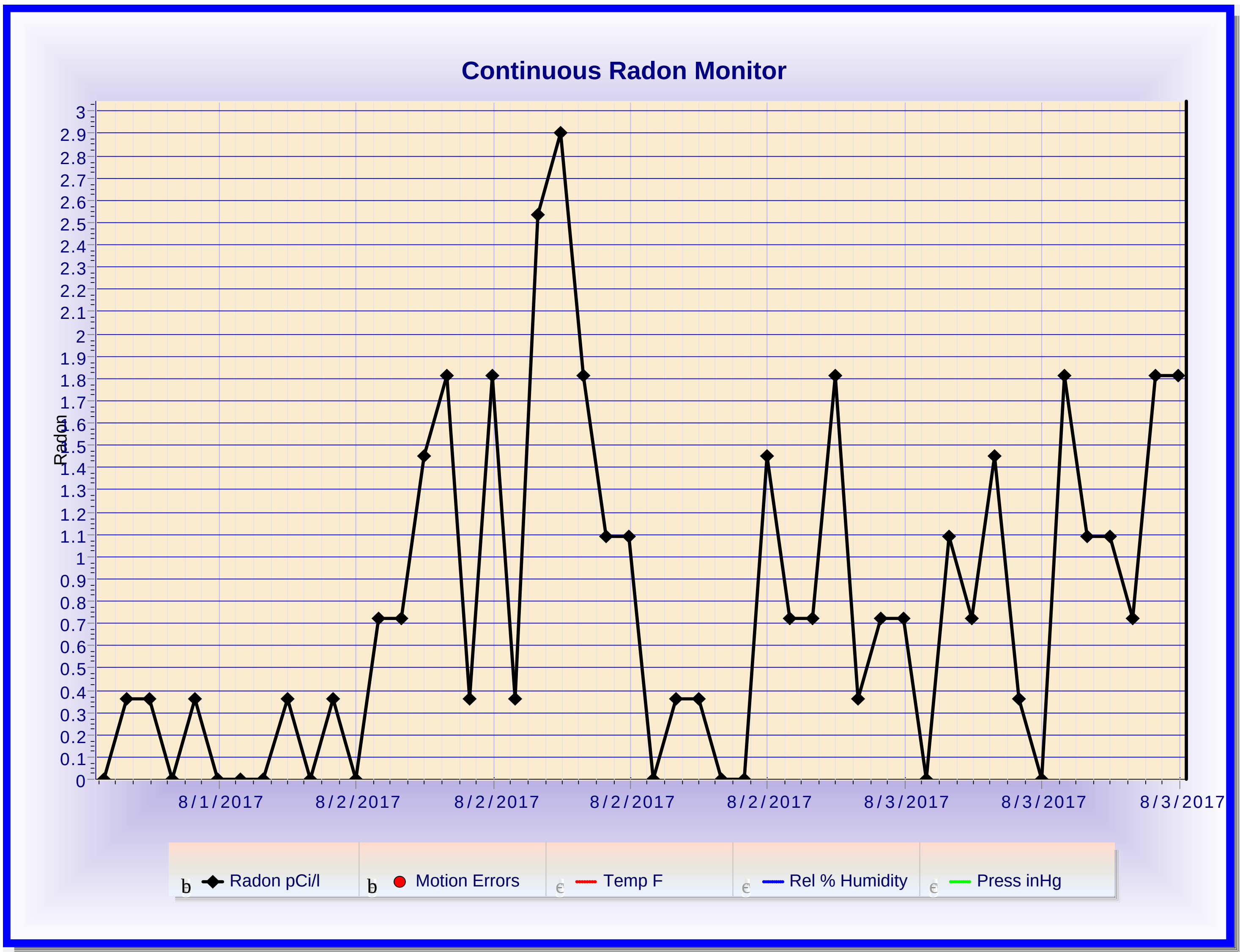
End Time: 08/03/2017 11:57

Measurement Interval(hr): 1.0

Elapse Time: 2 Days 0 hrs

Overall Avg: 0.8 pCi/l

EPA Avg: 0.8 pCi/l



********* 08/01/2017 *********

Time	Counts pCi/l	Flags
12:57	0.0	
13:57	0.4	
14:57	0.4	
15:57	0.0	
16:57	0.4	
17:57	0.0	
18:57	0.0	
19:57	0.0	
20:57	0.4	
21:57	0.0	
22:57	0.4	
23:57	0.0	

