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Report of Geotechnical Engineering Services

Camas Station
NW Brady Road & NW 16th Avenue

Camas, Washington

Prepared for

MAJ Development Corporation

February 7, 2022
0202499-006

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Report of Geotechnical Engineering Services

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1.0 INTRODUCTION

Hart Crowser, a division of Haley and Aldrich, (Hart Crowser), is pleased to present this report to MAJ Development Corporation (MAJ) summarizing the results of our recent geotechnical field explorations, testing, and engineering analyses completed for the proposed development project located in Camas, Washington. Our work was completed in general accordance with our geotechnical engineering services agreement dated November 30, 2021.

The rectangular shaped property is a single approximately 2.2-acre parcel (127357000) on the west side of NW Brady Road, just north of NW 16th Avenue. The subject parcel is currently undeveloped and is covered primarily with trees, shrubbery, and grass. The configuration of the development is still being planned, although conceptually it will include up to 3 single-story buildings. Two retail buildings that are planned to be 2800 square feet and 4,000 square feet will be located on the northern portion of the property. The southern portion of the property will contain a 7,300 square foot convenience store and car wash along with a fueling station and corresponding underground fuel tanks. Associated parking, landscaping, and trash enclosures will be included in the development.

Based on our experience with similar developments, we anticipate that the buildings will be supported on shallow footings with structural loads up to 2.5 kips/lineal foot for strip footings and up to 75 kips for column footings. We anticipate that new paving will be constructed throughout the site, including asphalt drive aisles and parking stalls, and possibly concrete sidewalks and trash container pads.

Based on the sloping nature of the site, we anticipate that grading mass grading will be required with mass cuts on the order of 4 to 8 feet deep and mass fills of approximately 15 feet thick. Additionally, deeper excavations (up to approximately 20 feet) will be required for installation of fuel underground storage tanks (USTs) and utilities.

This report contains the results of our analysis and provides recommendations for design and construction of the proposed development. The first section of this report provides an overview of the project information discussed in the text. The main body of the report presents our geotechnical engineering findings and recommendations in detail. Figures are presented at the end of the text. The location of the site is shown on Figure 1, and the existing site layout and topography with the proposed developments overlain is shown on Figure 2. Supporting information is provided in the appendices. Appendix A contains site subsurface exploration logs, Appendix B contains the results of laboratory testing completed for our analysis, and Appendix C contains a historical geotechnical data from a prior report prepared by Hart Crowser for work on NW Brady Road.

2.0 SCOPE OF SERVICES

The purpose of our geotechnical work was to evaluate the subsurface soil and groundwater conditions to aid in design and construction of the proposed development. Our scope of work was outlined in our proposal dated November 19, 2021, which generally included the following tasks:

- Reviewed readily available geologic, groundwater, and soil survey maps that cover the site vicinity.
- Conducted a field exploration program that included the following:
  - Marking the proposed exploration locations in the field and notifying the "One Call" service for public utility locates and engaging the services of a private utility locator for identifying on-site utilities.
  - Excavating 6 test pits to depths of 8.5 to 16 feet bgs.
  - Maintaining logs of the soils encountered in the explorations and collecting soil samples from the explorations.
  - Conducting 2 *in situ* infiltration tests at depths of 1.0 to 3.0 feet bgs.
- Conducted a program of laboratory testing on select soil samples. The laboratory tests conducted include moisture content, grain size distribution and Atterberg limits.
- Conducted engineering analysis to develop geotechnical design recommendations for infiltration systems, foundations, retaining walls, earthwork, pavements, and seismic design criteria.
- Evaluated code-based seismic hazards, including ground shaking and ground shaking amplification.
- Prepared this report outlining our findings and recommendations, including information related to the following:
  - Subsurface soil and groundwater conditions
  - Seismic hazards and design parameters
  - Site preparation and grading
  - Utility trench construction
  - Infiltration design parameters
  - Foundation design parameters
  - Lateral earth pressures
  - Pavement design
- Provided project management and support services, including coordinating staff and subcontractors and conducting telephone consultations and email communications with you and the design team.

3.0 SITE CONDITIONS

3.1 Geologic and Soil Mapping

The geology of the site is mapped in the United States Geologic Survey (USGS) Geologic *Map of the Camas Quadrangle, Clark County, Washington, and Multnomah County, Oregon* (Evarts and O'Connor 2008). The bedrock geology is mapped as "Basaltic Andesite of Prune Hill" (Qtbph), which are volcanic rocks of the Boring volcanic field and described as light to medium gray, microvesicular, basaltic andesite (Evarts and O'Connor 2008).

