REPORT OF GEOTECHNICAL ENGINEERING SERVICES

Block 8
1625 Main Street
Washougal, Washington

For
Lone Wolf Development, LLC
July 20, 2018

GeoDesign Project: LoneWolf-7-01

GEODESIGN INC

July 20, 2018

Lone Wolf Development, LLC
P.O. Box 1126
Washougal, WA 98671

Attention: Wes Hickey

Report of Geotechnical Engineering Services
Block 8
1625 Main Street
Washougal, Washington
GeoDesign Project: LoneWolf-7-01

GeoDesign, Inc. is pleased to submit this report for the proposed Block 8 development located at 1625 Main Street in Washougal, Washington. Our services for this project were conducted in accordance with our proposal dated May 17, 2018.

We appreciate the opportunity to be of service to you. Please contact us if you have questions regarding this report.

Sincerely,

GeoDesign, Inc.

Nick Paveglio, P.E.
Senior Associate Engineer

Jeffery D. Tucker, P.E.
Principal Engineer

NNP:JDT:kt
Attachments
One copy submitted (via email only)
Document ID: LoneWolf-7-01-072018-geor.docx

703 Broadway Street, Suite 650 I Vancouver, WA 98660 I 360.693.8416 www.geodesigninc.com

EXECUTIVE SUMMARY

The primary geotechnical considerations for the project are summarized as follows:

- The proposed building can be supported by conventional spread footings founded on the native soil underlying the site.

- While not directly observed in our explorations completed at the site, we anticipate undocumented fill associated with demolition of the building in the southeast portion of site will be present. Typically undocumented fill can be up to 3 feet thick, with the exception of drywells or other deeper subsurface elements. Based on the current layout of the building and the planned basement, we anticipate that any fill will be removed as part of the building construction. Should the basement be eliminated and footings are planned near grade, all undocumented fill will need to be completely removed from beneath building footings and replaced with compacted crushed rock.

  Undocumented fill could possibly be left beneath floor slabs and pavements, provided it meets the requirements described in the "Site Preparation" section. The geotechnical engineer should be notified to observe all undocumented fill to determine if it is suitable for use at the site.

- We recommend conventional soldier pile shoring with or without tieback anchors for support of the excavation for the below-grade level where sloping is not an option. Tieback anchors are required where existing structures or sensitive utilities are adjacent to the excavation to reduce lateral movements that will cause damage to the elements. Cantilever shoring might be possible to support the excavation where it is adjacent to the right-of-way if some settlement can be tolerated.

- Cobbles and boulders up to 3.5 feet in diameter were observed at all depths during our test pit explorations. When encountered, cobbles and especially boulders will result in difficult excavation conditions and may require special equipment and procedures for removal. The earthwork contractor should be prepared for wider than expected excavations and increased backfill material. Cobbles and boulders will also present challenges during soldier pile installation.

- The near-surface silty soil can be sensitive to disturbance when at a moisture content that is above optimum. As discussed in the "Construction" section, the subgrade should be protected from disturbance and damage by construction traffic.

- We recommend that confirmation infiltration testing be completed at the time of construction to verify the design infiltration rates. We also recommend that a contingency be in place if rates during confirmation testing do not meet the design rates.

- Based on the results of our explorations, the soil at the site is not subject to liquefaction or lateral spreading.

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ACRONYMS AND ABBREVIATIONS

AC	asphalt concrete
AOS	apparent opening size
ASTM	American Society for Testing and Materials
BGS	below ground surface
g	gravitational acceleration (32.2 feet/second^2)
GIS	geographic information system
HMA	hot mix asphalt
H:V	horizontal to vertical
IBC	International Building Code
ksf	kips per square foot
MCE	maximum considered earthquake
OSHA	Occupational Safety and Health Administration
pcf	pounds per cubic foot
pci	pounds per cubic inch
PG	performance grade
psf	pounds per square foot
psi	pounds per square inch
USGS	U.S. Geological Survey
WSS	Washington Standard Specifications for Road, Bridge, and Municipal Construction (2018)

1.0 INTRODUCTION

This report presents the results of our geotechnical engineering evaluation for the proposed Block 8 development at 1625 Main Street in Washougal, Washington. The site is shown relative to surrounding features on Figure 1. Figure 2 shows the locations of applicable explorations associated with project. Acronyms and abbreviations used herein are defined above, immediately following the Table of Contents.

2.0 PROJECT UNDERSTANDING

The approximately 0.7-acre site is located on three-quarters of a city block in downtown Washougal, Washington. The site is currently undeveloped and covered by gravel and AC. Aerial photography indicates that the southeast portion of the site was previously occupied by a single-story building until 2017, when it was demolished. The west portion of the site is an AC parking area that has been in place since at least 1990.

The proposed development includes construction of a four-story structure with a one-level basement and footprint of approximately 15,000 square feet. The building will be constructed on the south half of the site and the north half of the site will consist of at-grade AC vehicle parking and landscaping. Structural loading was unknown at the time of our report; however, we anticipate that the building column loads will be less than 400 kips, wall loads less than 8 kips per foot, and floor loads less than 200 psf. We anticipate that site cuts will be up to 15 feet BGS for the basement. Stormwater from development will be infiltrated on site.

3.0 PURPOSE AND SCOPE

The purpose of our services was to provide geotechnical engineering recommendations for design and construction of the proposed development. The specific scope of our services is summarized as follows:

- Reviewed available geologic and geotechnical reports for the site vicinity.
- Coordinated and managed the field explorations, including utility locates and scheduling subcontractors and GeoDesign field staff.
- Excavated two test pits to depths between 7.0 and 7.5 feet BGS.
- Completed three infiltration tests in locations requested by the civil engineer.
- Collected soil samples for laboratory testing at select depths from the explorations.
- Classified the material encountered in the explorations.
- Maintained a detailed log of each exploration. Observed groundwater conditions in the explorations.
- Completed a laboratory testing program that included the following:
  - Two particle-size analyses in general accordance with ASTM D1140
  - Two sieve analyses in general accordance with ASTM C117 and ASTM C136
  - Four moisture content determinations in accordance with ASTM D2216

- Prepared this report summarizing our findings, conclusions, and recommendations, including information related to the following:
    - Subsurface soil and groundwater conditions
    - Site preparation, grading and drainage, clearing and grubbing, fill type for imported material, compaction criteria, trench excavation and backfill, use of on-site soil, and wet/dry weather earthwork
    - Temporary shoring recommendations
    - Foundation recommendations, including bearing capacity and guidelines for shallow foundation design
    - Infiltration testing results
    - Pavements
    - Seismic design parameters in accordance with 2015 IBC

4.0 SITE CONDITIONS

4.1 GEOLOGIC CONDITIONS

The site is located in the east-central part of the Portland Basin physiographic province, which is bound by the Tualatin Mountains to the west and south and the Cascade Range to the east and north. The near-surface geologic unit is mapped as recent Holocene alluvium deposited in stream channels and on floodplains. The unit consists of unconsolidated gravel, sand, and silt (Mundorff, 1964). Underlying the Holocene alluvium is unconsolidated Pleistocene alluvium consisting of deltaic sand and gravel, fine sand, and silt deposited by the catastrophic Glacial Lake Missoula Floods, which occurred between 13,000 and 15,500 years ago (Palmer et al, 2004).

Underlying the flood deposits is the Troutdale Formation consisting of sand and gravel and a lower member of silt and clay. Tectonic events beginning 25 million years ago resulted in a large depression extending from Clark County to an area near Portland. Sediments from the surrounding mountains were transported by streams and deposited in the basin to make up the highly variable formation (Mundorff, 1964). Below the Troutdale Formation lies a variety of consolidated volcanics consisting of andesite, basalt, pyroclastics, and agglomerates with sedimentary interbeds (Mundorff, 1964).

4.2 SURFACE CONDITIONS

The approximately 0.7-acre site is located on three-quarters of a city block in Washougal, Washington. The northeast one-quarter of the block that is not part of the project is occupied by a dental office. The site is currently vacant and covered by an AC parking lot or gravel from a recently demolished building. The site grades gently upward from south to north between elevations of 46 and 52 feet (Clark County GIS, 2018). Vegetation at the site is limited to mature trees around the boundary of the site.

4.3 SUBSURFACE CONDITIONS

4.3.1 General

GeoDesign supplemented the existing explorations at the site by excavating two test pits (TP-1 and TP-2) to depths between 7.0 and 7.5 feet BGS. The site was previously explored by others with four drilled borings to depths between 4.0 and 11.2 feet BGS. GeoDesign and others have also completed boring and test pit explorations adjacent to the site. The locations of the recent and previous explorations in the site vicinity are shown on Figure 2. Logs of the recent GeoDesign test pits and results of laboratory testing are presented in Appendix A. Previous explorations completed by GeoDesign and others are presented in Appendix B.

4.3.2 Soil Conditions

4.3.2.1 Undocumented Fill

While not directly observed in explorations completed at the site, we anticipate undocumented fill associated with demolition of the building in the southeast portion of site will be present. Typically, undocumented fill can be up to 3 feet thick, with the exception of drywells or other deeper subsurface elements.

4.3.2.2 Silt with Boulders and Gravel

Directly beneath the pavement section, surficial crushed rock, or undocumented fill is medium stiff to stiff silt with boulders, gravel, and sand. The soil matrix is brown and moist with boulders up to 3.5 feet in diameter. This soil unit extends to depths between approximately 3 and 5 feet BGS.

4.3.2.3 Gravel and Sand with Cobbles and Boulders

Medium dense to very dense gravel and sand with cobbles and boulders is present below the silt with boulders and gravel. The gravel is brown, moist, and contains minor silt. This soil unit extends to the maximum depth explored in explorations in the site vicinity.

4.3.3 Groundwater

An boring at the site by GeoEngineers encountered groundwater at a depth of 8.5 feet BGS. GeoDesign previously completed borings at the development southeast of site and did not encounter groundwater to the maximum depth explored of 21 feet BGS. Test pits were also excavated at the site to a depth of 13.5 feet BGS shortly after a record rainfall and did not encounter groundwater. We also reviewed two geotechnical reports by GE Services completed east of the site, and explorations that advanced to depths of 10.0 and 15.5 feet BGS did not encounter groundwater.

We anticipate that the groundwater observed in the GeoEngineers boring was perched and groundwater at the site will be 20 feet BGS or more.

4.4 GEOLOGIC HAZARDS

4.4.1 General

Site classes as defined in the IBC range from A to F, with E having the highest relative ground amplification. Site Class F requires a site-specific seismic study. Based on the results of our explorations, mapping on the Clark County GIS website, and experience in the area, Site Class C is appropriate for the site.

4.4.2 Liquefaction and Earthquake-Induced Settlement

Liquefaction is caused by a rapid increase in pore water pressure that reduces the effective stress between soil particles to near zero. Granular soil, which relies on interparticle friction for strength, is susceptible to liquefaction until the excess pore pressures can dissipate. In general, loose, saturated sand soil with low silt and clay content is the most susceptible to liquefaction. Silty soil with low plasticity is moderately susceptible to liquefaction under relatively higher levels of ground shaking.

According to the Alternative Liquefaction Susceptibility Map of Clark County by Palmer et al. (2004), the site is described as having a very low liquefaction susceptibility. Based on the results of explorations and published geologic information, we do not consider liquefaction a design consideration for the project.

4.4.3 Lateral Spreading

Lateral spreading is a liquefaction-related seismic hazard and occurs on gently sloping or flat sites underlain by liquefiable sediment adjacent to an open face, such as a riverbank. Liquefied soil adjacent to an open face can flow toward the open face, resulting in lateral ground displacement. There are no major open faces, and the liquefaction potential at the site is low. Accordingly, the potential for lateral spreading at the site is not a design consideration for the project.

4.4.4 Fault Rupture

Based on USGS mapping, the nearest mapped fault to the site is the Lacamas Lake fault, which is located approximately 1 mile to the west. As such, fault rupture is not considered a hazard at the site.

4.4.5 Landslides

Landslides and slope stability are not a concern at the site under current conditions. Geologic mapping completed by Fiksdal (1975) indicates the site vicinity is a "stable area."

4.5 INFILTRATION TESTING

Infiltration testing was completed using the open pit testing method in which water was poured into the base of the test pits and the drop in water was recorded with respect to time. Testing was completed until consistent rates were achieved. The unfactored results of the infiltration testing are presented in Table 1.

Table 1. Unfactored Infiltration Testing Results

Exploration	Depth (feet BGS)	Observed Infiltration Rate[1] (inches per hour)	Percent Passing U.S. Standard No. 200 Sieve
TP-1	5.5	11.0	13
TP-2	6.0	11.0	2
TP-2	7.0	30.0	5

The results of testing are similar to infiltration rates at adjacent sites. Recommendations for the use of the test results are provided in the "Stormwater Infiltration" section.

5.0 DESIGN

5.1 FOUNDATION SUPPORT

5.1.1 General

The proposed building will include a one-level, below-grade basement, and foundations will be approximately 15 feet below current grades at the site. The soil conditions at the anticipated foundation depth are dense to very dense gravel, and the building can be supported on conventional spread footings.

Based on explorations, large cobbles and boulders 3 feet or more are expected at the site. If boulders are present within footing locations, they should be completely removed and replaced with compacted crushed rock to smooth out surface irregularities.

5.1.2 Bearing Capacity

Footings should be proportioned on a maximum allowable bearing pressure of 4,000 psf. This value is a net bearing pressure; the weight of the footing and overlying backfill can be ignored in calculating footing sizes. The recommended allowable bearing pressure applies to the total of dead plus long-term live loads and can be increased by 50 percent for short-term loads resulting from wind or seismic forces. Continuous wall and isolated spread footings should be at least 18 and 24 inches wide, respectively. The bottom of exterior footings should be at least 18 inches below the lowest adjacent exterior grade. The bottom of interior footings should be established at least 12 inches below the base of the slab.

Total post-construction settlement is expected to be less than 1.0 inch. Differential settlement equal to 0.5 inch is possible between similarly loaded foundations.

5.1.3 Resistance to Sliding

Lateral loads on building and retaining wall footings can be resisted by passive earth pressure on the sides of the structures and by friction on the base of footings. The allowable passive earth pressure for footings confined by the on-site soil or planned structural fill is 350 pcf. Adjacent floor slabs, pavements, or the upper 12-inch depth of adjacent, unpaved areas should not be considered when calculating passive resistance.

An allowable coefficient of friction equal to 0.4 can be used for footings resting on native soil.

5.1.4 Subgrade Observation and Preparation

All footing subgrades should be evaluated by a representative of GeoDesign to confirm suitable bearing conditions. Localized over-excavation of footing subgrades may be required to penetrate deleterious material.

5.2 SEISMIC DESIGN CRITERIA

Based on explorations, the following design parameters can be applied if the building is designed using the applicable provisions the 2015 IBC. The parameters in Table 2 should be used to compute seismic base shear forces. We selected a Site Class C based on mapping by the Clark County GIS and the results of explorations.

Table 2. IBC Seismic Design Parameters

Parameter	Short Period (T_s = 0.2 second)	1 Second Period (T_1 = 1.0 second)
MCE Spectral Acceleration, S	S_s = 0.839 g	S_1 = 0.359 g
Site Class	C	
Site Coefficient, F	F_a = 1.064	F_v = 1.441
Adjusted Spectral Acceleration, S_M	S_{MS} = 0.893 g	S_{M1} = 0.517 g
Design Spectral Response Acceleration Parameters, S_D	S_{DS} = 0.595 g	S_{D1} = 0.345 g

5.3 FLOOR SLABS

We have assumed that the basement floor loads will be less than 200 psf and the subgrade consisting of dense to very dense gravel is prepared in accordance with the "Site Preparation" section. A modulus of reaction of 150 pci can be used for slabs on grade constructed on subgrade prepared as recommended in the "Site Preparation" section.