********* 08/02/2017 *********

Time	Counts pCi/l	Flags
00:57	0.7	
01:57	0.7	
02:57	1.4	
03:57	1.8	
04:57	0.4	
05:57	1.8	
06:57	0.4	
07:57	2.5	
08:57	2.9	
09:57	1.8	
10:57	1.1	
11:57	1.1	
12:57	0.0	
13:57	0.4	
14:57	0.4	
15:57	0.0	
16:57	0.0	
17:57	1.4	
18:57	0.7	
19:57	0.7	
20:57	1.8	
21:57	0.4	
22:57	0.7	
23:57	0.7	

********* 08/03/2017 *********

Time	Counts pCi/l	Flags
00:57	0.0	
01:57	1.1	

********* 08/03/2017 *********

Time	Counts pCi/l	Flags
02:57	0.7	
03:57	1.4	
04:57	0.4	
05:57	0.0	
06:57	1.8	
07:57	1.1	
08:57	1.1	
09:57	0.7	
10:57	1.8	
11:57	1.8	

Error Flags:
M Motion:

Inspector Signature ____

PC Software Version: 2.1.3.0
Embedded Software Version: 109

August 14, 2017
HGSI Project No. 17-2212

Kevin Cavenaugh
Guerrilla Development
2500 NE Sandy Blvd, Suite C
Portland, OR 97232

kevin@guerilladev.co

Copy: Brett Schulz, brett@brettschulz.com

Via email with hard copies mailed on request

Subject: GEOTECHNICAL ENGINEERING REPORT
2834 SE GLADSTONE STREET
PORTLAND, OREGON

This report presents the results of a geotechnical engineering study conducted by Hardman Geotechnical Services Inc. (HGSI) for the above-referenced project. The purpose of this study was to evaluate subsurface conditions at the site and to provide geotechnical recommendations for site development. This geotechnical study was performed in accordance with HGSI Proposal No. 16-612, dated July 9, 2016, and your subsequent authorization of our proposal and *General Conditions for Geotechnical Services*.

SITE DESCRIPTION AND PROPOSED DEVELOPMENT

The site consists of a 0.11 acre rectangular property located at 2834 SE Gladstone Street in Portland, Oregon (Figure 1). The site currently has one single family residence originally constructed in 1900. Vegetation consists of large trees and lawn. The site is gently sloping.

Preliminary plans indicate the exiting building will be demolished and a new mixed use building up to 4 stories high will be constructed. Appurtenant facilities would include underground utilities and storm water facilities.

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 sub-rounded gravels and cobbles with silty sand.

At least three major seismic source 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.

FIELD EXPLORATION

Exploratory Hand Auger Borings

The site-specific exploration for this study was conducted on June 30, 2017 and consisted of two hand auger borings (designated HA-1 and HA-2) excavated to depths of 10 and 4.5 feet below ground surface (bgs), respectively. Approximate locations of the hand auger borings are 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 obtained from the borings 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. 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.

Summary hand auger logs are attached to this report. The stratigraphic contacts shown on the individual borehole 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 June 30, 2017, HGSI performed open-hole falling head infiltration tests in borings HA-1 and HA-2. Soils in the boring were pre-saturated prior to testing. Following the soil saturation, the infiltration test was conducted. The water level was measured to the nearest 0.1 inch from a fixed point. The change in water level was recorded at intervals for a total period of at least 2 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)
HA-1	10	Sand	48	60 - 20
HA-2	4.5	Silt	2	24 - 22

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 hand auger logs. Also, please note that subsurface conditions can vary between exploration locations, as discussed in the *Uncertainty and Limitations* section below.

Soil

On-site soils are anticipated to consist of silts and sands as described below.

> ***Silt –*** Underlying the surface, hand augers encountered medium stiff to stiff silt. This silt was brown in color usually moist and slightly micaceous, the sand content increased with depth. We interpreted this unit as belonging to the Willamette Formation. Silt extended to the maximum depth of hand auger HA-2, 4.5 feet bgs, and to a depth of about 9 feet bgs in HA-1.
>
> ***Sand –*** Beneath the silt hand auger HA-1 encountered medium dense sand at a depth of about 9 feet bgs. The soil was moist with and extended to the termination of HA-1 at 10 feet bgs. This unit was also interpreted as belonging to the Willamette Formation.