Based on available information regional groundwater in the project vicinity is greater than 100 feet below the ground surface. However, due to the presence of shallow bedrock, perched water may be encountered at the site. No nearby water well logs that encountered groundwater were found during our search of the Department of Ecology database. Nearby well logs were generally resource wells (e.g. geotechnical borings) which generally encountered clayey gravels, gravels and cobbles.

The near surface soils at the site are mapped by the U.S. Department of Agriculture (USDA) in the *Soil Survey of Clark County, Washington* (McGee 1972; USDA 2021) as mantled by Powell silt loam. The Powell unit is moderately well drained. The Powell soils are classified as hydrologic Soil Group D with a saturated hydraulic conductivity (permeability) of approximately 0.06 to 0.2 inches per hour in the most restrictive layer. Clark County (OTAK 2010) has classified the Powell silt loam as being part of Soil Group (SG) 3. As discussed in Section 6.0 Drainage Design Recommendations based on *in situ* soil characteristics and shallow perching water, it is our opinion that SG 4 is a more appropriate classification for the site soils.

3.2 Surface Conditions

The site is located at the northwest corner of NW 16th Avenue and NW Brady Road. The site is bound by single-family residential properties to the east, undeveloped property to the north, Prune Hill Sports Park to the south, and the recently closed Hidden Gardens Nursery to the west. The property is undeveloped and is covered in trees, shrubs, and grass.

The site slopes downward from the south to the north. The highest surveyed elevation of the site is about 525 feet (NAVD 88) located at the southwest corner of the site, and the lowest surveyed elevation of the site is 500 feet (NAVD 88) located at the northwest corner of the site.

3.3 Subsurface Conditions

3.3.1 General

Soil conditions interpreted from geologic maps and our explorations, in conjunction with soil properties inferred from field observations and laboratory tests, formed the basis for the conclusions and recommendations provided in this report.

We completed field explorations at the site by advancing six test pit excavations (designated TP-1 through TP-6) to depths of approximately 8.5 to 16.0 feet bgs. Two *in situ* soil infiltration tests were performed adjacent to test pits TP-2 (IT-2) and TP-5 (IT-1) at depths of approximately 3.0 to 1.0 feet bgs respectively.

The locations of the explorations are shown on Figure 2. Appendix A describes our field exploration procedures and presents field data and logs. Appendix B describes our laboratory testing procedures and results.

Based on the results of our explorations and visual field and laboratory observations of the site soils, the site is generally blanketed by residual soil derived from the underling basaltic andesite bedrock. The residual soil consists of an upper layer of lean clay that is underlain by very stiff gravelly clay, containing minor to moderate amounts of cobbles. Intact bedrock was not encountered in our explorations and is anticipated to be greater than approximately 15 feet bgs, though locally could be present at shallower depths. Detailed descriptions of the soils encountered are provided below.

3.3.2 Soils

Our explorations encountered 11 to 16 inches of topsoil throughout the site. Below the surface topsoil, we encountered residual soil. The uppermost layer of residual soil was between 6 and 8 feet thick and was gray-brown to yellow-brown lean clay. The consistency of the material determined from visual observation and excavator action indicate the material to be generally medium stiff to stiff. Moisture contents determined from laboratory testing varied between approximately 22 and 30 percent in this material. Fines content varied between approximately 88 and 92 percent.

Below the upper layer of residual soil, a lower layer was encountered that consisted of yellow-brown to red, clay with gravel and gravelly clay. These materials extended to depths of at least 16 feet bgs. The relative density/consistency of the material determined from visual observation and excavator action indicate the material is generally very dense/hard. Moisture contents determined from laboratory testing in the deeper residual soils ranged from approximately 20 to 28 percent. Fines content was approximately 57 percent for one sample from this layer. Grain size distribution analyses indicate that the material is classified as a gravelly clay with sand, however, based on the variability of the material observed during our explorations, we anticipate that this unit likely consists of clayey gravel and clay with gravel, as well. The material contained up to 20 percent of basaltic andesite cobbles. It was noted during lab testing that some of the gravels and cobbles were friable and could be broken down mechanically into smaller grain sizes.

Intact bedrock was not encountered in the explorations, though refer to the following section describing the condition of the bedrock that was encountered in nearby explorations.

3.3.3 Historical Borings

Historical borings from Hart Crower's previously completed project along Brady Road are generally consistent with the explorations at the site. Two borings, B-6 and B-8, were drilled adjacent to the current project site along Brady Road. Borings B-6 and B-8 indicates residual soil from below the pavement to explored depths of 5.5 feet and 16.5 feet, respectively.

Basaltic andesite was encountered below residual soil in historic borings and test pits to the north of the project site, including in test pit TP-1, located approximately 100 feet to the north of the subject site. That test pits encountered bedrock at a depth of 9 feet bgs. Several other test pits further from the site, encountered bedrock at depths of 7 to 15 feet bgs. The bedrock was described as consisting of slightly to highly weathered, gray to brown, moderately strong to very strong, slightly to moderately vesicular basaltic andesite. Cobbles consisting of basaltic andesite were also encountered within residual soils above intact bedrock.