A minimum 6-inch-thick layer of imported granular material should be placed and compacted over the prepared subgrade to assist as a capillary break. The floor slab base rock should be crushed rock or crushed gravel and sand that meet the requirements outlined in the "Structural Fill" section. The imported granular material should be placed in one lift and compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D1557. Floor slab base rock contaminated with excessive fines (greater than 5 percent by dry weight passing the U.S. Standard No. 200 sieve) should be replaced.

Flooring manufacturers often require vapor barriers to protect flooring and flooring adhesives. Many flooring manufacturers will warrant their product only if a vapor barrier is installed according to their recommendations. Selection and design of an appropriate vapor barrier, if needed, should be based on discussions among members of the design team. We can provide additional information to assist you with your decision.

All slab subgrades should be evaluated by the geotechnical engineer to confirm suitable bearing conditions. Observations should also confirm that loose or soft material, organics, unsuitable fill, prior topsoil zones, and softened subgrades have been removed and replaced with structural fill.

5.4 SHORING

5.4.1 General

For the below-grade level where sloping is not an option, we recommend conventional soldier pile shoring with or without tieback anchors for support of the excavation. Where existing structures or sensitive utilities are adjacent to the excavation, tieback anchors are required to reduce lateral movements that will cause damage to the structures. Cantilever shoring might be possible to support the excavation where it is adjacent to the right-of-way if some settlement can be tolerated. Settlement for cantilevered shoring should be assumed to be approximately 1 inch at the shoring face and become negligible a horizontal distance of 10 feet from the shoring.

Cobbles and boulders will be present in the subsurface soil and can result in larger than expected soldier pile shafts. The soldier pile contractor should be prepared to advance through or modify their procedures when cobbles and boulders are encountered. We recommend that the owner include a contingency in their budget for encountering cobbles and boulders at the site.

5.4.2 Cantilever Shoring

Soldier pile shoring can be designed using the values presented on Figure 3. These values do not include surcharged-induced lateral earth pressures. Figure 4 should be used to compute surcharge-induced lateral earth pressures. We recommend a vertical live load of 250 psf be applied at the surface of the retained soil where the shoring retains roadways.

If the surface at the top of the shoring is sloped, the recommended lateral earth pressures should be increased as indicated in Table 3.

Table 3. Lateral Earth Pressure Increase Factors for Sloped Soil

Slope of Retained Soil (degrees)	Lateral Earth Pressure Increase Factor
0	1.00
5	1.06
10	1.12
20	1.33
25	1.52
30	2.27

5.4.3 Anchored Shoring

Anchored soldier pile shoring can be designed using the values presented on Figure 3. These values do not include surcharged-induced lateral earth pressures. Figure 4 should be used to compute surcharge-induced lateral earth pressures. We recommend a vertical live load of 250 psf be applied at the surface of the retained soil where the wall shoring retains roadways and the appropriate slope factors in Table 3 are applied if necessary.

Structural design of the soldier piles should consider the lateral earth pressures discussed above. In addition to lateral earth pressures, the soldier piles will be subject to compressive forces as a result of the downward component of the tieback anchor loads. We recommend the tips of soldier piles are embedded at least 10 feet below the base of the excavation and into the firm native soil. An allowable bearing pressure of 4,000 psf may be used for the base of the soldier piles resting on the dense soil. Skin friction along the sides of the solider piles will also be able to resist downward forces. An allowable skin friction of 2 ksf may be used for the native soil. The bonded zone for the tieback anchors should be maintained outside of the "unbonded zone for anchors" shown on Figure 3. We anticipate that the tieback anchors can achieve allowable bond strength of between 1.0 and 4.0 kips per foot, depending on the method of construction. A variety of methods are available for construction of tieback anchors. Therefore, we recommend that the contractor be responsible for selecting the appropriate bonded length and installation methods to achieve the required anchor capacity. Tieback anchors should be locked off at 100 percent of the design load.

Prior to installing production anchors, we recommend that performance testing be conducted on a minimum of two anchors. The purpose of this testing is to verify the installation procedure selected by the contractor before a large number of anchors are installed. We recommend that proof testing be conducted on all production anchors. Performance and proof testing should be performed in accordance with the guidelines provided in *Recommendations for Prestressed Rock and Soil Anchors* (Post Tensioning Institute, 2014).

We anticipate that wood lagging will be used to span between the soldier piles. To maintain the integrity of the excavation, prompt and careful installation of lagging, particularly in areas of seepage and loose soil, is recommended. All voids behind the lagging should be completely backfilled with grout slurry.

5.5 RETAINING STRUCTURES

5.5.1 Basement Walls

Embedded walls that are braced more than one level should be designed using lateral earth pressures shown on Figure 3. These values do not include surcharged-induced lateral earth pressures. The values on Figure 4 can be used to compute surcharge-induced lateral earth pressures. Seismic earth pressures on retaining walls can be determined using a seismic earth pressure of 8H psf per foot length of the wall (where H is the wall height). This seismic pressure should be applied as a uniform rectangular pressure over the wall height.

5.5.2 Conventional Retaining Walls

Retaining walls can be designed using conventional Rankine theory. Design recommendations for retaining walls are based on the following assumptions: (1) the walls are not in contact with temporary shoring, (2) the walls are less than 10 feet in height, (3) the retained soil is level, and (4) drainage is provided behind the walls to prevent hydrostatic pressures from developing. Re-evaluation of our recommendations will be required if the retaining wall design criteria for the project vary from these assumptions.

Walls not restrained from rotation can be designed assuming an equivalent fluid pressure of 35 pcf. This should be increased to 55 pcf for design of walls restrained from rotation.

Lateral forces can be resisted by frictional resistance on wall foundations and by passive earth pressures between the wall and the soil in front of the wall. A frictional coefficient of 0.35 can be used between the foundation and subgrade soil. An equivalent fluid pressure equal to 350 pcf can be used to compute the available passive resistance. Adjacent floor slabs, pavements, or the upper 12 inches of adjacent areas should not be considered when calculating passive resistance. This value assumes that groundwater is below the base of the wall footing. Allowable bearing pressures for wall subgrades prepared as described in the "Site Preparation" section can be designed for an allowable bearing pressure of 3,000 psf. These values do not include surcharged-induced earth pressures. The values on Figure 4 can be used to compute surcharge-induced lateral earth pressures.

Seismic earth pressures on retaining walls can be determined using seismic earth pressures of 8H psf per foot length of the wall (where H is the wall height). This seismic pressure should be applied as a uniform rectangular pressure over the wall height.

5.5.3 Retaining Wall Backfill

Backfill should be placed and compacted as recommended for structural fill, with the exception of backfill placed immediately adjacent to walls. Backfill adjacent to walls should be compacted to a lesser standard to reduce the potential for generation of excessive pressure on the walls. Backfill located within a horizontal distance of 3 feet from the walls should be compacted to approximately 90 percent of the maximum dry density, as determined by ASTM D1557. Backfill placed within 3 feet of the wall should be compacted in lifts less than 6 inches thick using hand-operated tamping equipment (such as a jumping jack or vibratory plate compactor). If flatwork (slabs, sidewalk, or pavement) will be placed adjacent to the wall, we recommend the upper 2 feet of fill be compacted to 95 percent of the maximum dry density, as determined by ASTM D1557. Settlement of up to 1 percent of the wall height commonly occurs immediately adjacent to the wall as the wall rotates and develops active lateral earth pressures. Consequently, we recommend that construction of flatwork adjacent to retaining walls be postponed at least four weeks after construction, unless survey data indicates that settlement is complete prior to that time.

5.6 DRAINAGE

5.6.1 Temporary

During work at the site, the contractor should be made responsible for temporary drainage of surface water as necessary to prevent standing water and/or erosion at the working surface. During rough and finished grading of the site, the contractor should keep all pads and subgrade free of ponding water.

5.6.2 Surface

The ground surface at finished pads should be sloped away from their edges at a minimum 2 percent gradient for a distance of at least 5 feet. Roof drainage from the building should be directed into solid, smooth-walled drainage pipes that carry the collected water to the storm drain system. Trapped planter areas should not be created adjacent to roadways and structures without providing means for positive drainage (e.g., swales or catch basins).

5.6.3 Subsurface

Due to the proposed depth of the infiltration system north of the building and infiltration systems that may be present in the city rights-of-way, there is the potential water will be injected above the basement grades. Accordingly, there is some risk of water in the basement if the appropriate systems are not installed. We recommend providing drainage and waterproofing along the backs of the embedded walls. Drainage should include a minimum 6-inch-diameter, perforated collector pipe be installed at least 2 feet below the finished basement slab grade. The pipe should be embedded in a minimum 2-foot-wide zone of angular drain rock that extends up the back of the wall to within 1 foot of the finished grade. The drain rock should meet the specifications provided in WSS 9-03.12(4) – Gravel Backfill for Drains. The drain rock should be wrapped in a geotextile fabric that meets the specifications provided in WSS 9-33.2 – Geosynthetic Properties for drainage geotextiles. The perforated collector pipes should discharge at an appropriate location away from the base of the wall. The discharge pipe(s) should not be tied directly into stormwater drain systems, unless measures are taken to prevent backflow into the wall's drainage system.

5.6.4 Stormwater Infiltration

Stormwater will be infiltrated on site via shallow systems or drywells. The unfactored results of our infiltration and laboratory testing are presented in the "Infiltration Testing" section and Table 1.

Based on the results of infiltration testing, both shallow systems and drywells are feasible at the site. We recommend using an unfactored design rate of 11 inches per hour for native soil more than 5 feet BGS. As outlined in Table 6-1 of the Clark County Stormwater Manual, a soil correction factor of 1.5 should be applied to the rates described. This factor of safety is for geotechnical variability, and additional factors of safety may be required.

We recommend that GeoDesign review the design and placement of the proposed systems relative to the appropriateness of the referenced design rates used by the civil engineer. GeoDesign should be on site during installation of infiltration systems to verify the soil conditions are consistent with the design. In addition, GeoDesign should complete confirmation testing of the infiltration systems during construction to verify that infiltration rates meet the design. Furthermore, we recommend that a contingency be in place to increase the depth or enlarge the infiltration systems during construction if tested rates at the time of construction are unsuitable.

5.7 PERMANENT SLOPES

While not anticipated, permanent cut and fill slopes should not exceed 2H:1V. Upslope roads and pavements should be located at least 5 feet from the top of cut and fill slopes. The setback should be increased to 10 feet for buildings. The slopes should be planted with appropriate vegetation to provide protection against erosion as soon as possible after grading. Surface water runoff should be collected and directed away from slopes to prevent water from running down the face of the slope.

5.8 PAVEMENTS

Pavements should be installed on firm native subgrade or structural fill subgrade prepared in conformance with the "Site Preparation" and "Materials" sections. We anticipate that vehicle traffic will be primarily limited to passenger cars and light trucks.

5.8.1 Design Values

Our pavement recommendations are based on the following assumptions:

- We have assumed a resilient modulus value of 4,000 psi for native and structural fill subgrades prepared as indicated it the "Site Preparation" section.
- A pavement design life of 20 years.
- Initial and terminal serviceability indices of 4.2 and 2.0, respectively.
- Reliability of 85 percent and standard deviation of 0.45.
- No growth.

If any of these assumptions are incorrect, our office should be contacted with the appropriate information so that the pavement designs can be revised.

5.8.2 Recommended Design Sections

Our pavement design recommendations for the assumptions and loads provided above are summarized in Table 4.

Table 4. Recommended Standard Pavement Sections

Pavement Use	AC Thickness[1] (inches)	Aggregate Base Thickness[1] (inches)
Drive Aisles	3.5	8.0
Automobile Parking	2.5	6.0

1. All thicknesses are intended to be the minimum acceptable values.

The material thicknesses shown in Table 4 are intended to be minimum acceptable values for the final condition. The aggregate base thickness does not account for construction traffic, and haul roads and staging areas should be used as described in the "Construction" section.

6.0 CONSTRUCTION

6.1 SITE PREPARATION

6.1.1 Demolition

Demolition includes complete removal of existing improvements within areas to receive new buildings, engineered fill, or pavements. Demolished material should be transported off site for disposal. Excavations remaining from removing basements (if present), foundations, utilities, and other subsurface elements should be backfilled with structural fill where these are below planned site grades. The base of the excavations should be excavated to expose firm subgrade before filling. The sides of the excavations should be cut into firm material and sloped a minimum of 1½H:1V. Utility lines abandoned under new structural components should be completely removed and backfilled with structural fill. Soft or disturbed soil encountered during demolition should be removed and replaced with structural fill.

6.1.2 Stripping and Clearing

The site is predominately covered by gravel and AC, and stripping will be minimal at the site. Trees and their root balls should be grubbed out to the depth of the roots, which could exceed 3 feet BGS. Depending on the methods used to remove root balls, considerable disturbance and loosening of the subgrade could occur during site grubbing. We recommend that soil disturbed during grubbing operations be removed to expose firm, undisturbed subgrade. The resulting excavations should be backfilled with compacted structural fill.

6.1.3 Undocumented Fill

We anticipate that undocumented fill is present in the southeast portion of the site. Based on the proposed development plan, the fill will likely be in the building footprint and will be removed as part of the basement excavation.

If fill is located outside the building footprint, we recommend that it be identified immediately after demolition and evaluated as described in the "Subgrade Evaluation" section. If portions of the subgrade are determined to be unsuitable, they should be removed and replaced with granular fill as described in the "Structural Fill" section.

6.1.4 Subgrade Evaluation

Upon completion of stripping and prior to the placement of any structural fill or pavement, the exposed subgrade should be evaluated by proof rolling to identify soft, loose, or unsuitable areas. Proof rolling should be conducted with a fully loaded dump truck or similar heavy, rubber tire construction equipment. Qualified personnel should observe proof rolling to evaluate yielding of the ground surface. The subgrade should be evaluated by probing with a foundation probe when the subgrade is too wet. If soft or yielding subgrade is identified, the subgrade should be excavated and replaced with structural fill.

6.2 CONSTRUCTION CONSIDERATIONS

6.2.1 Near Surface Soil

The near-surface soil present on this site contains silt and is easily disturbed. If not carefully executed, site preparation, utility trench work, and excavation can create extensive soft areas and significant repair costs can result. Earthwork planning, regardless of the time of year, should include considerations for minimizing subgrade disturbance.

If construction occurs during or extends into the wet season, or if the moisture content of the surficial soil is more than a couple percentage points above optimum, site stripping and cutting may need to be accomplished using track-mounted equipment. Likewise, the use of granular haul roads and staging areas will be necessary for support of construction traffic during the rainy season or when the moisture content of the surficial soil is more than a few percentage points above optimum. The amount of staging and haul road areas, as well as the required thickness of granular material, will vary with the contractor's sequencing of a project and type/frequency of construction equipment. Based on our experience, between 12 and 18 inches of imported granular material is generally required in staging areas and between 18 and 24 inches in haul roads areas. Stabilization material may be used as a substitute, provided the top 4 inches of material consists of imported granular material. The actual thickness will depend on the contractor's means and methods and, accordingly, should be the contractor's responsibility. In addition, a geotextile separation fabric should be placed as a barrier between the subgrade and imported granular material in areas of repeated construction traffic. The imported granular material, stabilization material, and geotextile fabric should meet the specifications in the "Materials" section.