Groundwater

During the field exploration, groundwater was not encountered. Regional geologic mapping (Snyder, 2008) indicates that static groundwater is present at a depth of about 40 to 60 feet below the existing ground surface at the site. Perched groundwater conditions often occur over fine-grained native deposits such as those beneath the site, particularly during the wet season. It is anticipated that groundwater conditions will vary depending on the season, local subsurface conditions, changes in site utilization, and other factors. 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.

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. Recommendations are presented below regarding site preparation, engineered fill, wet weather earthwork, structural foundations, concrete slabs-on-grade, perimeter footing drains, seismic design, stormwater infiltration systems, excavating conditions and utility trench backfill and erosion control considerations.

Site Preparation

Proposed structure and parking areas should be cleared of debris. Undocumented fill within the proposed building footprints, beneath pavements or other settlement-sensitive improvements, should be completely removed and replaced with engineered fill. Following removal of surficial debris and undocumented fill, the exposed subgrade 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, the subgrade should be evaluated by probing the soil with a steel probe. Soft/loose soils identified during subgrade 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.

Following removal, portions of undocumented fill soils that do not contain organic or other deleterious material may be re-used as engineered fill during dry-weather construction. HGSI should be on-site during fill removal and recompaction efforts, to verify the suitability of soils for recompaction and to monitor the fill placement and compaction efforts. During wet weather, the predominantly silt undocumented fill soils will likely not be usable as engineered fill due to their moisture-sensitive nature.

If the removal and replacement with engineered fill option is selected for remediation of undocumented fill materials, there will likely be significant shoring required around the perimeter of the site to avoid destabilizing adjacent structures and street improvements. Additional information with regards to shoring is presented in a separate section below.

Based on the age of the existing structures on site, it is possible that one or more old dry wells may be present. In the event that old drywell(s) are encountered during site development, the following recommendations are made. Deeper portions of dry wells should be backfilled with controlled density fill (CDF), which is essentially a lean mix concrete consisting of water, sand and cement. We recommend use of "excavatable" CDF so that future excavations can be made through the dry well backfill if any new utilities or other excavations are needed in the affected areas. Above a depth of about 8 feet, at the contractor's option, backfill may consist of granular soils such as "reject rock," recycled concrete or similar material approved by HGSI. The granular backfill should be placed in lifts no thicker than about 18 inches and compacted with a "hoe-pac" excavator attachment to a minimum of 90 percent of Modified Proctor (ASTM D-1557). This backfill specification should also be used for any basements or other depressions that require fill during the demolition process.

Engineered Fill

In general, we anticipate that on-site soils will be suitable for use as engineered fill in dry weather conditions, provided they are relatively free of organics and are properly moisture conditioned for compaction. Imported fill material must be approved 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 8 inches using standard compaction equipment. We recommend that engineered fill be compacted to at least 90 percent of the maximum dry density determined by ASTM D1557 (Modified Proctor) or equivalent. On-site soils may be wet or dry of optimum; therefore, we anticipate that moisture conditioning 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 the project geotechnical engineer or his representative. 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.

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, HGSI should be contacted for additional recommendations.

Under wet weather, the construction area will unavoidably become wet and the condition of exposed fill and native soils will degrade. To limit the impacts of wet weather on the finished building pad surface, consideration may be given to placement of a crushed aggregate pad. Where used, we recommend the working pad be constructed using 1½"–0 crushed aggregate, and should have minimum thickness of at least

12 inches. This thickness is considered adequate to support light construction traffic, but will not be sufficient to support heavy traffic such as loaded dump trucks or other heavy rubber-tired equipment.

Structural 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 the footings. The recommended maximum allowable bearing pressure may be increased by 1/3 for short term transient conditions such as wind and seismic loading. Minimum footing depths and 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 390 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. All loose or softened soil should be removed from the footing excavation prior to placing reinforcing steel bars. We recommend that footing excavations be observed by HGSI prior to placing steel and concrete, to verify that the recommendations of this report have been followed, and that an appropriate bearing stratum has been exposed.

The above foundation recommendations are for dry weather conditions. Due to the high moisture sensitivity of engineered fill and native soils on the lots, houses constructed during the wet weather season are likely to require overexcavation of footings and backfill with up to 12 inches of compacted, crushed aggregate. The need for, and thickness of crushed rock layer (if needed) should be evaluated by HGSI during our observation of the foundation excavation.