Based on our review of these historical explorations and on available geologic information, we anticipate that intact bedrock is greater than 15 bgs at the subject site, though localized zones of intact rock could be encountered at shallower depths on site.

3.3.4 Groundwater

A regional groundwater table was not encountered during our explorations to a depth of 16 feet bgs. Based on our review of available groundwater data sources, we anticipate the groundwater at the site to be deep, greater than 100 feet bgs. However, during field exploration activities, we encountered ponding water and seepage in infiltration test hole IT-2 and test pits TP-2 through TP-6 at depths of 2 to 7 feet bgs. Seepage was observed to occur at the base of the topsoil layer and at the base of the upper layer of residual soil. Therefore, we expect to encounter perched water across the site, particularly during or after periods of rainfall.

3.3.5 Infiltration Testing

We performed two *in situ* infiltration tests at the project site. The tests were completed in shallow test holes advanced adjacent to the primary test pits. The infiltration tests were performed in general conformance with the methods prescribed in the City of Camas – Stormwater Design Standards Manual (Camas 2016). The results of the field testing and fines content and soil type are provided in Table 1. The drawdown and hydraulic conductivity values presented in Table 1 are not to be used for design but are provided to show the direct results of the field measurements and the calculated hydraulic conductivity.

Table 1 – Infiltration Test Data

Infiltration Test No.	Test Pit No.	Approximate Test Depth (feet)	Field Drawdown Rate (inches/hour)	Calculated Hydraulic Conductivity (inches/hour)	Soil Type (USCS)	Fines Content (percent)
IT-1	TP-2	3	0.7	0.15	CL	84.7
IT-2	TP-5	1	0[a]	0[a]	CL	93.2

Notes:

a. No infiltration observed during field testing. Perched water built up in the hole during testing.

The hydraulic conductivity of the on-site soils is sensitive to both the overall fines content as well as the relative size (gradation) of the sand particles. Both tests were performed in clay (CL).

Please refer to *Section 6.4 Infiltration Systems* for a discussion of our findings and recommendations regarding the design of infiltration systems.

3.4 Geologic and Seismic Hazards

3.4.1 Seismic Shaking

We evaluated potential seismic shaking at the site using data obtained from the U.S. Geologic Survey (USGS) Seismic Design Maps. The expected peak bedrock acceleration having a 2 percent probability of exceedance in 50 years (2,475-year return period) is 0.38 g. This value represents the peak acceleration on bedrock beneath the site and does not account for ground motion amplification due to site-specific effects. The peak ground acceleration (PGA) is determined by applying a site class factor to the peak bedrock acceleration. The PGA accounting for site amplification is PGA_M = 0.462 g. Refer to *Section 3.4.2 Site Classification* for a discussion of ground motion amplification.

We obtained a deaggregation of the seismic sources contributing to the expected peak bedrock acceleration shown above from the National Seismic Hazard Mapping Project website (USGS 2021a). Seismic sources contributing to this potential ground shaking include the shallow crustal faults of the Portland Hills fault system and the Cascadia Subduction Zone (CSZ) megathrust and intraplate sources. The data indicated that the “modal source” for shaking at the site at all potential periods of interest (0.0 to 2.0) is a magnitude 9.0+ earthquake epicentered at the CSZ approximately 94 kilometers from the site. The modal source generally signifies the earthquake with the highest contribution to the site earthquake hazard, in this instance a rupture along the CSZ.

3.4.2 Site Classification

The “Site Class” is a designation used by the 2018 International Building Code (IBC) and ASCE 7-16 to quantify ground motion amplification. The classification is based on the stiffness in the upper 100 feet of soil and bedrock materials at a site. The Washington State Department of Natural Resources (WSDNR) *Geologic Information Portal* (WSDNR 2021) maps the portion of the site to be developed as Site Class B. This is likely due to the mapping of shallow bedrock that is typically encountered in the area. However, during our explorations we did not encounter bedrock, and per section 3.3.3, we anticipate that bedrock is greater than 15 feet bgs. Therefore, due to the thick soil cover at our site, we recommend Site Class D.

3.4.3 Liquefaction

Liquefaction is a phenomenon caused by a rapid increase in pore water pressure that reduces the effective stress between soil particles, resulting in the sudden loss of shear strength in the soil. Granular soils, which rely on interparticle friction for strength, are susceptible to liquefaction until the excess pore pressures can dissipate. Sand boils and flows observed at the ground surface after an earthquake are the result of excess pore pressures dissipating upwards, carrying soil particles with the draining water. In general, loose, saturated sand soils with low silt and clay contents are the most susceptible to liquefaction. Silty soils with low plasticity are moderately susceptible to liquefaction or cyclic softening under relatively higher levels of ground shaking. For any soil type, the soil must be saturated for liquefaction to occur.