6.2.2 Basement Subgrade

Based on the proposed development configuration, the soil at the bottom of the basement excavation will likely consist of dense to very dense gravel. The gravel will be capable of supporting some construction equipment; however, a granular working blanket consisting of filter fabric overlain by imported granular material may be needed to support construction activities and prevent subgrade damage. Based on our experience, a 12-inch-thick working blanket will likely be required to support heavy equipment operating within the basement excavation; however, the actual thickness of the working blanket should be selected by the contractor based on the anticipated construction traffic volumes and loads.

The imported granular material for working blankets, haul roads, and staging areas should consist of crushed rock that is well graded and has less than 8 percent by dry weight passing the U.S. Standard No. 200 sieve. In areas where silt is exposed at the ground surface, a geotextile should be placed below the granular material. The geotextile should have a minimum Mullen burst strength of 250 psi for puncture resistance and an AOS between U.S. Standard No. 70 and No. 100 sieves.

6.3 TEMPORARY SLOPES

Temporary slopes less than 15 feet high should be no steeper than 1½H:1V, provided groundwater seepage does not occur. If slopes greater than 10 feet high are required, GeoDesign should be contacted to make additional recommendations. We recommend a minimum horizontal distance of 5 feet from the edge of the existing improvements to the top of the temporary slope. All cut slopes should be protected from erosion by covering them during wet weather. If sloughing or instability is observed, the slope should be flattened or supported by shoring. Excavations should not undermine adjacent utilities, foundations, walkways, streets, or other hardscapes unless special shoring or underpinned support is provided.

6.4 EROSION CONTROL

The on-site soil is susceptible to erosion. Consequently, we recommend that slopes be covered with an appropriate erosion control product if construction occurs during periods of wet weather. We recommend that all slope surfaces be planted as soon as practical to minimize erosion. Surface water runoff should be collected and directed away from slopes to prevent water from running down the slope face. Erosion control measures, such as straw bales, sediment fences, and temporary detention and settling basins, should be used in accordance with local and state ordinances.

6.5 EXCAVATION

Cobbles and boulders up to 3.5 feet in diameter are anticipated at the site. When encountered, cobbles and especially boulders will result in difficult excavations and may require special equipment and procedures for removal. If difficult excavations are encountered, trenches may also be wider than anticipated, increasing the amount of backfill material required. The earthwork contractor should be prepared to use open excavation techniques or approved temporary shoring when excavating on site soil. A variety of shoring systems are available; consequently, we recommend that the contractor be responsible for selecting the appropriate system.

In lieu of large and open cuts, approved temporary shoring may be used for excavation support. A variety of shoring systems are available; consequently, we recommend the contractor be responsible for selecting the appropriate system.

Excavations should be made in accordance with applicable OSHA and state regulations. While this report describes certain approaches to excavation, the contractor should be responsible for selecting excavation methods, dewatering, monitoring the excavations for safety, and providing shoring, as required to protect personnel and adjacent utilities and structures.

6.6 MATERIALS

6.6.1 Structural Fill

Fills should only be placed over subgrade that has been prepared in conformance with the "Site Preparation" section. A variety of material may be used as structural fill at the site. However, all material used as structural fill should be free of organic matter or other unsuitable material and should meet the specifications provided in WSS 9-03 – Aggregates, depending on the application. A brief characterization of some of the acceptable materials and our recommendations for their use as structural fill are provided below.

6.6.1.1 On-Site Soil

The on-site soil is suitable for structural fill, provided it is free of organic matter and unsuitable material. Based on the moisture content of the on-site soil, we anticipate moisture conditioning, including drying, mixing, and addition of water, may be required to use the on-site soil for structural fill during certain times of the year. Accordingly, extended dry weather and sufficient area to dry the soil will be required to adequately condition the soil for use as structural fill. The on-site fine-grained soil should not be used as structural fill during the wet season.

When used as structural fill, the on-site fine-grained soil should be placed in lifts with a maximum uncompacted thickness of 8 inches and compacted to not less than 92 percent of the maximum dry density, as determined by ASTM D1557.

6.6.1.2 Imported Granular Material

Imported granular material used during periods of wet weather, for building pad subgrades, and for staging areas should be pit- or quarry-run rock, crushed rock, or crushed gravel and sand and should meet the specifications provided in WSS 9-03.9(1) – Ballast, WSS 9-03.14(1) – Gravel Borrow, or WSS 9-03.14(2) – Select Borrow. The imported granular material should be fairly well graded between coarse and fine material, should have less than 5 percent by dry weight passing the U.S. Standard No. 200 sieve, and should have a minimum of two mechanically fractured faces.

Imported granular material should be placed in lifts with a maximum uncompacted thickness of 8 to 12 inches and compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D1557. During the wet season or when wet subgrade conditions exist, the initial lift should be approximately 18 inches in uncompacted thickness and compacted with a smooth-drum roller without using vibratory action.

Where imported granular material is placed over wet or soft soil subgrades, we recommend a geotextile be placed as a barrier between the subgrade and imported granular material. Depending on site conditions, the geotextile should meet the specifications provided in WSS 9-33.2(1) – Geotextile Properties (Table 3) for soil separation or stabilization. The geotextile should be installed in conformance with WSS 2-12 – Construction Geosynthetic.

6.6.1.3 Stabilization Material

Stabilization material used to create haul roads for construction traffic or at the base of unstable trenches should consist of pit- or quarry-run rock or crushed rock. The material should have a maximum particle size of 6 inches and less than 5 percent by dry weight passing the U.S. Standard No. 4 sieve, should have at least two mechanically fractured faces, and should be free of organic matter and other deleterious material. Material meeting the specifications provided in WSS 9-27.3(6) – Stone is generally acceptable for use. Stabilization material should be placed in lifts between 12 and 18 inches thick and compacted to a firm condition with a smooth-drum roller without using vibratory action.

Where the stabilization material is used to stabilize soft subgrade beneath pavements or construction haul roads, a geotextile should be placed as a barrier between the soil subgrade and the imported granular material. Geotextile is not required where stabilization material is used at the base of utility trenches.

6.6.1.4 Trench Backfill

Trench backfill placed beneath, adjacent to, and for at least 2 feet above utility lines (i.e., the pipe zone) should consist of well-graded, granular material with a maximum particle size of 1½ inches and less than 7 percent by dry weight passing the U.S. Standard No. 200 sieve and should meet the specifications provided in WSS 9-03.12(3) – Gravel Backfill for Pipe Zone Bedding. The pipe zone backfill should be compacted to at least 90 percent of the maximum dry density, as determined by ASTM D1557, or as required by the pipe manufacturer or local building department.

Within roadway alignments or beneath proposed or future building pads, the remainder of the trench backfill should consist of well-graded, granular material with a maximum particle size of 2½ inches and less than 7 percent by dry weight passing the U.S. Standard No. 200 sieve and should meet the specifications provided in WSS 9-03.19 – Bank Run Gravel for Trench Backfill. This material should be compacted to at least 92 percent of the maximum dry density, as determined by ASTM D1557, or as required by the pipe manufacturer or local building department. The upper 2 feet of the trench backfill should be compacted to at least 95 percent of the maximum dry density, as determined by ASTM D1557. Outside of structural improvement areas (e.g., roadway alignments or building pads), trench backfill placed above the pipe zone may consist of general fill material that is free of organics and material over 6 inches in size and meets the specifications provided in WSS 9-03.14(3) – Common Borrow and WSS 9-03.15 – Native

Material for Trench Backfill. This general trench backfill should be compacted to at least 90 percent of the maximum dry density, as determined by ASTM D1557, or as required by the pipe manufacturer or local building department.

6.6.1.5 Aggregate Base Rock

Imported granular material placed beneath pavements and floor slabs should be clean, crushed rock or crushed gravel and sand that are fairly well graded between coarse and fine. The granular material should contain no deleterious material, should have a maximum particle size of 1½ inches, should meet the specifications provided in WSS 9-03.9(3) – Crushed Surfacing and WSS 9-03.10 – Aggregate for Gravel Base, should have less than 5 percent by dry weight passing the U.S. Standard No. 200 sieve, and should have a minimum of two mechanically fractured faces. The imported granular material should be placed in one lift and compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D1557.

6.6.1.6 Retaining Wall Select Backfill

Backfill material placed behind retaining walls and extending a horizontal distance of ½H (where H is the height of the retaining wall) should consist of select granular material that meets the requirements provided in WSS 9-03.12(2) – Gravel Backfill for Walls. We recommend the select granular wall backfill be separated from general fill, native soil, and/or topsoil using a geotextile fabric that meets the specifications provided below for drainage geotextiles.

The wall backfill should be compacted to a minimum of 95 percent of the maximum dry density, as determined by ASTM D1557. However, backfill located within a horizontal distance of 3 feet from a retaining wall should only be compacted to approximately 90 percent of the maximum dry density, as determined by ASTM D1557. Backfill placed within 3 feet of walls should be compacted in lifts less than 6 inches thick using hand-operated tamping equipment (such as a jumping jack or vibratory plate compactor). If flatwork (sidewalks or pavements) will be placed atop the wall backfill, we recommend the upper 2 feet of material be compacted to 95 percent of the maximum dry density, as determined by ASTM D1557.

6.6.2 Geotextile Separation Fabric

A geotextile separation fabric will be required at the interface of the existing soil and imported granular material beneath the proposed walls. In addition, geotextile fabric may be required where soft subgrade is encountered. The separation fabric should meet the specifications provided in WSS 9-33.2(1) – Geotextile Properties (Table 3) for soil separation. The geotextile should be installed in conformance the specifications provided in WSS 2-12 – Construction Geosynthetic.

6.6.3 AC

6.6.3.1 General

The AC pavement should conform to WSS 5-04 - Hot Mix Asphalt. AC should consist of ½-inch HMA. The asphalt cement binder should be PG 64-22 Performance Grade Asphalt Cement conforming to WSS 9-02.1(4) – Performance Graded Asphalt Binder. The layer thickness should be 2.0 to 3.5 inches. The job mix formula should meet the requirements for non-statistical

½-inch HMA (WSS 5-04 – Hot Mix Asphalt and WSS 9-03.8 – Aggregates for Hot Mix Asphalt) and be compacted to 91 percent of the maximum specific gravity or as required by the local jurisdiction in public right-of-way areas.

6.6.3.2 Cold Weather Paving Considerations

In general, AC paving is not recommended during cold weather (temperatures less than 40 degrees Fahrenheit). Compacting under these conditions can result in low compaction and premature pavement distress. Each AC mix design has a recommended compaction temperature range that is specific for the particular AC binder used. In colder temperatures, it is more difficult to maintain the temperature of the AC mix as it can lose heat while stored in the delivery truck, as it is placed, and in the time between placement and compaction. The AC surface temperature during paving should be at least 40 degrees Fahrenheit for lift thickness greater than 2.5 inches and at least 50 degrees Fahrenheit for lift thickness between 2.0 and 2.5 inches.

If paving activities must take place during cold-weather construction as defined above, the project team should be consulted and a site meeting should be held to discuss ways to lessen low compaction risks.

7.0 OBSERVATION OF CONSTRUCTION

Satisfactory earthwork and foundation performance depends to a large degree on the quality of construction. Subsurface conditions observed during construction should be compared with those encountered during the subsurface explorations. Recognition of changed conditions often requires experience; therefore, qualified personnel should visit the site with sufficient frequency to detect whether subsurface conditions change significantly from those anticipated. In addition, sufficient observation of the contractor's activities is a key part of determining that the work is completed in accordance with the construction drawings and specifications.

8.0 LIMITATIONS

We have prepared this report for use by Lone Wolf Development, LLC. and members of the design and construction team for the proposed development. The data and report can be used for estimating purposes, but our report, conclusions, and interpretations should not be construed as a warranty of the subsurface conditions and are not applicable to other sites.

Soil explorations indicate soil conditions only at specific locations and only to the depths penetrated. They do not necessarily reflect soil strata or water level variations that may exist between exploration locations. If subsurface conditions differing from those described are noted during the course of excavation and construction, re-evaluation will be necessary.

The site development plans and design details were not finalized at the time this report was prepared. When the design has been finalized and if there are changes in the site grades or location, configuration, design loads, or type of construction for the building, the conclusions and recommendations presented may not be applicable. If design changes are made, we should be retained to review our conclusions and recommendations and to provide a written evaluation or modification.

The scope of our services does not include services related to construction safety precautions, and our recommendations are not intended to direct the contractor's methods, techniques, sequences or procedures, except as specifically described in our report for consideration in design.

Within the limitations of scope, schedule, and budget, our services have been executed in accordance with the generally accepted practices in this area at the time this report was prepared. No warranty or other conditions, express or implied, should be understood.

♦ ♦ ♦

We appreciate the opportunity to be of service to you. Please call if you have questions concerning this report or if we can provide additional services.

Sincerely,

GeoDesign, Inc.

Nick Paveglio, P.E.
Senior Associate Engineer

Jeffery D. Tucker, P.E.
Principal Engineer

NICHOLAS N. PAVEGLIO
STATE OF WASHINGTON
45872
REGISTERED
PROFESSIONAL ENGINEER

Signed 07/20/2018

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FIGURES

VICINITY MAP BASED ON AERIAL PHOTOGRAPH OBTAINED FROM GOOGLE EARTH PRO®

GEODESIGN INC 703 Broadway Street - Suite 650 Vancouver WA 98660 360.693.8416 www.geodesigninc.com	LONEWOLF-7-01	**VICINITY MAP**	
	JULY 2018	BLOCK 8 WASHOUGAL, WA	**FIGURE 1**