Concrete Slabs-on-Grade

Preparation of areas beneath concrete slab-on-grade floors should be performed as recommended in the *Site Preparation* section. Care should be taken during excavation for foundations and floor slabs, to avoid disturbing subgrade soils. If subgrade soils have been adversely impacted by wet weather or otherwise disturbed, the surficial soils should be scarified to a minimum depth of 8 inches, moisture conditioned to within about 3 percent of optimum moisture content, and compacted to engineered fill specifications. Alternatively, disturbed soils may be removed and the removal zone backfilled with additional crushed rock. For evaluation of the concrete slab-on-grade floors using the beam on elastic foundation method, a modulus of subgrade reaction of 200 kcf (115 pci) should be assumed for the soils anticipated at subgrade depth. This value assumes the concrete slab system is designed and constructed as recommended herein, with a minimum thickness of crushed rock of 8 inches beneath the slab.

Interior slab-on-grade floors should be provided with an adequate moisture break. The capillary break material should consist of ODOT open graded aggregate per ODOT Standard Specifications 02630-2. The minimum recommended thickness of capillary break materials on re-compacted soil subgrade is 8 inches. The total thickness of crushed aggregate will be dependent on the subgrade conditions at the time of construction, and should be verified visually by proof-rolling. Under-slab aggregate should be compacted to at least 90% of its maximum dry density as determined by ASTM D1557 or equivalent.

In areas where moisture will be detrimental to floor coverings or equipment inside the proposed structure, appropriate vapor barrier and damp-proofing measures should be implemented. A commonly applied vapor barrier system consists of a 10-mil polyethylene vapor barrier placed directly over the capillary break material. With this type of system, an approximately 2-inch thick layer of sand is often placed over the vapor barrier to protect it from damage, to aid in curing of the concrete, and also to help prevent cement from bleeding down into the underlying capillary break materials. Other damp/vapor barrier systems may also be feasible. Appropriate design professionals should be consulted regarding vapor barrier and damp proofing systems, ventilation, building material selection and mold prevention issues, which are outside HGSI's area of expertise.

Perimeter Footing Drains

Due to the potential for perched surface water above fine grained deposits and engineered fill such as those encountered at the site, we recommend the outside edge of perimeter footings be provided with a drainage system consisting of 3-inch minimum diameter perforated PVC pipe embedded in a minimum of 1 ft^3 per lineal foot of clean, free-draining sand and gravel or 1"- ¼" 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.

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 2012 International Building Code (IBC) with applicable 2014 Oregon Structural Specialty Code (OSSC) revisions. We recommend Site Class D be used for design per ASCE 7-10, Chapter 20. 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 (2012 IBC / 2014 OSSC)

Parameter	Value
Location (Lat, Long), degrees	45.4932, -122.6369
Mapped Spectral Acceleration Values (MCE, Site Class B):	
Short Period, S_s	0.981 g
1.0 Sec Period, S_1	0.419 g
Site Specific Design Values for Site Class D:	
F_a	1.107
F_v	1.581
$SD_s = 2/3 \times F_a \times S_s$	0.725 g
$SD_1 = 2/3 \times F_v \times S_1$	0.441 g

Potential seismic impacts also include secondary effects such as soil liquefaction, fault rupture potential, and other hazards as discussed below:

- **Soil Liquefaction Potential –** 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. On-site soils generally consist of stiff silt and medium dense to dense sand above the water table. The seasonal high ground water table lies at a mapped depth of 40 to 60 feet below ground surface at the site. It is our opinion that special design or construction measures are not required to mitigate the effects of liquefaction.

- **Fault Rupture Potential** – Based on our review of available geologic literature, we are not aware of any mapped active (demonstrating movement in the last 10,000 years) faults on the site. During our field investigation, we did not observe any evidence of surface rupture or recent faulting. Therefore, we conclude that the potential for fault rupture on site is low.

- **Seismic Induced Landslide** – Topography in the vicinity of the subject site is moderately sloping and the potential for a global failure due to a seismic event is considered low to very low.

- **Effects of Local Geology and Topography** – In our opinion, no additional seismic hazard will occur due to local geology or topography. The site is expected to have no greater seismic hazard than surrounding properties and the Portland area in general.

Stormwater Infiltration Systems

Infiltration test results and observed soil conditions indicate site soils have low to high infiltration characteristics. Near surface soils are comprised of silt and exhibit low infiltration rates. From the geotechnical perspective the site is well suited for use of deep infiltration facilities such as dry wells. An infiltration rate of 48 inches/hour is recommended for facilities that reach into the native sand material, below depths of about 10 feet bgs. Shallow infiltration facilities such as swales, planter boxes, pervious pavements, etc. should be designed for an infiltration rate of 2 inches/hour. The infiltration rate presented herein does 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.