Based on the clay-rich and generally stiff nature of the site soils, we consider the liquefaction potential of the site to be low.

3.4.4 Surface Fault Rupture

The nearest mapped active fault is the middle to late Quaternary Lacamas Lake Fault, located approximately 2.3 miles from the site (USGS 2021b). Therefore, we consider the risk of surface fault rupture at the site to be low.

3.4.5 Earthquake-Induced Landsliding/Lateral Spreading

Based on the stiff to hard nature of the site soils, it is our opinion the potential for earthquake-induced landsliding and lateral spreading is very low.

4.0 CONCLUSIONS

Based on our explorations, testing, and analyses, it is our opinion that the site is suitable for the proposed development, provided the recommendations in this report are included in design and construction. We offer the following general summary of our conclusions.

- The site is generally blanketed by an approximately 6- to 8-foot-thick zone of medium stiff to stiff clay. The underlying soils generally consist of hard/very dense gravelly clay and clayey gravel. The soils are residual materials derived from the weathering of deeper bedrock materials and contains cobbles. Our explorations did not encounter intact bedrock to the maximum depth explored (16 feet), however, based on nearby explorations localized zones of bedrock may be encountered at shallower depths.

- The presence of cobbles (and potentially intact rock at depth) may make excavation difficult, particularly below 10 feet bgs.

- The site soils are clayey and moist, and will be easily disturbed during construction. The use of wet weather earthwork practices will likely be required at all times.

- A thick layer (up to 16 inches) of soft, organic-rich topsoil is present across the site. This material will need to be removed prior to the placement of fills, slabs, pavements, and/or foundations.

- The regional groundwater table is deep, though infiltrating surface water perches at shallow depth in the fine-grained residual soils. Our explorations encountered perched water at depths of 2 to 7 feet bgs. The project design should account for subsurface drainage systems to intercept perched water which may emerge from cut slopes or pads in cut areas, while the contractor should be prepared to encounter perched water across the site during construction.

- The site soils are suitable for support of structural improvements using conventional spread footing foundation systems.

- The site soils are clayey and are not suitable for infiltration of stormwater.

The following sections present our specific recommendations for structural and earthworks components of the project.

5.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

5.1 Foundation Support Recommendations

5.1.1 General

Proposed structures (e.g. buildings, fuel canopies, trash enclosures, and other miscellaneous features) may be supported by conventional spread footings that bear on native soils or new fill. If undocumented fill, organic soils, or soft soils are encountered below foundations or slabs, then such materials should be removed and/or recompacted. New fill should be placed and compacted to a dense condition per *Section 8.0 Earthwork Recommendations* of this report.

The following recommendations are based on the assumption that maximum structural loads will be up to 75 kips for column footings and 2.5 kips per linear foot for continuous wall footings. If structural loads are greater, then we should be contacted to verify that our recommendations are appropriate.

5.1.2 Dimensions and Design Criteria

Spread footings may be designed using an allowable bearing pressure of 3,000 pounds per square foot (psf) for footings in native soil or on newly placed fill material. Continuous strip footings should have a minimum width of 1.25 feet, while isolated footings should have a minimum dimension of 2.0 feet. The bottom of perimeter footings should extend at least 16 inches below the adjacent exterior grade.

The bearing value provided above represents a net bearing pressure; the weight of the footings 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 may be increased by one-third for short-term loads, such as wind or seismic forces.

5.1.3 Lateral Resistance

Lateral loads on footings can be resisted by passive earth pressures on the sides of footings and by friction on the bearing surface. We recommend that passive earth pressures be calculated using an equivalent fluid density of 300 pounds per cubic foot (pcf). We recommend using a friction coefficient of 0.35 for foundations placed on native soil or 0.45 for foundations on a minimum 6-inch-thick aggregate base subgrade. The passive earth pressure and friction components may be combined, provided the passive component does not exceed two-thirds of the total. The lateral resistance values do not include safety factors.

5.1.4 Settlement

We estimate that total post-construction settlements should be less than approximately 1 inch, with differential settlement of half that amount between adjacent columns or over a 50-foot span for strip footings.

5.1.5 Foundation Subgrade Preparation

Footing excavations should expose competent native soil or new engineered fill. If undocumented fill, soft soils, or organic materials are encountered, these materials should be removed and/or reworked (e.g., organics and debris removed and then recompacted). Refer to *Section 8.0 Earthwork Recommendations* of this report for guidelines related to the placement of structural fill at the site.