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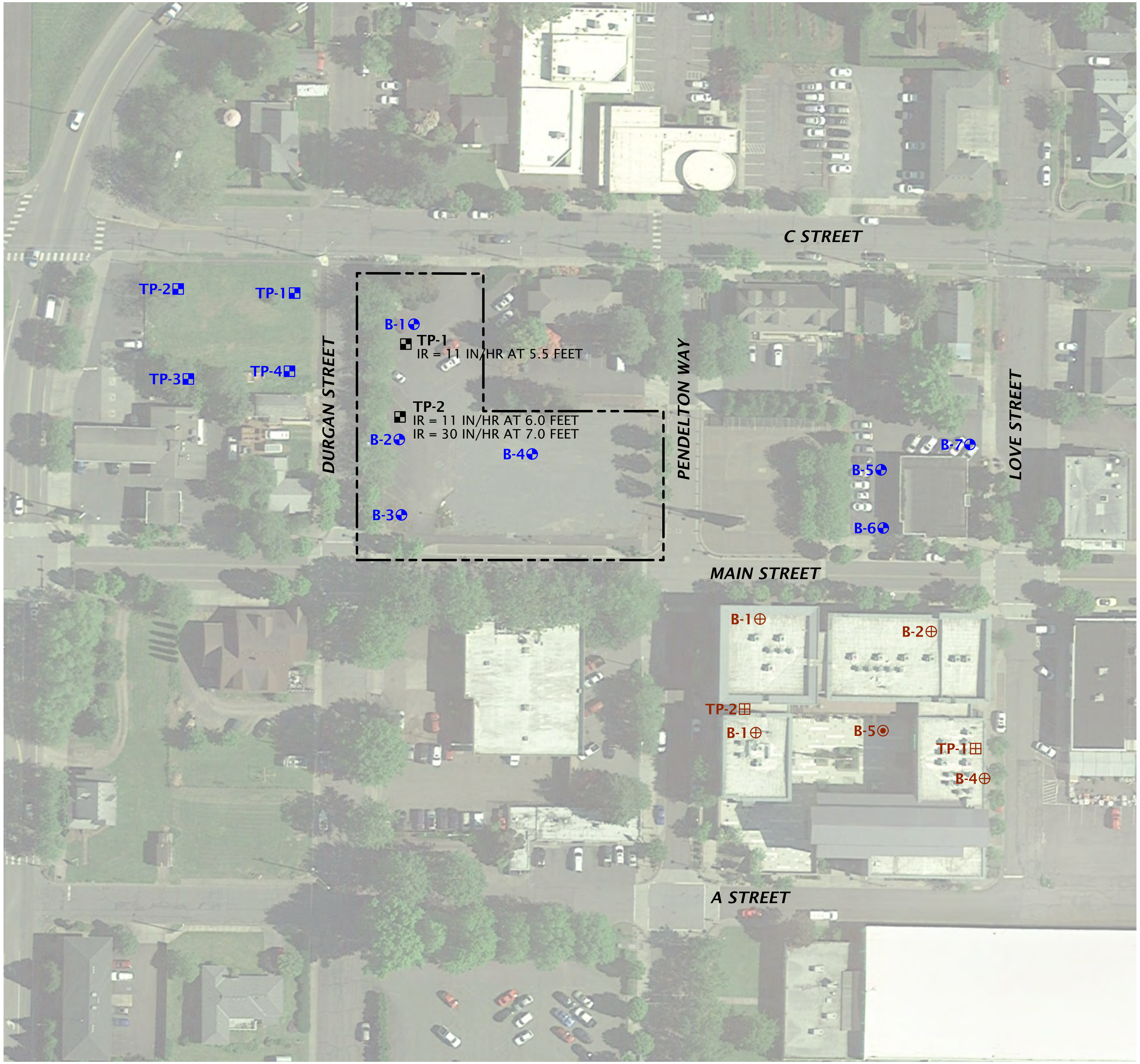
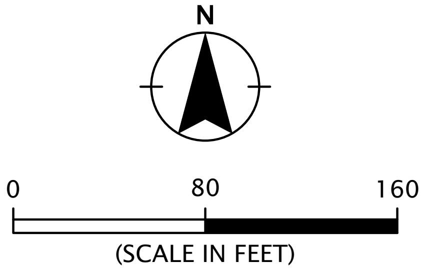
File Name: J:\E-L\LoneWolf\LoneWolf-7\LoneWolf-7-01\Figures\CAD\LoneWolf-7-01-VM01.dwg | Layout: FIGURE 1



LEGEND:

TP-1 ◩	TEST PIT (GEODESIGN, 2018)
B-1 ◕	BORING (GEOENGINEERS, 2007)
TP-1 ◩	TEST PIT (GEOENGINEERS, 2007
B-5 ◉	BORING (GEODESIGN, 2006)
TP-1 ⊞	TEST PIT (GEODESIGN, 2006)
B-1 ⊕	BORING (GEODESIGN, 2005)
— — —	SITE BOUNDARY
IR = 11 IN/HR AT 5.5 FEET	UNFACTORED INFILTRATION RATE



SITE PLAN BASED ON AERIAL PHOTOGRAPH
OBTAINED FROM GOOGLE EARTH PRO®,
MAY 31, 2018

FIGURE 2

SITE PLAN

BLOCK 8
WASHOUGAL, WA

LONEWOLF-7-01

JULY 2018

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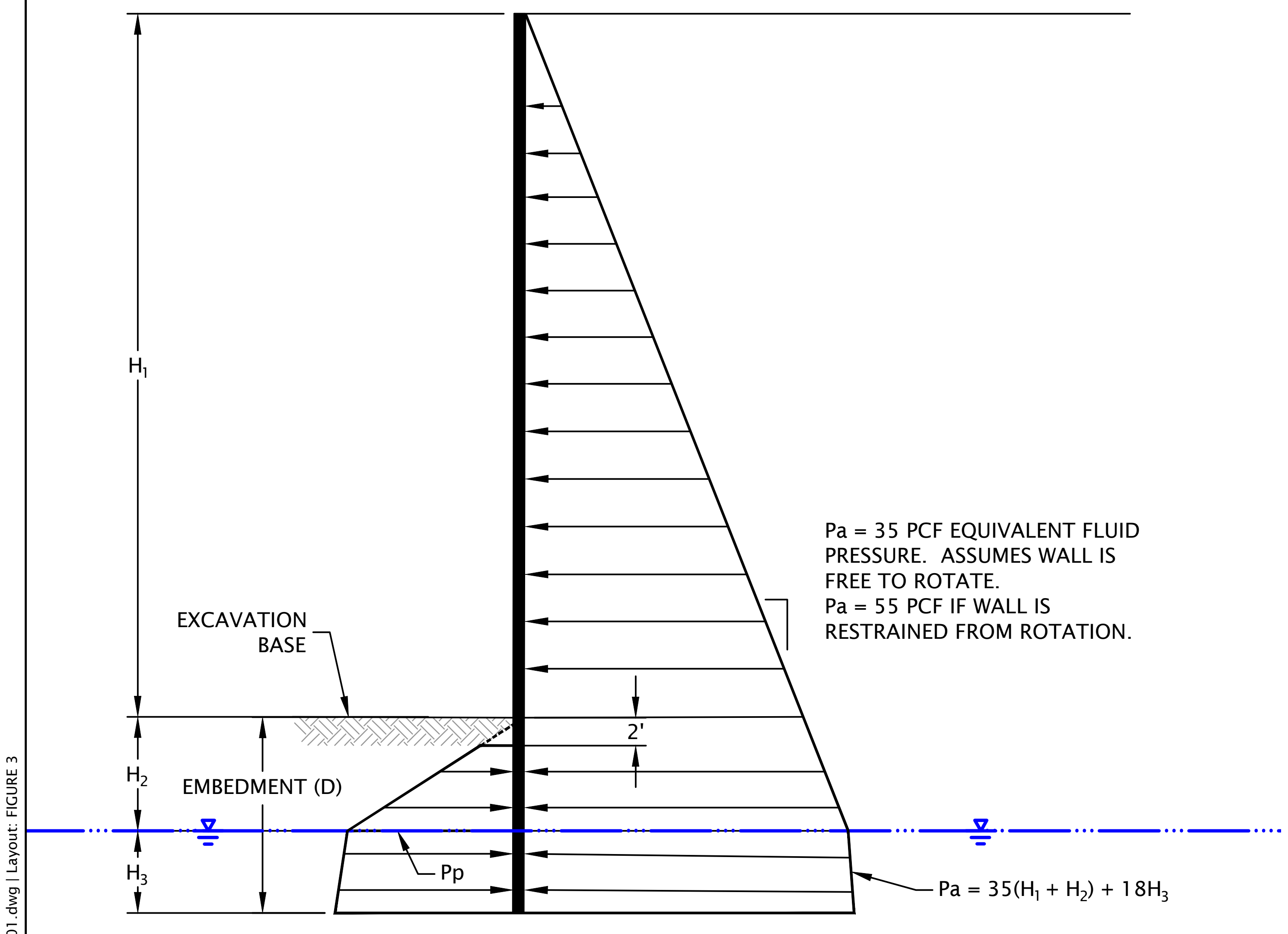
RECOMMENDED DESIGN PARAMETERS FOR CANTILEVERED WALL



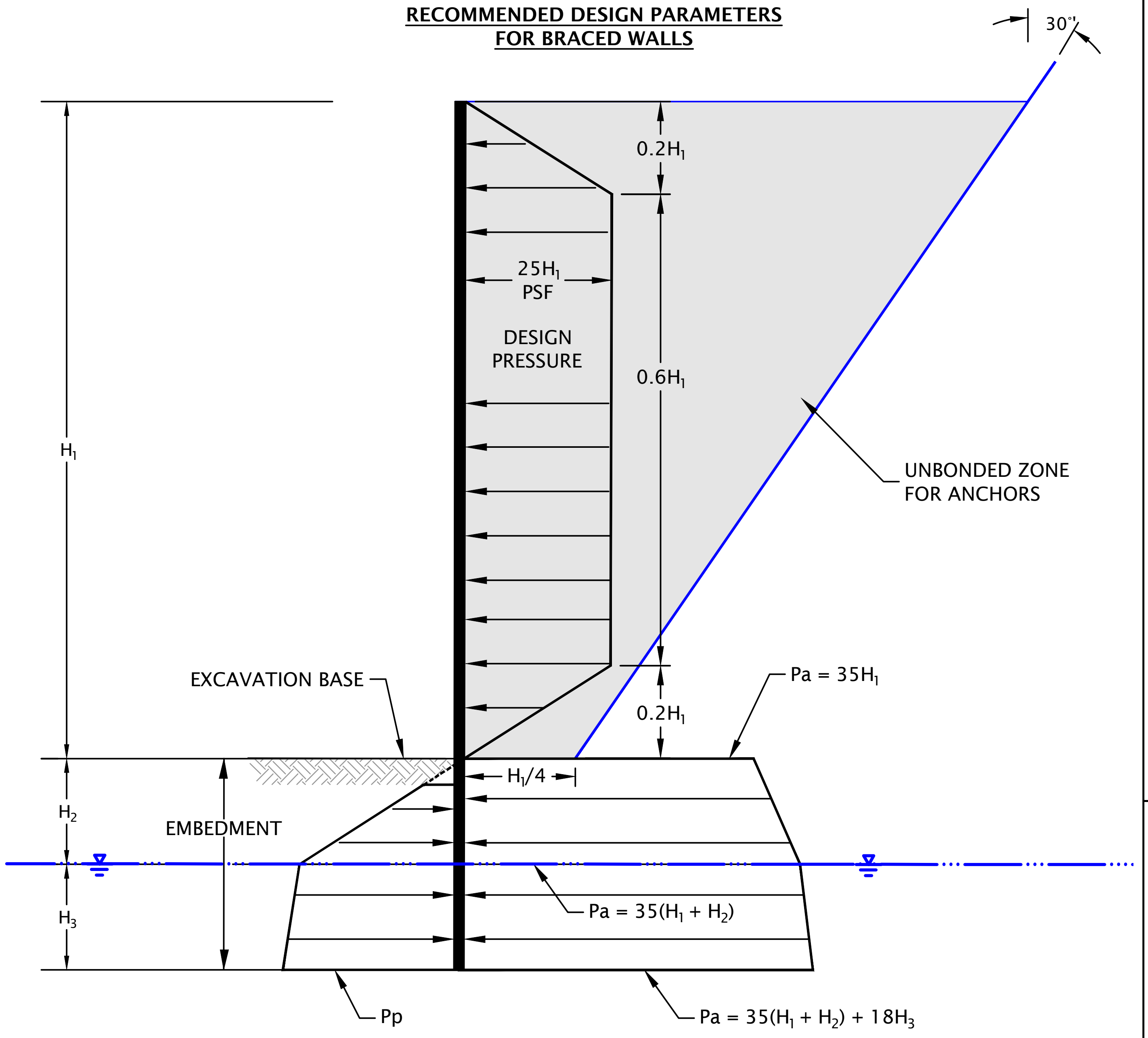
EXPLANATION:

Pp = 400 H_2 PCF FOR SILT FILL AND NATIVE SOILS ABOVE WATER TABLE
Pp = 200 H_3 PCF FOR SILT FILL AND NATIVE SOILS BELOW WATER TABLE
H_1 = HEIGHT OF SOLDIER PILE EXPOSED HEIGHT IN FEET
PASSIVE PRESSURE ACTS OVER 2.5X THE PILE WIDTH
ACTIVE PRESSURE ACTS OVER 1X THE PILE WIDTH BELOW EXCAVATION BASE

NOTES:

1. FIGURE DOES NOT INCLUDE SURCHARGE OR SEISMIC LOADS.
2. LATERAL EARTH PRESSURE ASSUMES WATER WILL BE MAINTAINED BELOW THE BASE OF THE EXCAVATION.
3. THE LATERAL EARTH PRESSURES ARE UNFACTORED.
4. PASSIVE PRESSURE RESISTANCE SHOULD BE NEGLECTED 2 FEET BELOW THE BOTTOM OF THE EXCAVATION.
5. REFER TO THE GEOTECHNICAL REPORT FOR APPROPRIATE GROUNDWATER ELEVATION.
6. GROUNDWATER IS AT AN ELEVATION OF 30 FEET.

RECOMMENDED DESIGN PARAMETERS FOR BRACED WALLS



EXPLANATION:

Pp = 400 H_2 PCF FOR SILT FILL AND NATIVE SOILS ABOVE WATER TABLE
Pp = 200 H_3 PCF FOR SILT FILL AND NATIVE SOILS BELOW WATER TABLE
H_1 = HEIGHT OF SOLDIER PILE EXPOSED HEIGHT IN FEET
PASSIVE PRESSURE ACTS OVER 2X THE PILE WIDTH
ACTIVE PRESSURE ACTS OVER 1X THE PILE WIDTH BELOW EXCAVATION BASE

NOTES:

1. DOES NOT INCLUDE SURCHARGE OR SEISMIC LOADS.
2. TIEBACKS SHOULD BE LOCKED OFF AT 100 PERCENT OF DESIGN LOAD.
3. THE LATERAL EARTH PRESSURES ARE UNFACTORED.
4. PASSIVE PRESSURE RESISTANCE SHOULD BE NEGLECTED 2 FEET BELOW THE BOTTOM OF THE EXCAVATION.
5. REFER TO THE GEOTECHNICAL REPORT FOR APPROPRIATE GROUNDWATER ELEVATION.
6. GROUNDWATER ELEVATION IS AT AN ELEVATION OF 30 FEET.