Excavating Conditions and Utility Trench Backfill

We anticipate that on-site soils can be excavated to depths of at least 10 feet using conventional heavy equipment such as trackhoes. Weathered basalt bedrock was not encountered in any of the hand auger borings, excavated to depths of 10 feet bgs.

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, 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. Flatter temporary excavation slopes will be needed if groundwater is present, or if significant thicknesses of sandy soils are present in excavation sidewalls.

Perched groundwater conditions often occur over fine-grained native deposits such as those beneath the site, particularly during the wet season. If encountered, the contractor should be prepared to implement an appropriate dewatering system for installation of the utilities. At this time, we anticipate that dewatering systems consisting of ditches, sumps and pumps would be adequate for control of groundwater where encountered during construction conducted during the dry season. Regardless of the dewatering system used, it should be installed and operated such that in-place soils are prevented from being removed along with the groundwater.

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.

Utility trench backfill should consist of ¾"-0 crushed rock, compacted to at least 90% of the maximum dry density obtained by Modified Proctor (ASTM D1557) or equivalent. Initial backfill lift thick nesses 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, one density test is taken for every 4 vertical feet of backfill on each 200-lineal-foot section of trench.

Erosion Control Considerations

During our field exploration program, we did not observe soil types that would be considered highly susceptible to erosion. In our opinion, the primary concern regarding erosion potential will occur during construction, in areas that have been stripped of vegetation. Erosion at the site during construction can be minimized by implementing the project erosion control plan, which should include judicious use of straw bales and silt fences. If used, these erosion control devices should be in place and remain in place throughout site preparation and construction.

Erosion and sedimentation of exposed soils can also be minimized by quickly re-vegetating exposed areas of soil, and by staging construction such that large areas of the project site are not denuded and exposed at the same time. Areas of exposed soil requiring immediate and/or temporary protection against exposure should be covered with either mulch or erosion control netting/blankets. Areas of exposed soil requiring permanent stabilization should be seeded with an approved grass seed mixture, or hydroseeded with an approved seed-mulch-fertilizer mixture.

UNCERTAINTIES AND LIMITATIONS

We have prepared this report for the owner and his/her consultants for use in design of this project only. This report should be provided in its entirety to prospective contractors for bidding and estimating purposes; however, 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
8-14-17
EXPIRES: 06-30-2019

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

Attachments: References
Figure 1 – Vicinity Map
Figure 2 – Site and Exploration Plan
Logs of Hand Auger Borings HA-1 and HA-2

REFERENCES

Beeson, M.H., Tolan, T.L., and Madin, I.P., 1991, Geologic map of the Lake Oswego 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.

Schlicker, H.G. and Finlayson, C.T., 1979, Geology and Geologic Hazards of northwestern Clackamas County, Oregon: Oregon Department of Geology and Mineral Industries, Bulletin No. 99, 79 p., scale 1:24,000.