Prior to the placement of reinforcing steel in the footing excavations, all loose or disturbed soils should be removed. If water infiltrates and pools in the excavation, the water, along with any disturbed soil, should be removed before placing the reinforcing steel. If construction is undertaken during periods of rain, we recommend that a layer of imported granular material or a lean concrete mud mat be placed over the base of footing excavations, as water will tend to perch/pond within excavations. The granular material or lean concrete reduces subgrade disturbance from standing water and from foot traffic during forming and tying of reinforcing steel. Typically, 3 to 4 inches of granular material that is lightly compacted until well keyed provides sufficient protection from disturbance.

We recommend that Hart Crowser observe all foundation excavations before placement of aggregate or mud mat base to determine that bearing surfaces have been adequately prepared and that the soil conditions are consistent with those anticipated during design.

5.2 Canopy Foundation Support Recommendations

5.2.1 General

It is our understanding that drilled shafts or formed column (e.g., excavated and formed with sonotube) systems are the preferred foundation type to support the proposed fueling station canopy structure. If desired, spread footings as noted in the prior section can also be used to support the canopies. It is also our understanding that typical design axial downward loads for the canopy structure are approximately 25 kips per column but that uplift forces generally control design. Shaft foundations are typically 4 to 5 feet in diameter and generally 5 to 8 feet deep, depending upon subsurface conditions and design loads.

5.2.2 Axial Capacity

Shafts/columns that are founded in the upper residual soils (approximately 8 feet) may derive their support by side friction or end bearing. An allowable skin friction of 550 psf may be use or an end bearing of 3,500 psf. However, the upper 2 feet of the shaft should be ignored for both uplift and downward loads. If shafts/columns extend to the hard/very dense residual soil found approximately 8 feet below grade, an allowable end bearing of 10,000 psf may be assumed.

Uplift forces can also be resisted by the weight of the shafts. The full weight of the pier can be used in uplift calculations without application of a safety factor.

5.2.3 Lateral Resistance

Lateral loads on shafts can be resisted by passive earth pressures on the sides of shafts. We recommend that passive earth pressures be calculated using an equivalent fluid density of 300 pcf. Below a depth of 8 feet, a passive resistance of 500 pcf may be assumed. The passive resistance for individual shafts can be applied over 2 projected shaft diameters. We recommend that the upper 2 feet of the shaft be ignored for passive resistance. These passive resistance values do not have a factor of safety applied.

5.2.4 Settlement

Shafts designed and constructed as recommended are expected to experience static settlements of less than 1 inch under static loading for shafts design via frictional resistance, and less than 0.5 inch for end bearing shafts. Differential settlements of up to one half of the total settlement magnitude can be expected between adjacent footings supporting comparable loads.

5.2.5 Construction Considerations

The site soils are fine grained and shaft excavations should generally remain stable for short periods of time. However, if groundwater seepage is present, then the soils will tend to cave and slough in unsupported excavations. The contractor should be prepared to use casing if perched water is present. If seepage builds up in the base of the shaft excavation, then we recommend that a minimum 6 inch layer of clean, crushed rock be placed in the base of the shaft. Also, perched water that collects at the base of an excavated shaft should be removed just prior to the placement of concrete.

5.3 Floor Slabs

Satisfactory subgrade support for concrete slabs supporting up to 175 psf areal loading can be obtained from the new structural fill or native subgrade prepared in accordance with *Section 8.0 Earthwork Recommendations*.

A minimum 6-inch-thick layer of aggregate base should be placed over the prepared subgrade to assist as a capillary break. Aggregate base material placed directly below the slab should be 3/4- to 1-inch maximum size and have less than 5 percent fines. 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.

Slabs should be reinforced according to their proposed use and per the structural engineer's recommendations. Load-bearing concrete slabs may be designed assuming a modulus of subgrade reaction, k, of 200 pounds per square inch per inch, provided the site is prepared as recommended in this report.

In areas where structures are constructed over "cut" pads, it may be prudent to install a subslab drainage system to intercept seepage which may emanate from the subgrade. Refer to Section 6.3 Subsurface Drainage for additional discussion.

5.4 Seismic Design

We anticipate that seismic design will be in accordance with the 2018 IBC and ASCE 7-16 requirements. We obtained seismic design parameters from the U.S. Geologic Survey (USGS) Seismic Design Maps for

Latitude 45.591 and Longitude -122.454 for the 2,475-year return period. The parameters provided in Table 2 are appropriate based on the assumption that ASCE 7-16 Chapter 11.4.7 Exceptions are applicable.