CANTILEVERED AND BRACED WALLS DESIGN CRITERIA

FIGURE 3

BLOCK 8
WASHOUGAL, WA

LONEWOLF-7-01

JULY 2018

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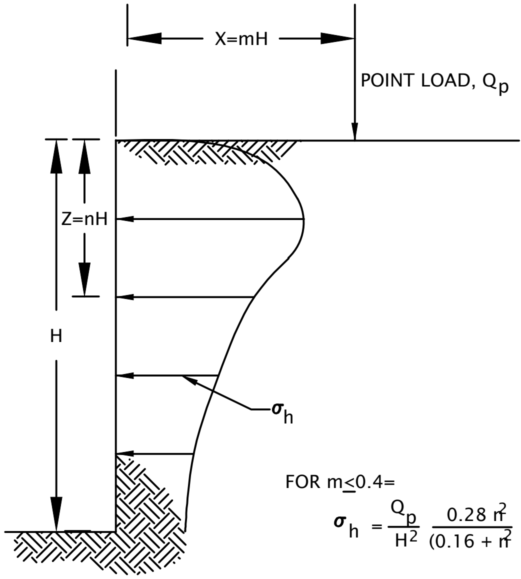
File Name: J:\E-L\LoneWolf\LoneWolf-7\LoneWolf-7-01\Figures\CAD\LoneWolf-7-01-DET01.dwg | Layout: FIGURE 3

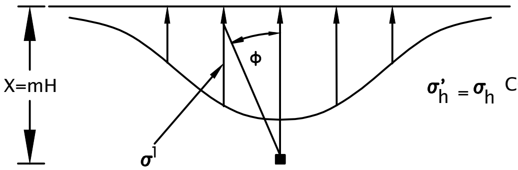


FOR $m \leq 0.4$=

$$\sigma_h = \frac{Q_p}{H^2} \frac{0.28 n^2}{(0.16 + n^2)^3}$$

FOR $m > 0.4$=

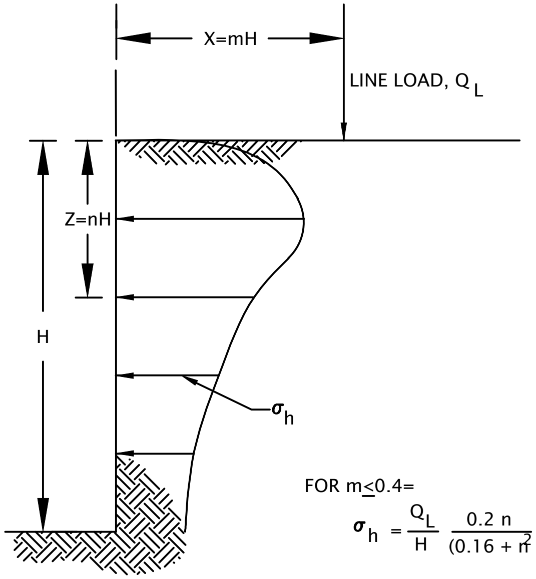
$$\sigma_h = \frac{Q_p}{H^2} \frac{1.77 m^2 n^2}{(m^2 + n^2)^3}$$



$$\sigma'_h = \sigma_h \ \mathrm{COS}^2 (1.1\phi)$$

DISTRIBUTION OF HORIZONTAL PRESSURES

VERTICAL POINT LOAD



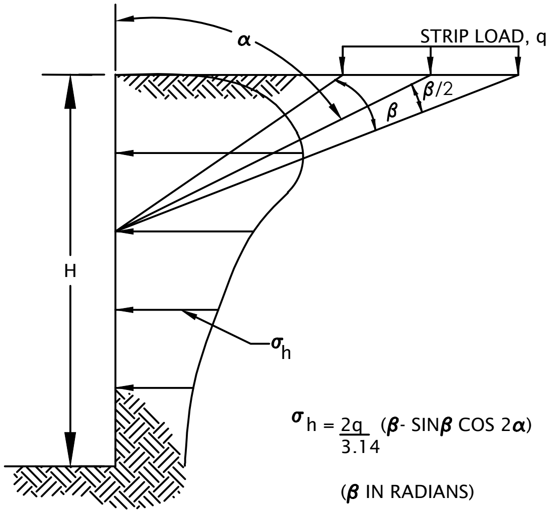
FOR $m \leq 0.4$=

$$\sigma_h = \frac{Q_L}{H} \frac{0.2 n}{(0.16 + n^2)^2}$$

FOR $m > 0.4$=

$$\sigma_h = \frac{Q_L}{H} \frac{1.28 m^2 n}{(m^2 + n^2)^2}$$

LINE LOAD PARALLEL TO WALL



$$\sigma_h = \frac{2q}{3.14} (\beta - \mathrm{SIN}\beta \ \mathrm{COS}\ 2\alpha)$$

(β IN RADIANS)

STRIP LOAD PARALLEL TO WALL

NOTES:

1. THESE GUIDELINES APPLY TO RIGID WALLS WITH POISSON'S RATIO ASSUMED TO BE 0.5 FOR BACKFILL MATERIALS.
2. LATERAL PRESSURES FROM ANY COMBINATION OF ABOVE LOADS MAY BE DETERMINED BY THE PRINCIPLE OF SUPERPOSITION.
3. VALUES IN THIS FIGURE ARE UNFACTORED.

GEODESIGN INC 703 Broadway Street - Suite 650 Vancouver WA 98660 360.693.8416 www.geodesigninc.com	LONEWOLF-7-01	**SURCHARGE-INDUCED LATERAL EARTH PRESSURES**	
	JULY 2018	BLOCK 8 WASHOUGAL, WA	**FIGURE 4**

APPENDIX A

APPENDIX A

FIELD EXPLORATIONS

We explored subsurface conditions at the site by excavating two test pits (TP-1 and TP-2) to depths between 7.0 and 7.5 feet BGS. Excavation services were provided by Premier Civil Works of La Center, Washington. The exploration logs are presented in this appendix.

The locations of the explorations are shown on Figure 2 and were determined in the field by pacing and taping from existing site features. This information should be considered accurate only to the degree implied by the methods used.

A member of our geotechnical staff observed the explorations. We collected representative samples of the various soils encountered in the explorations for geotechnical laboratory testing.

SOIL SAMPLING

Disturbed samples of the soil observed in the test pits were collected from the walls or base of the test pits using the excavator bucket. Sampling intervals are shown on the exploration logs.

SOIL CLASSIFICATION

The soil samples were classified in accordance with the “Exploration Key” (Table A-1) and “Soil Classification System” (Table A-2), which are presented in this appendix. The exploration logs indicate the depths at which the soils or their characteristics change, although the change actually could be gradual. If the change occurred between sample locations, the depth was interpreted. Classifications are shown on the exploration logs.

LABORATORY TESTING

CLASSIFICATION

The soil samples were classified in the laboratory to confirm field classifications. The laboratory classifications are shown on the exploration logs if those classifications differed from the field classifications.

MOISTURE CONTENT

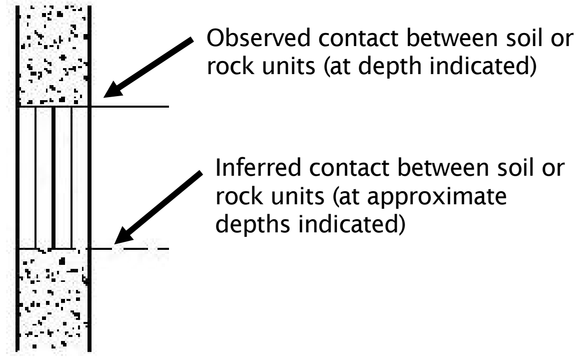
We tested the natural moisture content of select soil samples in general accordance with ASTM D2216. The natural moisture content is a ratio of the weight of the water to soil in a test sample and is expressed as a percentage. The test results are presented in this appendix.

GRAIN-SIZE TESTING

We completed grain-size testing on select soil samples in order to determine the distribution of soil particle sizes. The testing consisted percent fines determination (percent passing the U.S. Standard No. 200 sieve) analyses completed in general accordance with ASTM D1140 (P200) and sieve analyses completed in general accordance with ASTM C117 and ASTM C136.

SYMBOL	SAMPLING DESCRIPTION
◨	Location of sample obtained in general accordance with ASTM D 1586 Standard Penetration Test with recovery
▮	Location of sample obtained using thin-wall Shelby tube or Geoprobe® sampler in general accordance with ASTM D 1587 with recovery
▥	Location of sample obtained using Dames & Moore sampler and 300-pound hammer or pushed with recovery
▤	Location of sample obtained using Dames & Moore and 140-pound hammer or pushed with recovery
⊠	Location of sample obtained using 3-inch-O.D. California split-spoon sampler and 140-pound hammer
☒	Location of grab sample
▒	Rock coring interval
▽	Water level during drilling
▼	Water level taken on date shown

Graphic Log of Soil and Rock Types



GEOTECHNICAL TESTING EXPLANATIONS

ATT	Atterberg Limits	P	Pushed Sample
CBR	California Bearing Ratio	PP	Pocket Penetrometer
CON	Consolidation	P200	Percent Passing U.S. Standard No. 200 Sieve
DD	Dry Density		
DS	Direct Shear	RES	Resilient Modulus
HYD	Hydrometer Gradation	SIEV	Sieve Gradation
MC	Moisture Content	TOR	Torvane
MD	Moisture-Density Relationship	UC	Unconfined Compressive Strength
NP	Nonplastic	VS	Vane Shear
OC	Organic Content	kPa	Kilopascal

ENVIRONMENTAL TESTING EXPLANATIONS

CA	Sample Submitted for Chemical Analysis	ND	Not Detected
P	Pushed Sample	NS	No Visible Sheen
PID	Photoionization Detector Headspace Analysis	SS	Slight Sheen
		MS	Moderate Sheen
ppm	Parts per Million	HS	Heavy Sheen

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703 Broadway Street - Suite 650
Vancouver WA 98660
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EXPLORATION KEY

TABLE A-1

RELATIVE DENSITY - COARSE-GRAINED SOIL

Relative Density	Standard Penetration Resistance	Dames & Moore Sampler (140-pound hammer)	Dames & Moore Sampler (300-pound hammer)
Very Loose	0 – 4	0 - 11	0 - 4
Loose	4 – 10	11 - 26	4 - 10
Medium Dense	10 – 30	26 - 74	10 - 30
Dense	30 – 50	74 - 120	30 - 47
Very Dense	More than 50	More than 120	More than 47

CONSISTENCY - FINE-GRAINED SOIL

Consistency	Standard Penetration Resistance	Dames & Moore Sampler (140-pound hammer)	Dames & Moore Sampler (300-pound hammer)	Unconfined Compressive Strength (tsf)
Very Soft	Less than 2	Less than 3	Less than 2	Less than 0.25
Soft	2 - 4	3 – 6	2 - 5	0.25 - 0.50
Medium Stiff	4 - 8	6 – 12	5 - 9	0.50 - 1.0
Stiff	8 - 15	12 – 25	9 - 19	1.0 - 2.0
Very Stiff	15 - 30	25 – 65	19 – 31	2.0 - 4.0
Hard	More than 30	More than 65	More than 31	More than 4.0

PRIMARY SOIL DIVISIONS			GROUP SYMBOL	GROUP NAME
COARSE-GRAINED SOIL (more than 50% retained on No. 200 sieve)	GRAVEL (more than 50% of coarse fraction retained on No. 4 sieve)	CLEAN GRAVEL (< 5% fines)	GW or GP	GRAVEL
		GRAVEL WITH FINES (≥ 5% and ≤ 12% fines)	GW-GM or GP-GM	GRAVEL with silt
			GW-GC or GP-GC	GRAVEL with clay
		GRAVEL WITH FINES (> 12% fines)	GM	silty GRAVEL
			GC	clayey GRAVEL
			GC-GM	silty, clayey GRAVEL
	SAND (50% or more of coarse fraction passing No. 4 sieve)	CLEAN SAND (<5% fines)	SW or SP	SAND
		SAND WITH FINES (≥ 5% and ≤ 12% fines)	SW-SM or SP-SM	SAND with silt
			SW-SC or SP-SC	SAND with clay
		SAND WITH FINES (> 12% fines)	SM	silty SAND
			SC	clayey SAND
			SC-SM	silty, clayey SAND
FINE-GRAINED SOIL (50% or more passing No. 200 sieve)	SILT AND CLAY	Liquid limit less than 50	ML	SILT
			CL	CLAY
			CL-ML	silty CLAY
			OL	ORGANIC SILT or ORGANIC CLAY
		Liquid limit 50 or greater	MH	SILT
			CH	CLAY
			OH	ORGANIC SILT or ORGANIC CLAY
HIGHLY ORGANIC SOIL			PT	PEAT

MOISTURE CLASSIFICATION

Term	Field Test
dry	very low moisture, dry to touch
moist	damp, without visible moisture
wet	visible free water, usually saturated

ADDITIONAL CONSTITUENTS

Secondary granular components or other materials such as organics, man-made debris, etc.					
Percent	Silt and Clay In:		Percent	Sand and Gravel In:	
	Fine-Grained Soil	Coarse-Grained Soil		Fine-Grained Soil	Coarse-Grained Soil
< 5	trace	trace	< 5	trace	trace
5 – 12	minor	with	5 – 15	minor	minor
> 12	some	silty/clayey	15 – 30	with	with
			> 30	sandy/gravelly	Indicate %

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SOIL CLASSIFICATION SYSTEM

TABLE A-2

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	● MOISTURE CONTENT % 0 50 100	COMMENTS
0.0		ASPHALT CONCRETE (3.0 to 4.0 inches).					
		AGGREGATE BASE (6.0 inches).	0.3				
		Stiff, brown SILT with cobbles (ML), minor sand; moist.	0.8				
		with boulders at 2.0 feet			☒		Moderate caving observed from 2.0 to 4.0 feet.
2.5							
		Dense, brown GRAVEL (GW-GM) with silt, sand, cobbles, and boulders; moist.	3.5				
					☒		
5.0							
				SIEV	☒	●	Infiltration test: 11.0 inches per hour at 5.5 feet.
				P200		●	P200 = 8%
		Exploration completed at a depth of 7.0 feet.	7.0				
7.5							
							No groundwater seepage observed to the depth explored. Surface elevation was not measured at the time of exploration.
10.0						0 50 100	

EXCAVATED BY: Premier Civil Works

LOGGED BY: A. Ferrero

COMPLETED: 06/29/18

EXCAVATION METHOD: trackhoe (see document text)

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LONEWOLF-7-01

JULY 2018

TEST PIT TP-1

BLOCK 8
WASHOUGAL, WA

FIGURE A-1

MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	● MOISTURE CONTENT %	COMMENTS
ASPHALT CONCRETE (3.0 to 4.0 inches).					
AGGREGATE BASE (6.0 inches).	0.3				
Stiff, brown SILT with cobbles (ML), minor sand, trace organics; moist.	0.8		☒		
Medium dense, brown GRAVEL with sand, silt, cobbles, and boulders (GW-GM); moist.	3.0		☒		Minor caving observed from 3.0 to 4.0 feet.
Dense, brown GRAVEL with sand, cobbles, and boulders (GW), minor silt; moist.	6.0	P200 SIEV	☒ ☒	● ●	Infiltration test: 11.0 inches per hour at 6.0 feet. P200 = 2% Infiltration test: 30.0 inches per hour at 7.0 feet.
Exploration completed at a depth of 7.5 feet.	7.5				No groundwater seepage observed to the depth explored. Surface elevation was not measured at the time of exploration.