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

Base Map: USGS Lake Oswego Quad, from US Topo, 2015

Not to Scale

Project: 2834 SE Gladstone Street Portland, Oregon	Project No. 17-2212	FIGURE 1

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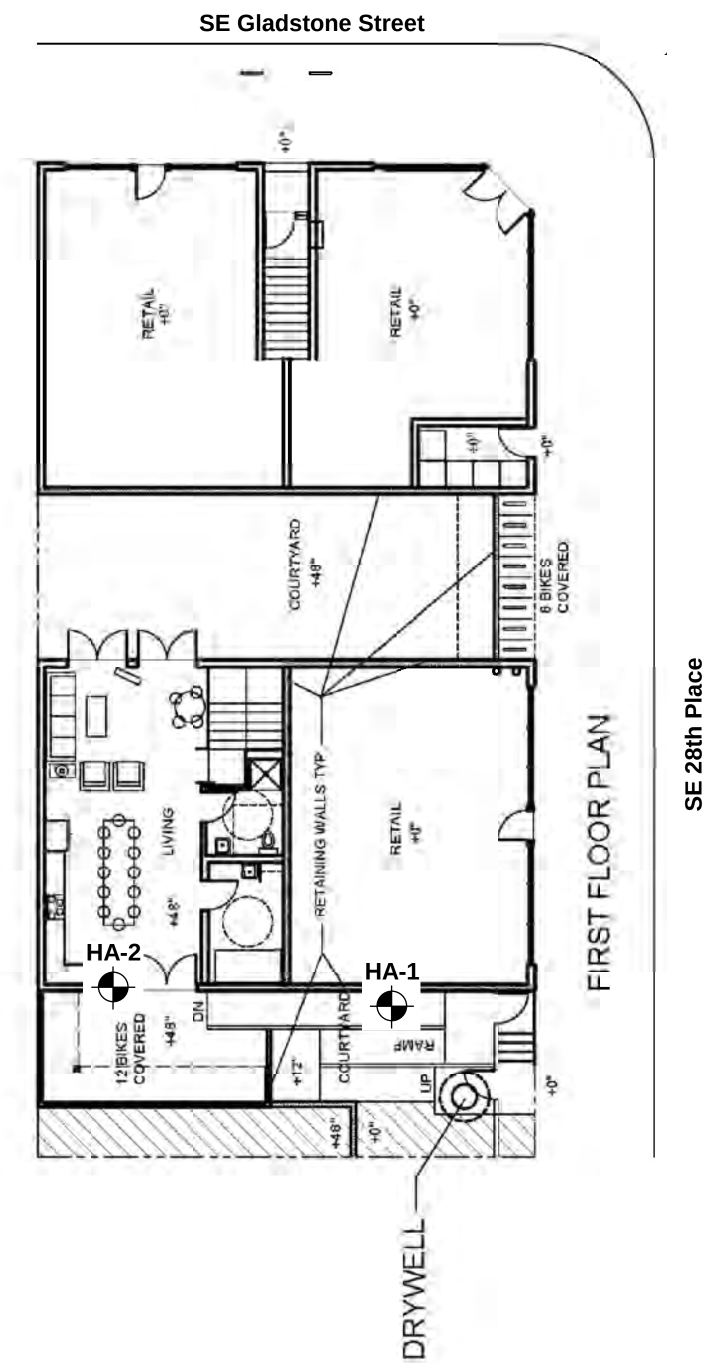
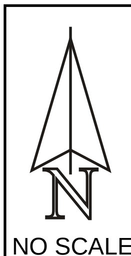
SITE PLAN





Legend

HA-2 Hand Auger Boring

Base map provided by Brett Schulz Architect

Project: 2834 SE Gladstone Street Portland, Oregon	Project No. 17-2212	FIGURE 2

HAND AUGER BORING LOG

Project: 2834 SE Gladstone Street Portland, Oregon	Project No. 17-2212	Boring No. HA-1

Depth (ft)	Sample Interval	Sample Designation	In-Situ Dry Density (lb/ft^3)	Moisture Content (%)	Groundwater	Material Description
						Soft, organic SILT (OL), bark brown, dry (Topsoil)
2						Medium stiff, SILT with sparse angular gravel (ML) and an interbedded layer of fine SAND (SP), brown, dry (Fill)
4						Stiff, SILT (ML), micaceous, brown with subtle gray mottling, dry (Willamette Formation) Slightly moist below 4.5 feet
6						
8						Trace fine sand below 7.5 feet
						Medium dense, fine SAND (SP), light brown, slightly moist (Willamette Formation)
10						Boring terminated at 10 feet No groundwater or seepage encountered
12						
14						
16						

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Practical Cost-Effective Geotechnical Solutions
10110 SW Nimbus Avenue, Suite B-5
Portland, Oregon 97223
(503) 530-8076

LEGEND
S-1 Soil Sample Depth Interval and Designation
Water Level at Time of Drilling

Date Drilled: 6/30/17
Logged By: PBR

HAND AUGER BORING LOG

Project: 2834 SE Gladstone Street Portland, Oregon	Project No. 17-2212	Boring No. HA- 2

Depth (ft)	Sample Interval	Sample Designation	In-Situ Dry Density (lb/ft^3)	Moisture Content (%)	Groundwater	Material Description
0–0.5						Medium stiff, SILT with sparse debris (ML), brown, dry (Fill)
0.5–4.5						Stiff to very stiff, SILT (ML), micaceous, brown with subtle gray mottling, dry to slightly moist (Willamette Formation)
4.5						Boring terminated at 4.5 feet No groundwater or seepage encountered

Depth scale (ft): 2, 4, 6, 8, 10, 12, 14, 16

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LEGEND

S-1 Soil Sample Depth Interval and Designation

Water Level at Time of Drilling

Date Drilled: 6/30/17

Logged By: PBR