Table 2 – Seismic Design Parameters

Parameter	Value
Site Class	D
Spectral Response Acceleration, S_s	0.838 g
Spectral Response Acceleration, S_1	0.360 g
Site Coefficient, F_a	1.165
Site Coefficient, F_v	1.940 [a]
Spectral Response Acceleration (Short Period), S_{DS}	0.651 g
Spectral Response Acceleration (1-Second Period), S_{D1}	0.465 [a]
Highest Period of Peak Spectral Acceleration, 1.5Ts (s)	1.071 [b]
Maximum Considered Earthquake Peak Ground Acceleration, PGA	0.378 g
Site Coefficient, F_{PGA}	1.222
Site Modified PGA, PGA_M	0.462 g

Notes:

a. Site Class D with S_1 greater than or equal to 0.2 shall have a site-specific ground motion hazard analysis shall be performed unless excepted per ASCE 7-16 Section 11.4.8. Values in the table include all relevant exceptions and assume that no base isolation will be included in design of the foundations. F_v provided for calculation of T_s only.

b. Per exception 2 of ASCE 7-16, Section 11.4.8, provided the structure will not include base isolation, structures on Site Class D with S_1 greater than or equal to 0.2, the seismic response coefficient C_s is determined by Eq. (12.8-2) for values of $T \leq 1.5T_s$ and taken as equal to 1.5 times the value computed in accordance with either Eq. (12.8-3) for $T_L \geq T > 1.5T_s$ or Eq. (12.8-4) for $T > T_L$. The value shown includes the 1.5 multiplier for T_s.

5.5 UST Design Considerations

The USTs will be installed not be installed below the regional groundwater table, however, due to the presence of shallow perched seepage, it is our opinion that there is a high potential for UST excavations to fill up with perched water. Therefore, USTs should be designed to resist uplift forces. Uplift forces can be resisted by using gravity loads and anchors (e.g., thick concrete support slabs) or uplift resistance from deep foundation elements. If the use of deep foundation elements is desired for resistance to uplift, then additional analysis will be required.

5.6 Permanent Retaining Structures

Permanent retaining walls will be required to retain the slopes on the eastern and northern edges of the property, as well as select locations within the interior parking areas. Per City of Camas regulations, the maximum exposed retaining wall height allowed is 6 feet; therefore, terraced retaining wall will likely be required. We anticipate the terraced retaining walls will have a maximum combined height of approximately 12 feet. The retaining walls may consist of cast-in-place (CIP) concrete walls or modular block walls founded on native soils, imported borrow, or a layer of aggregate base over native soils. The walls should be designed and constructed in accordance with the following recommendations.

5.6.1 CIP Walls

5.6.1.1 Earth Pressures

CIP concrete walls should be designed to resist wall backfill earth pressures as shown in Table 3. The designer should also include hydrostatic forces, as appropriate.

Table 3 – CIP Wall Earth Pressures

Retained Material	Condition	Horizontal Earth Pressure Coefficient	Lateral Pressure
Native Soil	Active – level backslope	$K_{\alpha} = 0.41$	45 pcf
	Active – 2H:1V backslope	$K_{\alpha} = 0.85$	94 pcf
	At-Rest	$K_0 = 0.58$	64 pcf
General Structural Fill	Active – level backslope	$K_{\alpha} = 0.28$	35 pcf
	Active – 2H:1V backslope	$K_{\alpha} = 0.44$	55 pcf
	At-Rest	$K_0 = 0.47$	59 pcf
Select Structural Fill	Active – level backslope	$K_{\alpha} = 0.24$	31 pcf
	Active – 2H:1V backslope	$K_{\alpha} = 0.35$	45 pcf
	At-Rest	$K_0 = 0.41$	54 pcf
All Backfill Seismic Surcharge	Active – level backslope	-	10H psf
	Active – 2H:1V backslope		15H psf
	At Rest	-	16H psf

Notes:

a. Native Soil assumed to have a unit weight of 110 pcf and a friction angle of 25 degrees.
b. General Structural Fill assumed to have a unit weight of 125 pcf, have a friction angle of 32 degrees, and be placed and compacted per *Section 8.0 – Earthwork Recommendations.*
c. Select Structural Fill assumed to have a unit weight of 130 pcf, have a friction angle of 36 degrees, and meet the specifications provided in WSS 9-03.12(2) – Gravel Backfill or WSS 9-03.12(4) – Gravel Backfill for Drains.
d. For the seismic condition, it was assumed that the active condition will be reached during the event as per the Mononobe-Okabe method. The seismic pressure should be modeled as a rectangular pressure centered at a height of 0.6 of the wall's height.
e. For intermediate backslopes between level and 2 horizontal to 1 vertical (2H:1V), linearly interpolate between the values provided.

For walls that can move at least 0.1 percent of its height (e.g., yielding wall), the design should use active pressures. Where walls are restrained from moving this distance, then they are considered "non-yielding" and should be designed with at-rest pressures.

The walls should be designed to resist surcharge loads from adjacent footings, equipment, materials, and vehicular loads placed within a 1H:1V projection from the base of wall.