EXCAVATED BY: Premier Civil Works

LOGGED BY: A. Ferrero

COMPLETED: 06/29/18

EXCAVATION METHOD: trackhoe (see document text)

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LONEWOLF-7-01

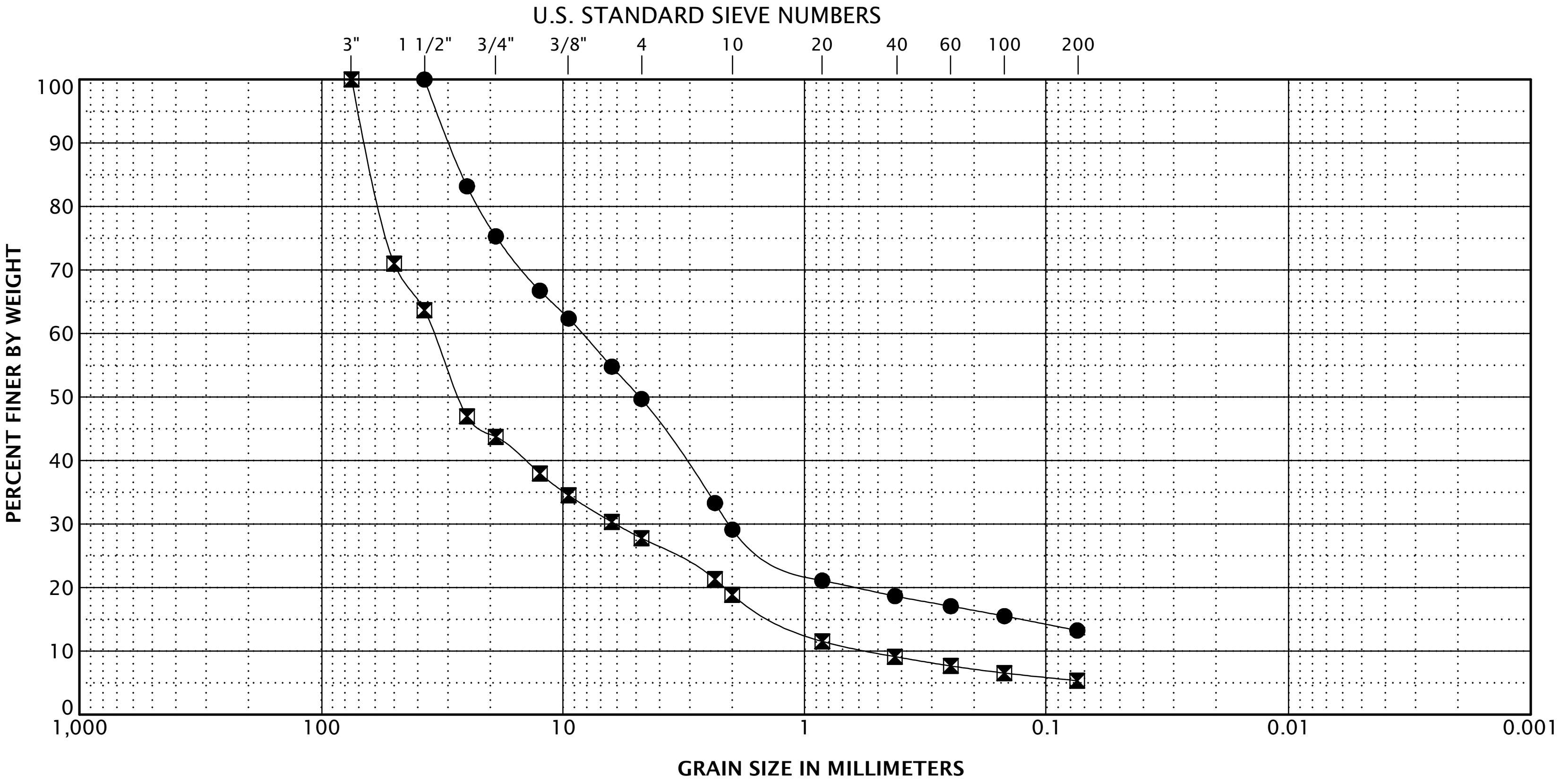
JULY 2018

TEST PIT TP-2

BLOCK 8
WASHOUGAL, WA

FIGURE A-2

TEST PIT LOG - 1 PER PAGE LONEWOLF-7-01-TP1_2.GPJ GEODESIGN.GDT PRINT DATE: 7/20/18:KM:KT



BOULDERS	COBBLES	GRAVEL		SAND			FINES	
		COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY

KEY	EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	MOISTURE CONTENT (PERCENT)	D60	D50	D30	D10	D5	GRAVEL (PERCENT)	SAND (PERCENT)	SILT (PERCENT)	CLAY (PERCENT)
●	TP-1	5.5	17	8.36	4.83	2.07			50	36	13	
☒	TP-2	7.0	12	34.30	26.91	6.07	0.55		72	22	5	

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LONEWOLF-7-01

JULY 2018

GRAIN-SIZE TEST RESULTS

BLOCK 8
WASHOUGAL, WA

FIGURE A-3

SAMPLE INFORMATION			MOISTURE CONTENT (PERCENT)	DRY DENSITY (PCF)	SIEVE			ATTERBERG LIMITS		
EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	ELEVATION (FEET)			GRAVEL (PERCENT)	SAND (PERCENT)	P200 (PERCENT)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
TP-1	5.5		17		50	36	13			
TP-1	6.5		12				8			
TP-2	6.0		9				2			
TP-2	7.0		12		72	22	5			

LAB SUMMARY LONEWOLF-7-01-TP1_2.GPJ GEODESIGN.GDT PRINT DATE: 7/19/18:KM

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LONEWOLF-7-01

JULY 2018

SUMMARY OF LABORATORY DATA

BLOCK 8
WASHOUGAL, WA

FIGURE A-4

APPENDIX B

APPENDIX B

PREVIOUS EXPLORATIONS IN THE SITE VICINITY

The boring and test pit logs for explorations previously completed at the site or in the site vicinity are presented in this appendix. The locations of the explorations are shown on Figure 2.

Date(s) Drilled	06/04/07	Logged By	YZ	Checked By	JMN
Drilling Contractor	Subsurface Technologies, Inc.	Drilling Method	Hollow Stem Auger	Sampling Methods	SPT / D&M
Auger Data	8-inch	Hammer Data	140 lb hammer/30 in drop	Drilling Equipment	Mobile Drill B-53
Total Depth (ft)	4	Surface Elevation (ft)		Groundwater Level (ft. bgs)	Not Determined
Vertical Datum		Datum/ System		Easting(x): Northing(y):	

Depth feet	SAMPLES Interval	Recovered (in)	Blows/foot	Sub-Sample Sample Number	Water Level	Graphic Log	Group Symbol	MATERIAL DESCRIPTION			OTHER TESTS AND NOTES
0						■	AC	4 inches asphalt concrete and 4 inches gravel base			
						‖	ML	Brown silt, trace clay and some rock fragments (very stiff, moist)			
	■	7	34	1							
						○	GP	Cobbles and some silt and sand (dense, moist) Auger refusal at 3 feet			B-1 (2.5-3.5)
								Bottom of hole at 4 feet Groundwater not encountered during drilling			
5											
10											
15											

Note: See Figure A-1 for explanation of symbols.

LOG OF BORING B-1

GeoEngineers

Project: Lone Wolf Investment

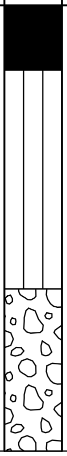
Project Location: Washougal, Washington

Project Number: 16370-001-00

Figure: A- 2

Sheet 1 of 1

Date(s) Drilled	06/04/07	Logged By	YZ	Checked By	JMN
Drilling Contractor	Subsurface Technologies, Inc.	Drilling Method	Hollow Stem Auger	Sampling Methods	SPT / D&M
Auger Data	8-inch	Hammer Data	140 lb hammer/30 in drop	Drilling Equipment	Mobile Drill B-53
Total Depth (ft)	5.5	Surface Elevation (ft)		Groundwater Level (ft. bgs)	Not Determined
Vertical Datum		Datum/ System		Easting(x): Northing(y):	



Depth feet	SAMPLES Interval	Recovered (in)	Blows/foot	Sub-Sample Sample Number	Water Level	Graphic Log	Group Symbol	MATERIAL DESCRIPTION			OTHER TESTS AND NOTES
0							AC	4 inches asphalt concrete and 4 inches gravel base			
							ML	Brown silt with trace clay (stiff, moist)			
		6	18	1							B-2 (2.5-4)
							GW	Cobbles, some gravel, sand and silt (dense, moist to damp)			
5		0	50/4"	2							

Boring terminated at 5.5 feet due to auger refusal
Groundwater not encountered during drilling

10

15

Note: See Figure A-1 for explanation of symbols.

LOG OF BORING B-2

GeoEngineers

Project:	Lone Wolf Investment
Project Location:	Washougal, Washington
Project Number:	16370-001-00

Figure: A- 3
Sheet 1 of 1

Date(s) Drilled	06/04/07	Logged By	YZ	Checked By	JMN
Drilling Contractor	Subsurface Technologies, Inc.	Drilling Method	Hollow Stem Auger	Sampling Methods	SPT / D&M
Auger Data	8-inch	Hammer Data	140 lb hammer/30 in drop	Drilling Equipment	Mobile Drill B-53
Total Depth (ft)	11.2	Surface Elevation (ft)		Groundwater Level (ft. bgs)	8.5
Vertical Datum		Datum/ System		Easting(x): Northing(y):	

Depth feet	SAMPLES Interval	Recovered (in)	Blows/foot	Sub-Sample Sample Number	Water Level	Graphic Log	Group Symbol	MATERIAL DESCRIPTION			OTHER TESTS AND NOTES
0							AC	4 inches asphalt concrete and 4 inches gravel base			
							ML	Brown silt, trace clay (stiff, moist)			
		10	11	1							
							GP	Cobbles and gravel with little sand (dense, damp)			
5		7	48	2							
		13	68	3							
							SP	Brown coarse sand with trace silt and clay (very dense, moist)			
					▼						
10		10	50/5"	4			BSLT	Gray medium weathered basalt (possible boulder) with trace silt, sand and clay (very dense, moist)			

Boring terminated at 11.2 feet due to auger refusal
Groundwater encountered at 8.5 feet

15

Note: See Figure A-1 for explanation of symbols.

V6_GTBORING P:\16\1637000100\FINALS\1637000100.GPJ GEIV6_1.GDT 6/29/07

LOG OF BORING B-3

GEOENGINEERS

Project: Lone Wolf Investment
Project Location: Washougal, Washington
Project Number: 16370-001-00

Figure: A- 4
Sheet 1 of 1

Date(s) Drilled	06/04/07	Logged By	YZ	Checked By	JMN
Drilling Contractor	Subsurface Technologies, Inc.	Drilling Method	Hollow Stem Auger	Sampling Methods	SPT / D&M
Auger Data	8-inch	Hammer Data	140 lb hammer/30 in drop	Drilling Equipment	Mobile Drill B-53
Total Depth (ft)	7	Surface Elevation (ft)		Groundwater Level (ft. bgs)	Not Determined
Vertical Datum		Datum/ System		Easting(x): Northing(y):	

Depth feet	SAMPLES Interval	Recovered (in)	Blows/foot	Sub-Sample Sample Number	Water Level	Graphic Log	Group Symbol	MATERIAL DESCRIPTION			OTHER TESTS AND NOTES
0							AC	4 inches asphalt concrete and 4 inches gravel base			
							ML	Brown silt, trace gravel (medium stiff, moist)			
		7	6	1							
5		10	38	2							
							GP	Moderately weathered basalt (possible boulder) some coarse sand and gravel and silt (dense, moist)			
								Boring terminated at 7 feet due to auger refusal Groundwater not encountered during drilling			
10											
15											

Note: See Figure A-1 for explanation of symbols.

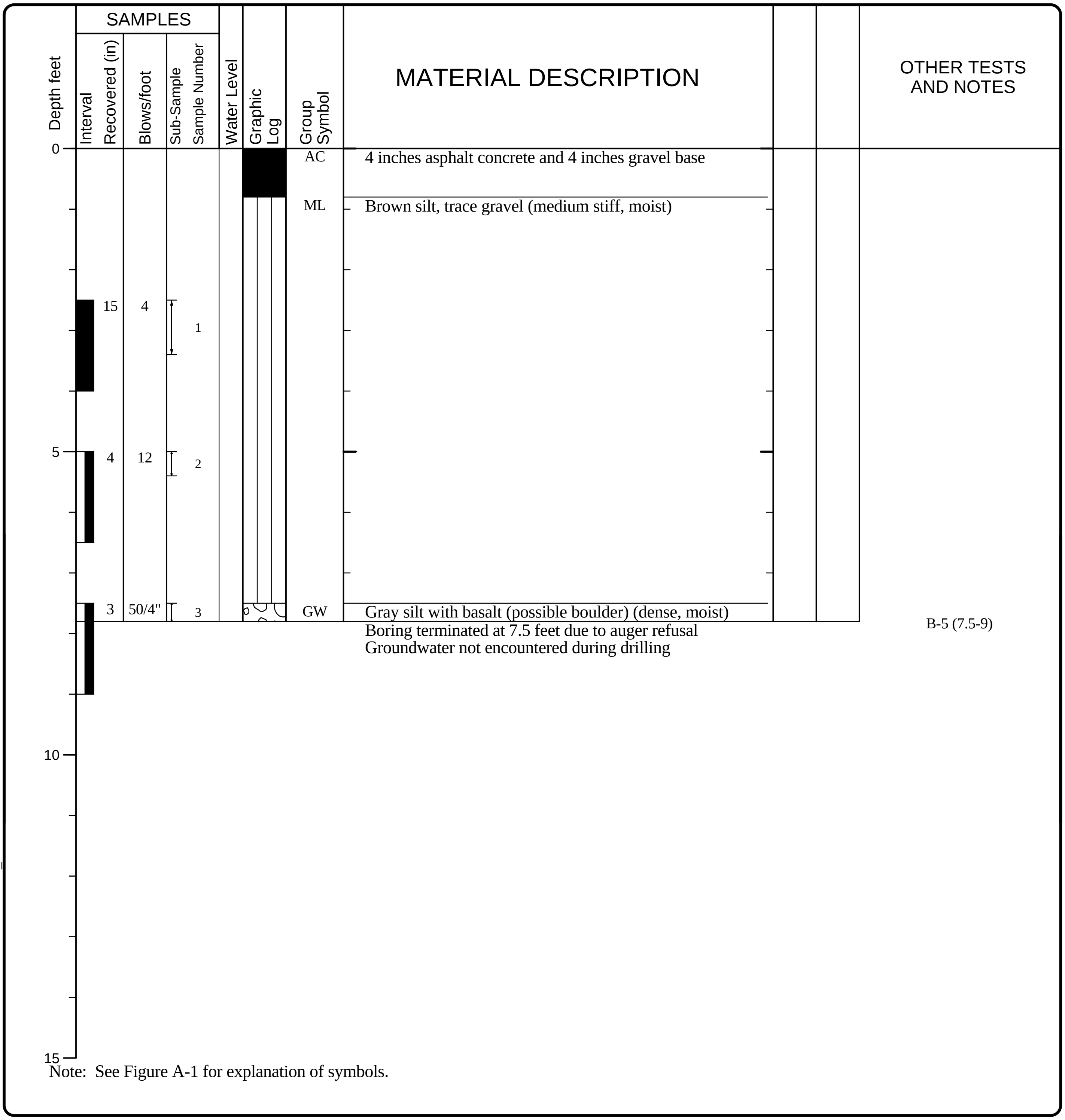
LOG OF BORING B-4

GeoEngineers

Project: Lone Wolf Investment
Project Location: Washougal, Washington
Project Number: 16370-001-00

Figure: A- 5
Sheet 1 of 1

Date(s) Drilled	06/05/07	Logged By	YZ	Checked By	JMN
Drilling Contractor	Subsurface Technologies, Inc.	Drilling Method	Hollow Stem Auger	Sampling Methods	SPT / D&M
Auger Data	8-inch	Hammer Data	140 lb hammer/30 in drop	Drilling Equipment	Mobile Drill B-53
Total Depth (ft)	7.8	Surface Elevation (ft)		Groundwater Level (ft. bgs)	Not Determined
Vertical Datum		Datum/ System		Easting(x): Northing(y):	



Depth feet	SAMPLES Interval	 Recovered (in)	 Blows/foot	 Sub-Sample Sample Number	Water Level	Graphic Log	Group Symbol	MATERIAL DESCRIPTION			OTHER TESTS AND NOTES
0							AC	4 inches asphalt concrete and 4 inches gravel base			
							ML	Brown silt, trace gravel (medium stiff, moist)			
		15	4	1							
5		4	12	2							
		3	50/4"	3			GW	Gray silt with basalt (possible boulder) (dense, moist) Boring terminated at 7.5 feet due to auger refusal Groundwater not encountered during drilling			B-5 (7.5-9)
10											
15											

Note: See Figure A-1 for explanation of symbols.