5.6.1.2 CIP Foundations

The foundations for gravity retaining walls should be designed in accordance with *Section 5.1 Foundation Support Recommendations*.

5.6.2 Modular Block Walls

5.6.2.1 Design Parameters

Modular block walls can consist of geogrid-reinforced mechanically stabilized earth (MSE) walls or gravity block walls. Modular block wall design should be based on the soil parameters presented in Table 4.

Table 4 – SE and Modular Block Wall Design Parameters

Material	Unit Weight, γ (pcf)	Friction Angle, ϕ (degrees)	Cohesion, c (psf)
Reinforced Zone Fill	130	36	0
Retained Soil (*In Situ*)	110	25	50
Retained Soil (New Fill)	130	36	0
Foundation Soil (*In Situ*)	115	26	50
Foundation Soil (New Fill)	125	34	0

The “reinforced zone fill” used for backfill within the geogrid zone behind the MSE blocks should meet the specifications provided in Washington State Department of Transportation (WSDOT) *Standard Specifications* (WSS) 9-03.14(4) – Gravel Borrow for Geosynthetic Retaining Wall. “Foundation Soil (New Fill)” is new mass fills or an area of native soil that is overexcavated and replaced with new fill per Section 8.6 Structural Fill and Backfill. “Foundation soil (*in situ*)” should be competent native soils evaluated by Hart Crowser or their representative.

For the seismic evaluation of modular block retaining walls, the designer may assume an allowable displacement of 4 inches during seismic shaking. The determination of a horizontal seismic coefficient, k_h, should be based on the PGA adjusted for site class (PGA_M from Table 2). The vertical acceleration coefficient, k_V, may be assumed to be 0.

Modular block walls should be constructed in general accordance with the specifications provided in WSS 6-13 – Structural Earth Walls. In general walls over 4 feet tall will require reinforcing geosynthetic that has a length equal to approximately 70 percent of the wall height. Terraced retaining walls will need to be evaluated for global stability, though in general the reinforcing for the lower wall is sized as though it is the lower portion of a wall with a total height equal to the combined wall height. However, this will vary depending upon the spacing and height of the walls.

5.6.2.2 Modular Block Wall Foundations

The design of modular block wall foundations should be based on the soil parameters presented in Table 4. In order to satisfy global stability, the base of any block wall should be embedded no less than 18 inches below the lowest adjacent grade where there are slopes steeper than 4H:1V below the slope. Where there are no slopes below the walls, the base of block wall foundations should extend a minimum of 12 inches below lowest adjacent grade.

If the walls are founded on native soils, there is a potential that localized zones of soft, loose, or organic material may be present. Localized removal and replacement of such material may be required.

All walls should be underlain by a minimum 6-inch-thick layer of compacted gravel. The gravel pad should extend at least 6 inches in front and behind the blocks.

5.6.3 Wall Drainage and Backfill

The above design parameters have been provided assuming that back-of-wall drains will be installed to prevent hydrostatic pressures above the groundwater table.

Unless the retaining walls are designed to resist earth pressures from native soils (as noted above), the backfill material placed behind the walls and extending a horizontal distance equal to at least half of the height of the retaining wall should consist of select granular retaining wall backfill.

A minimum 12-inch-wide zone of drain rock, extending from the base of the wall to within 6 inches of finished grade, should be placed against the back of all retaining walls. Perforated collector pipes should be embedded at the base of the drain rock. The drain rock should meet the requirements provided in *Section 8 Earthwork Recommendations*. 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.

The wall backfill should be compacted to a minimum of 95 percent of the maximum dry density, as determined by ASTM D 1557. However, backfill located within a horizontal distance of 3 feet from retaining walls should only be compacted to approximately 90 percent of the maximum dry density, as determined by ASTM D 1557. Backfill placed within 3 feet of the wall should be compacted in lifts less than 6 inches thick using hand-operated tamping equipment (e.g., jumping jack or vibratory plate compactors). If flat work (e.g., sidewalks or pavements) will be placed atop the wall backfill, we recommend that the upper 2 feet of material be compacted to at least 95 percent of the maximum dry density, as determined by ASTM D 1557.

Settlements of up to 1 percent of the wall height commonly occur immediately adjacent to the wall as the wall rotates and develops active lateral earth pressures. Consequently, we recommend that construction of buildings directly above retaining walls be postponed at least 4 weeks after backfilling of the wall, unless survey data indicate that settlement is complete prior to that time.

6.0 DRAINAGE DESIGN RECOMMENDATIONS

6.1 Temporary Drainage

During mass grading 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 building site, the contractor should keep all footing excavations and building pads free of water.