LOG OF BORING B-5

GEOENGINEERS

Project: Lone Wolf Investment

Project Location: Washougal, Washington

Project Number: 16370-001-00

Figure: A- 6

Date(s) Drilled	06/05/07	Logged By	YZ	Checked By	JMN
Drilling Contractor	Subsurface Technologies, Inc.	Drilling Method	Hollow Stem Auger	Sampling Methods	SPT / D&M
Auger Data	8-inch	Hammer Data	140 lb hammer/30 in drop	Drilling Equipment	Mobile Drill B-53
Total Depth (ft)	8.5	Surface Elevation (ft)		Groundwater Level (ft. bgs)	Not Determined
Vertical Datum		Datum/ System		Easting(x): Northing(y):	

Depth feet	SAMPLES Interval	Recovered (in)	Blows/foot	Sub-Sample Sample Number	Water Level	Graphic Log	Group Symbol	MATERIAL DESCRIPTION			OTHER TESTS AND NOTES
0							AC	4 inches asphalt concrete and 4 inches gravel base			
							ML	Brown silt with trace sand and gravel (soft, moist)			
		0	2	1							
								Grades to sandy and hard below 5 feet			
5		11	66	2							B-6 (5-6.5)
							SW	Brown gravelly medium to coarse sand with some silt and basalt fragments (dense, moist)			
		12	50/6"	3							
								Boring terminated at 8.5 feet due to auger refusal Groundwater not encountered during drilling			
10											
15											

Note: See Figure A-1 for explanation of symbols.

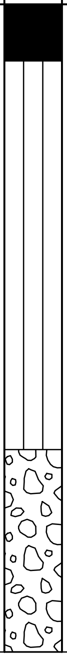
LOG OF BORING B-6

GEOENGINEERS

Project: Lone Wolf Investment
Project Location: Washougal, Washington
Project Number: 16370-001-00

Figure: A- 7
Sheet 1 of 1

Date(s) Drilled	06/05/07	Logged By	YZ	Checked By	JMN
Drilling Contractor	Subsurface Technologies, Inc.	Drilling Method	Hollow Stem Auger	Sampling Methods	SPT / D&M
Auger Data	8-inch	Hammer Data	140 lb hammer/30 in drop	Drilling Equipment	Mobile Drill B-53
Total Depth (ft)	8	Surface Elevation (ft)		Groundwater Level (ft. bgs)	Not Determined
Vertical Datum		Datum/ System		Easting(x): Northing(y):	



Depth feet	SAMPLES: Recovered (in)	Blows/foot	Sub-Sample Sample Number	Water Level	Group Symbol	MATERIAL DESCRIPTION	OTHER TESTS AND NOTES
0					AC	4 inches asphalt concrete and 3 inches gravel base	
					ML	Brown silt with trace cobbles and gravel (stiff, moist)	
	0	16	1				D&M sampler driven on cobbles; no sample recovered
5	18	36	2				
					GW	Brown sandy gravel with some silt trace cobbles (dense, moist)	
	2	50/5"	3				

Boring terminated at 8 feet due to auger refusal
Groundwater not encountered during drilling

Note: See Figure A-1 for explanation of symbols.

LOG OF BORING B-7

GeoEngineers

Project: Lone Wolf Investment
Project Location: Washougal, Washington
Project Number: 16370-001-00

Figure: A- 8
Sheet 1 of 1

Date Excavated: 06/06/07
Logged by: BDN
Equipment: Hitachi EX30 Trackhoe
Surface Elevation (ft):

Depth feet	Sample	Sample Number	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	OTHER TESTS AND NOTES
0				CR	4 inches crushed rock (construction-related fill)		
				AC	2 inches asphalt concrete pavement		
				CR	8 inches pit run basalt rock (fill)		
	☒	E1		ML	Dark brown silt with organics and fine roots (medium stiff to stiff, moist) (old topsoil) Few organics below 1 foot		
	☒	G1					
				SM	Brown fine silty sand with boulders up to 1.5 feet diameter (dense, moist)		
5	☒	G2			Grades with increased coarse sand		

Test pit terminated at 5.5 feet due to refusal on boulders
Groundwater not encountered during drilling

10

15

Note: See Figure A-1 for explanation of symbols.

LOG OF TEST PIT TP-1

GEOENGINEERS

Project: Lone Wolf Investment
Project Location: Washougal, Washington
Project Number: 16370-001-00

Figure: A- 9
Sheet 1 of 1

Date Excavated: 06/06/07
Logged by: BDN
Equipment: Hitachi EX30 Trackhoe
Surface Elevation (ft):

Depth feet	Sample	Sample Number	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	OTHER TESTS AND NOTES
0				CR	6 inches crushed rock (construction haul road)		
	☒	E1		ML	Dark brown silt with fine roots, organic odor (medium stiff, moist) (fill)		
	☒	G1		SM	Brown silty fine sand with boulders up to 2 feet diameter (medium dense, moist)		
5	☒	G1					
					Grades with increased coarse sand		
				SP	Light brown sandy gravel with cobbles (dense, moist)		

Test pit terminated at 6.5 feet due to refusal on boulders
Groundwater not encountered during drilling

10

15

Note: See Figure A-1 for explanation of symbols.

LOG OF TEST PIT TP-2

GeoEngineers

Project: Lone Wolf Investment
Project Location: Washougal, Washington
Project Number: 16370-001-00

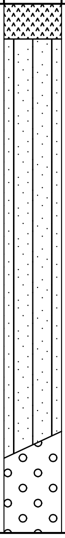
Figure: A- 10
Sheet 1 of 1

Date Excavated: 06/06/07

Logged by: BDN

Equipment: Hitachi EX30 Trackhoe

Surface Elevation (ft):

Depth feet	Sample	Sample Number	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	OTHER TESTS AND NOTES
0				TS	4 inches silt deposit with fine roots		
				SM	Brown fine silty sand with boulders up to 2.5 feet diameter; fine roots (loose to medium dense, moist)		
	☒	G1					
	☒	E1					
5				GP	Light brown sandy gravel with cobbles (dense, moist)		
	☒	G2, E2					TP-3 (5.5-6)

Test pit terminated at 6 feet due to refusal on boulders
Groundwater not encountered during drilling

10

15

Note: See Figure A-1 for explanation of symbols.

LOG OF TEST PIT TP-3

Project: Lone Wolf Investment
Project Location: Washougal, Washington
Project Number: 16370-001-00

Figure: A- 11
Sheet 1 of 1

Date Excavated: 06/06/07

Logged by: BDN

Equipment: Hitachi EX30 Trackhoe

Surface Elevation (ft):

Depth feet	Sample	Sample Number	Graphic Log	Group Symbol	MATERIAL DESCRIPTION	Moisture Content %	OTHER TESTS AND NOTES
0				TS	6 inches silt topsoil with roots		
				SM	Brown fine silty sand with boulders up to 2.5 feet diameter with occasional roots up to 1 inch (medium stiff, moist)		
	☒	G1					
	☒	E1					
5				GP-GM	Brown sandy gravel with rounded cobbles (dense, moist)		
	☒	E2					
	☒	G2					TP-4 (6.5-7)
10							
15							

Test pit terminated at 7 feet due to refusal on boulders
Groundwater not encountered during drilling
Minor caving observed at 2-3 feet

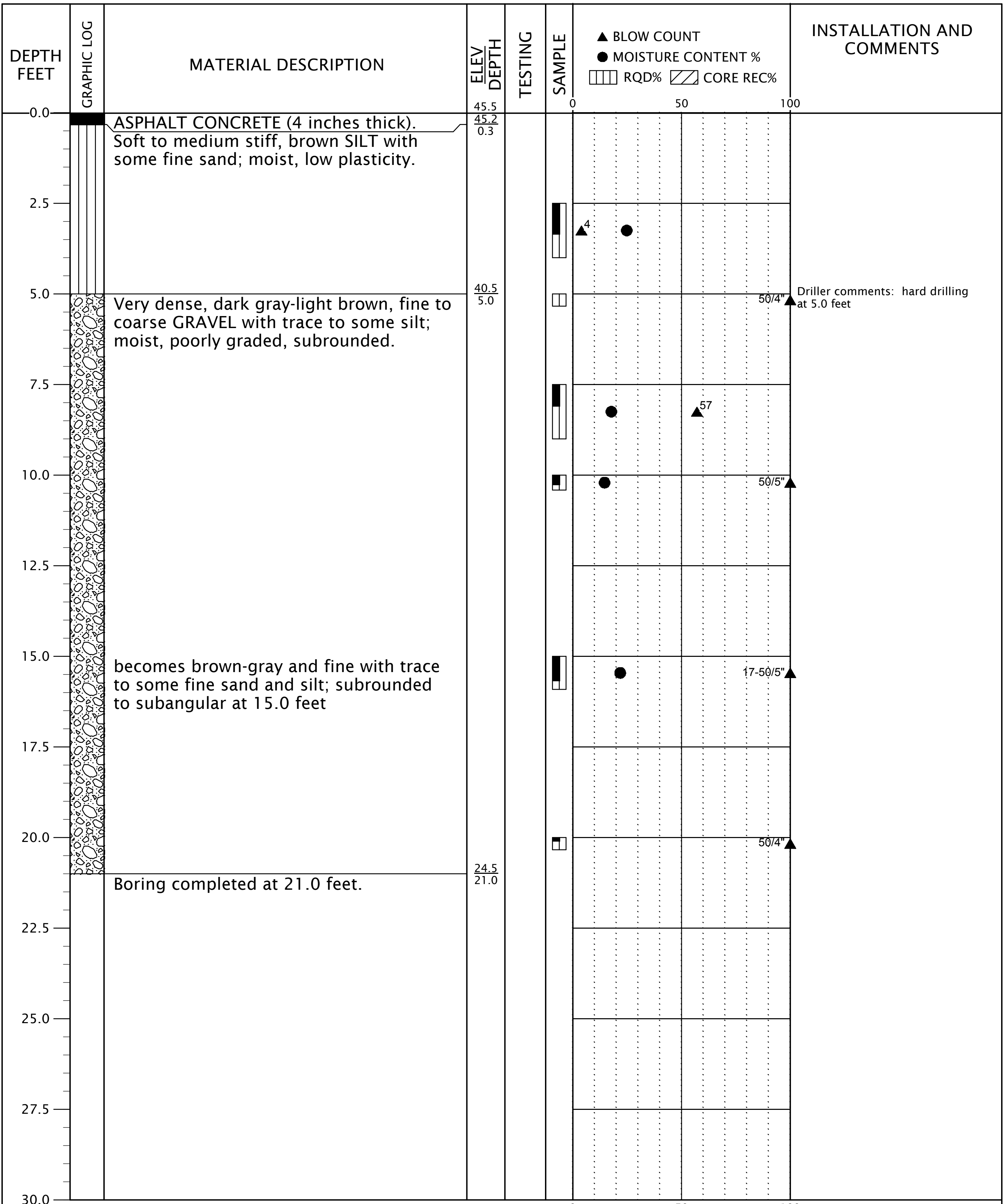
Note: See Figure A-1 for explanation of symbols.

LOG OF TEST PIT TP-4

GEOENGINEERS

Project: Lone Wolf Investment
Project Location: Washougal, Washington
Project Number: 16370-001-00

Figure: A- 12
Sheet 1 of 1



ASPHALT CONCRETE (4 inches thick).

Soft to medium stiff, brown SILT with some fine sand; moist, low plasticity.

Very dense, dark gray-light brown, fine to coarse GRAVEL with trace to some silt; moist, poorly graded, subrounded.

becomes brown-gray and fine with trace to some fine sand and silt; subrounded to subangular at 15.0 feet

Boring completed at 21.0 feet.

Driller comments: hard drilling at 5.0 feet

DRILLED BY: Subsurface Technologies

LOGGED BY: BBP

COMPLETED: 07/29/05

BORING METHOD: mud rotary (see report text)

BORING BIT DIAMETER: 4 7/8-inch

GEODESIGN INC

1201 SE Tech Center Drive - Suite 160
Vancouver WA 98683
Off 360.693.8416 Fax 360.693.8426

LONEWOLF-1-01

AUGUST 2005

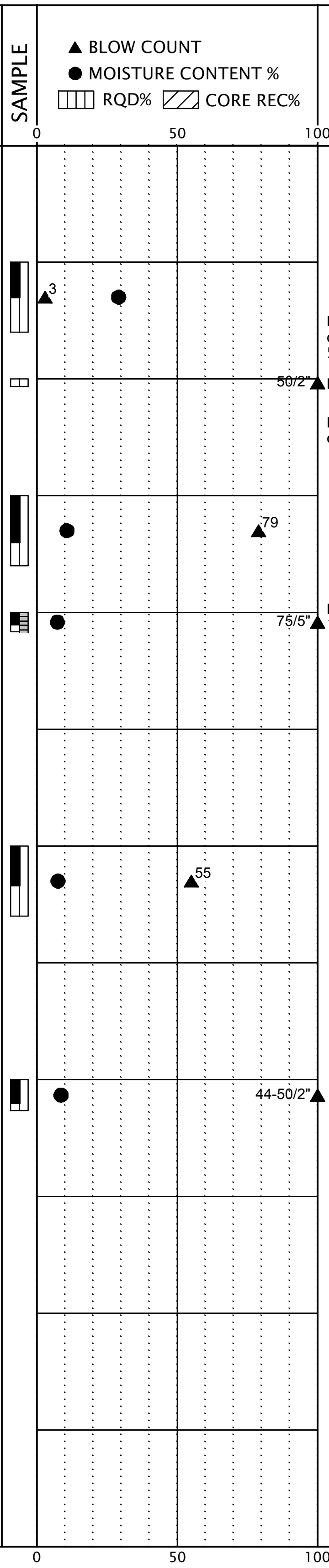
BORING B-1

BLOCK 11 MIXED-USE DEVELOPMENT
WASHOUGAL, WA

FIGURE A-1

BORING LOG LONEWOLF-1-01-B1-4.GPJ GEODESIGN.GDT PRINT DATE: 8/19/05:IAO:RLF

DEPTH FEET	MATERIAL DESCRIPTION	ELEV DEPTH	TESTING	INSTALLATION AND COMMENTS
0.0	ASPHALT CONCRETE (6 inches thick).	45.0		
	BASE ROCK (12 inches thick).	44.5 0.5		
	Soft, brown SILT with some fine sand to sandy SILT; moist, nonplastic, few small fragments of plant debris.	43.5 1.5	ATT	
2.5				Hard drilling at 4.0 feet; kicked off auger; moved approximately 5 feet northeast and redrilled
	Very dense, gray-brown, fine, silty GRAVEL; moist, poorly graded, subrounded to subangular, some iron-oxide staining.	41.0 4.0		
5.0				No recovery at 5.0 feet
				Hard, slow drilling; gravel in cuttings at 6.0 feet
7.5				
10.0	Very dense, dark gray, gravelly COBBLES with trace silt; moist, poorly graded, subrounded.	35.0 10.0	SIEV	Infiltration test: negligible at 10.0 feet
12.5				
15.0	Very dense, brown-gray, fine to coarse GRAVEL with some silt; moist, poorly graded, subrounded.	30.0 15.0		
17.5				
20.0			SIEV	
	Boring completed at 20.7 feet.	24.3 20.7		
22.5				
25.0				
27.5				
30.0				



DRILLED BY: Subsurface Technologies

LOGGED BY: BBP

COMPLETED: 07/29/05

BORING METHOD: hollow-stem auger (see report text)

BORING BIT DIAMETER: 8-inch

GEODESIGN INC

1201 SE Tech Center Drive - Suite 160
Vancouver WA 98683
Off 360.693.8416 Fax 360.693.8426

LONEWOLF-1-01

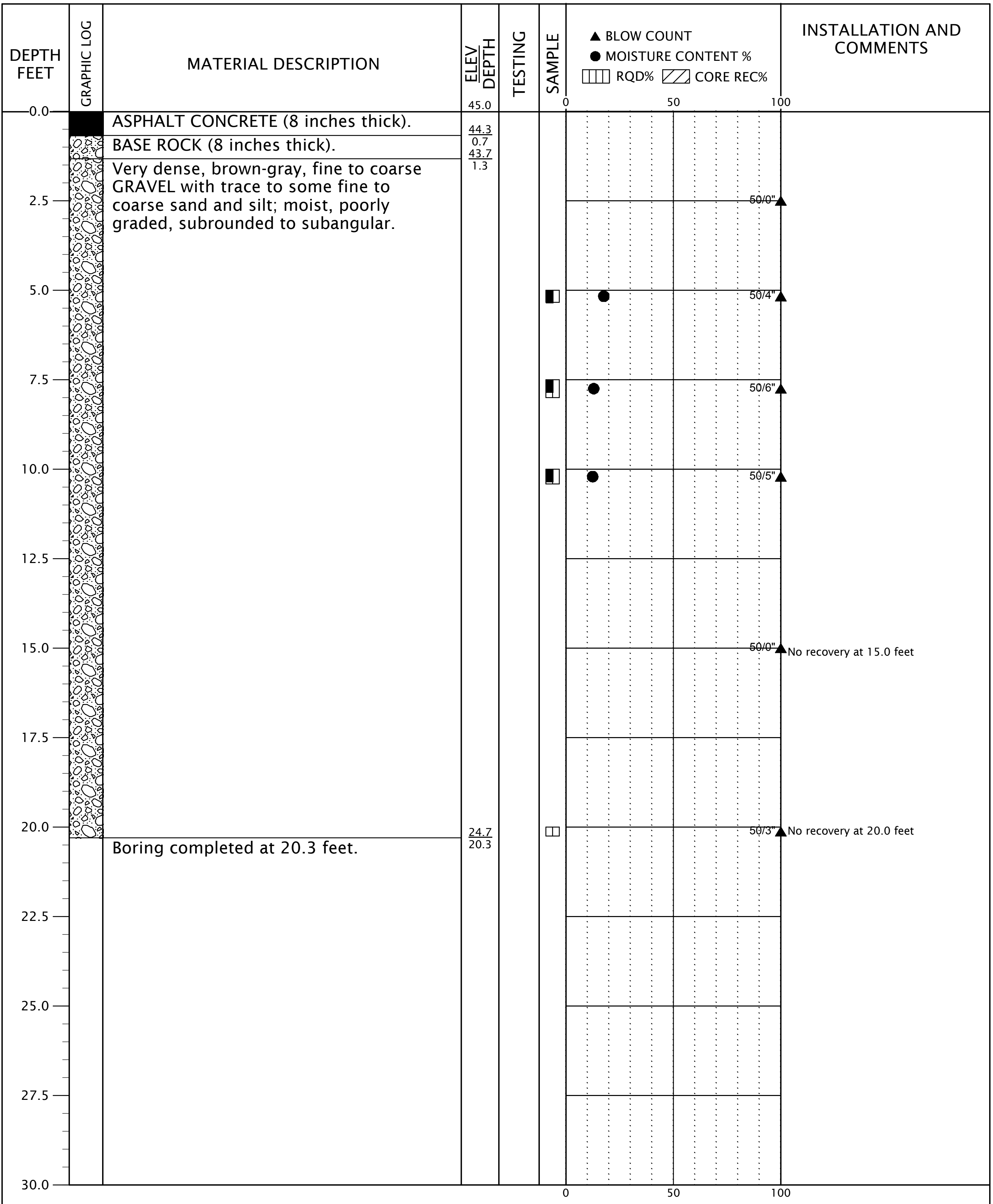
AUGUST 2005

BORING B-2

BLOCK 11 MIXED-USE DEVELOPMENT
WASHOUGAL, WA

FIGURE A-2

BORING LOG LONEWOLF-1-01-B1-4.GPJ GEODESIGN.GDT PRINT DATE: 8/19/05:IAO:RLF



DEPTH FEET	MATERIAL DESCRIPTION	ELEV DEPTH
0.0	ASPHALT CONCRETE (8 inches thick).	45.0
0.7	BASE ROCK (8 inches thick).	44.3 0.7
1.3	Very dense, brown-gray, fine to coarse GRAVEL with trace to some fine to coarse sand and silt; moist, poorly graded, subrounded to subangular.	43.7 1.3
20.3	Boring completed at 20.3 feet.	24.7 20.3

No recovery at 15.0 feet

No recovery at 20.0 feet

DRILLED BY: Subsurface Technologies

LOGGED BY: BBP

COMPLETED: 07/29/05

BORING METHOD: mud rotary (see report text)

BORING BIT DIAMETER: 4 7/8-inch

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Vancouver WA 98683
Off 360.693.8416 Fax 360.693.8426

LONEWOLF-1-01

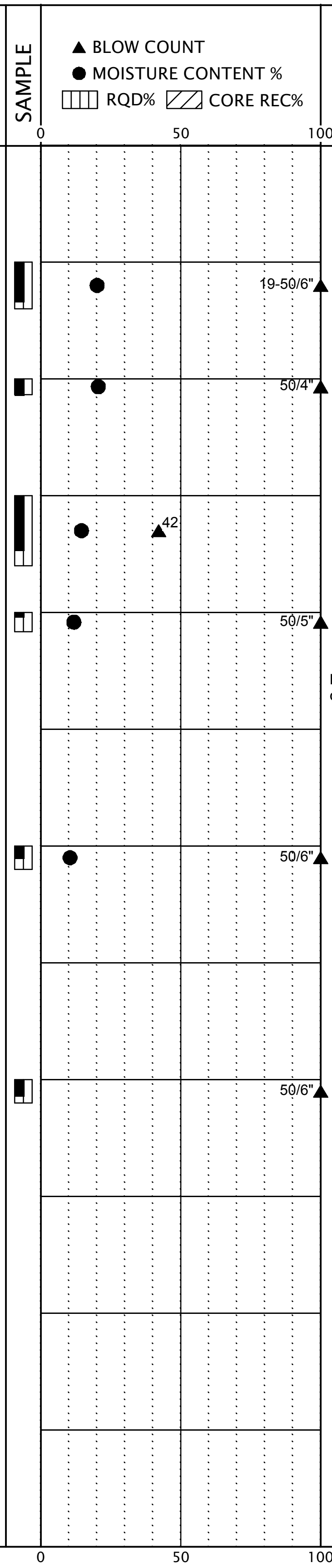
AUGUST 2005

BORING B-3

BLOCK 11 MIXED-USE DEVELOPMENT
WASHOUGAL, WA

FIGURE A-3

DEPTH FEET	MATERIAL DESCRIPTION	ELEV DEPTH	INSTALLATION AND COMMENTS
0.0		42.0	
	ASPHALT CONCRETE (3 inches thick).	41.8 0.3	
	BASE ROCK (8 inches thick).	41.1 0.9	
	Hard, brown, gravelly SILT with trace to some fine sand; moist, nonplastic.		
2.5			
5.0	Very dense, brown-gray, fine GRAVEL with trace sand and silt; moist, poorly graded, subrounded to subangular.	37.0 5.0	
7.5	becomes fine to coarse at 7.5 feet		
10.0			
			Lost approximately 20 gallons of mud at 11.5 feet
12.5			
15.0			
17.5			
20.0			
	Boring completed at 20.5 feet.	21.5 20.5	
22.5			
25.0			
27.5			
30.0			



DRILLED BY: Subsurface Technologies

LOGGED BY: BBP

COMPLETED: 07/29/05

BORING METHOD: mud rotary (see report text)

BORING BIT DIAMETER: 4 7/8-inch

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Vancouver WA 98683
Off 360.693.8416 Fax 360.693.8426

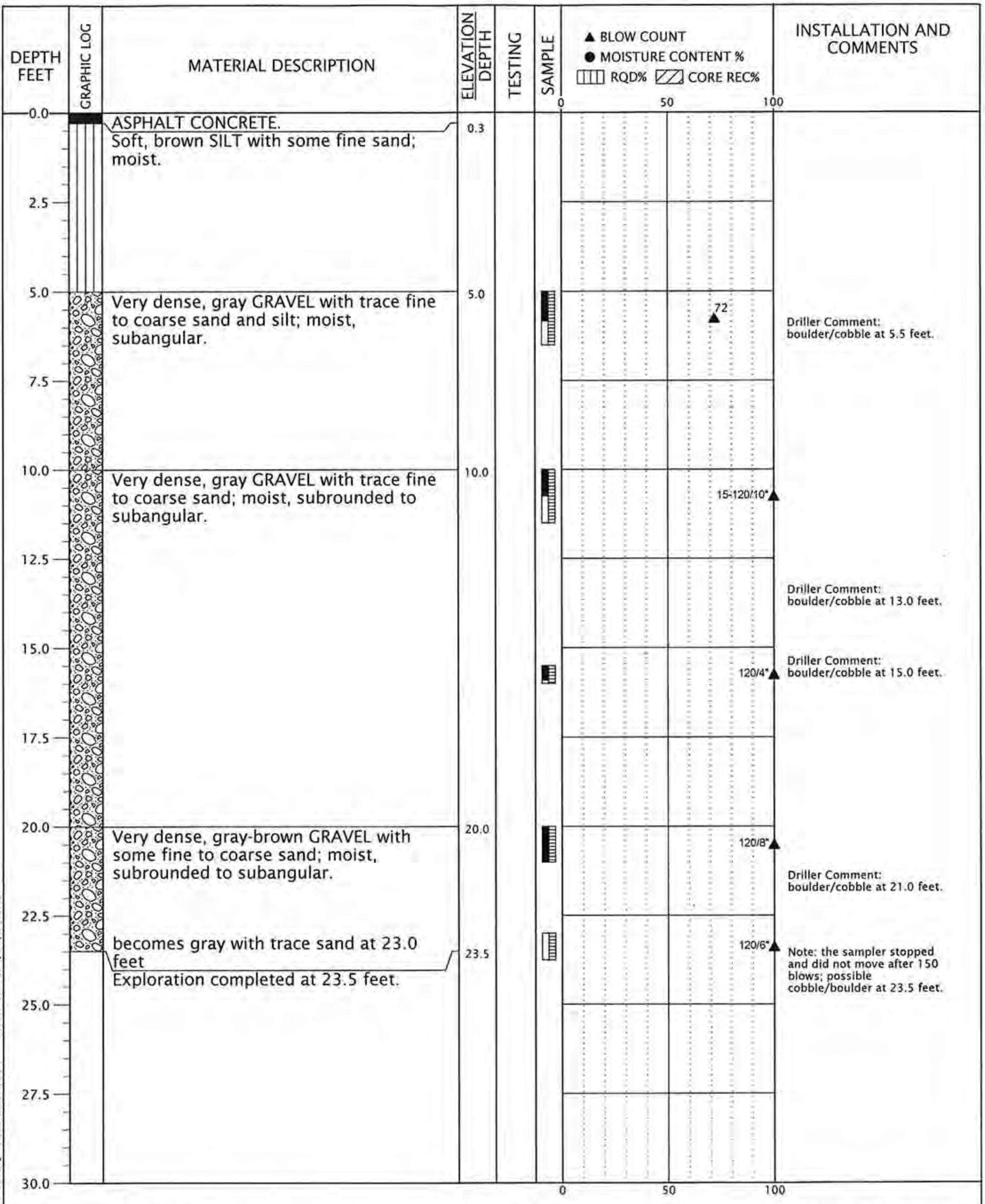
LONEWOLF-1-01

BORING B-4

AUGUST 2005

BLOCK 11 MIXED-USE DEVELOPMENT
WASHOUGAL, WA

FIGURE A-4



DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	BLOW COUNT	INSTALLATION AND COMMENTS
0.0	ASPHALT CONCRETE.	0.3		
	Soft, brown SILT with some fine sand; moist.	5.0		
5.0	Very dense, gray GRAVEL with trace fine to coarse sand and silt; moist, subangular.	10.0	72	Driller Comment: boulder/cobble at 5.5 feet.
10.0	Very dense, gray GRAVEL with trace fine to coarse sand; moist, subrounded to subangular.	20.0	15-120/10"	Driller Comment: boulder/cobble at 13.0 feet.
15.0			120/4"	Driller Comment: boulder/cobble at 15.0 feet.
20.0	Very dense, gray-brown GRAVEL with some fine to coarse sand; moist, subrounded to subangular.		120/8"	Driller Comment: boulder/cobble at 21.0 feet.
23.0	becomes gray with trace sand at 23.0 feet	23.5	120/6"	Note: the sampler stopped and did not move after 150 blows; possible cobble/boulder at 23.5 feet.
23.5	Exploration completed at 23.5 feet.			

DRILLED BY: Geo-Tech Explorations, Inc. LOGGED BY: NNP COMPLETED: 04/24/06

BORING METHOD: mud rotary (see report text) BORING BIT DIAMETER:

GEODESIGN INC 1201 SE Tech Center Drive - Suite 160 Vancouver WA 98683 Off 360.693.8416 Fax 360.693.8426	LONEWOLF-1-01	**BORING B-5**	
	MAY 2006	BLOCK 11 MIXED-USE DEVELOPMENT WASHOUGAL, WA	**FIGURE 2**

BORING LOG LONEWOLF-1-01-B5.GPJ GEODESIGN.GDT PRINT DATE: 4/27/06:SMS

TP-1

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE	● MOISTURE CONTENT % (0 50 100)	COMMENTS
0.0		ASPHALT CONCRETE.	0.4				
2.5 5.0		Medium stiff to stiff, brown SILT; moist, with 8-inch-thick organic silt layer at 7 inches. becomes sandy with some rounded gravel, cobbles, and boulders at 4.0 feet	6.0		☒		
7.5		Medium dense to dense, brown, coarse, silty SAND; moist.	9.0		☒	●	
10.0		Dense, gray-brown GRAVEL with some sand and silt; moist.	11.5	SIEV	☒	●	P200 = 14% Infiltration test: 60 inches/hour at 11.5 feet.
12.5		Test pit completed at 11.5 feet.					

TP-2

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE	● MOISTURE CONTENT % (0 50 100)	COMMENTS
0.0		ASPHALT CONCRETE.	0.4				
		Medium stiff to stiff, brown SILT; moist (fill).	1.5				
2.5 5.0 7.5		Medium stiff, brown SILT with black mottles and trace sand and boulders; moist (8-inch-thick organic silt layer at 1.5 feet). becomes sandy with some gravel, cobbles, and boulders at 4.5 feet	8.0		☒ ☒	● ●	
10.0 12.5		Dense, brown, silty SAND with some gravel, cobbles, and boulders; moist.	13.5	 SIEV	☒ ☒	 ●	P200 = 26% Infiltration test: 14 inches/hour at 11.0 feet.
		Test pit completed at 13.5 feet.			☒		

EXCAVATED BY: McDonald Excavating

LOGGED BY: MEM

COMPLETED: 03/10/06

EXCAVATION METHOD: trackhoe (see report text)

GEODESIGN INC

1201 SE Tech Center Drive - Suite 160
Vancouver WA 98683
Off 360.693.8416 Fax 360.693.8426

LONEWOLF-1-01

APRIL 2006

TEST PIT

BLOCK 11 MIXED-USE DEVELOPMENT
WASHOUGAL, WA

FIGURE A-1

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