6.2 Surface Drainage

The finished ground surface around buildings should be sloped away from their foundations at a minimum 2 percent gradient for a distance of at least 5 feet. Downspouts or roof scuppers should discharge into a storm drain system that carries the collected water to an appropriate stormwater system. They should not be attached to footing or subslab drains. Trapped planter areas should not be created adjacent to buildings without providing means for positive drainage (i.e., swales or catch basins).

We note that for evaluation of pre- and post-development runoff from the site the Western Washington Hydrology Model for Clark County (WWHMCC) as discussed in OTAK (2010) has identified the site soils (Powell silt loam) as being Clark County Hydrologic Soil Group (SG) 3. The WWHMCC notes that soils with mapped permeability rates of 0.2 to 0.63 inches/hour were mapped as SG 3, while soils with permeability rates less than 0.06 inches/hour were mapped as SG 4. The on-site Powell silt loam soils have an intermediate mapped permeability of 0.06 to 0.2 inches per hour. They were classified as SG 3 based on "…(*Soil Survey*) information indicating that this soil is moderately well drained with a moderate water capacity." However, based on our field infiltration testing indicating little to no infiltration, our laboratory testing indicating the soil is a clay, and the presence of shallow perching water, **the soils at the site are more appropriately classified as SG 4.**

6.3 Subsurface Drainage

As notes previously perched water and shallow seepage was encountered at the site. This is a common phenomenon in the project vicinity. Therefore, we recommend the use of subsurface drainage systems in "cut" areas, including in cut slopes and beneath structures in cut areas. The subsurface drainage system can consist of perimeter footing drains, subslab drains, or mid-slope subdrains, depending on the condition exposed in the field.

We also note that the use of irrigation and improper maintenance of surface drainage gradients adjacent to buildings can often result in adverse conditions which direct irrigation or surface runoff towards buildings. Because of the impermeable nature of the site soils, it would be prudent though not required, to install a perimeter footing drainage system around the proposed buildings. Alternatively, if a subslab radon barrier system is installed beneath slabs, then subdrainage can be incorporated into that system.

We recommend that Hart Crowser be provided an opportunity to review proposed grading plans and to identify areas where subdrainage may be appropriate.

We note that the discharges for subsurface drainage systems should not be tied directly into the stormwater drainage system unless mechanisms are installed to prevent backflow.

6.3.1 Footing Drain or Slope Subdrain

Where seepage is anticipated or identified on slopes or where a footing drain is to be installed around the perimeter of a building, a subdrain should be installed. The subdrain system should consist of a filter fabric-wrapped, drain rock-filled trench that extends at least 12 inches below the lowest adjacent grade adjacent to buildings or 2 feet below grade on slopes. A perforated pipe should be placed at the base to collect water that gathers in the drain rock. The drain rock and filter fabric should meet specifications outlined in *Section 8.6 - Structural Fill and Backfill*.

6.3.2 Subslab Drain

A subslab drain is similar to a subdrain, but instead of a trench it consists of a layer of drain rock covering the subgrade beneath a slab. This rock blanket can replace the aggregate base required beneath slabs. Also, subslab radon collection systems can act as a blanket drain if properly detailed for drainage.

In general, subslab drains will include an 8- to 12-inch-thick layer of clean drain rock underlain by a layer drainage geotextile. Perforated drainage pipes should be embedded in the drain rock to collect water which gathers in the subgrade. The drain rock should be covered with a vapor barrier. Placement of a thin sand layer over the drain rock is often considered, though its use should be reviewed with the architect and flooring manufacturers.

6.4 Infiltration Systems

The results of on-site field infiltration testing are described in Section 3.3.5 Infiltration Testing. In general, we found that the soils are not suitable for infiltration with unfactored hydraulic conductivity values between 0.0 and 0.15 inches/hour. With a minimum soils correction factor of 2, the maximum allowable design infiltration rate would be 0.0 to 0.08 inches/hour. However, due to the variability of the test results, a higher soils correction factor would be identified, and the allowable design rate would likely be 0.04 inches/hour or less. As such, we consider the use of infiltration systems to be infeasible.

7.0 PAVEMENT DESIGN RECOMMENDATIONS

7.1 General

Our pavement design recommendations for the commercial pavement areas include options for conventional flexible AC or rigid PCC pavement. Our design thicknesses assume that new pavements will be supported by a subgrade prepared in conformance with *Section 8.0 Earthwork Recommendations* of this report.

We include our assumptions regarding traffic in the section below. If any of these assumptions are inaccurate, please contact us to develop updated recommendations.

7.2 Pavement Design Assumptions

We made the following assumptions regarding, and used the following parameters for, the design of the pavement sections.

- Traffic to the site will include the following:
  - Up to 5,000 passenger vehicles and light trucks per day distributed over several drive aisles
  - Up to 3 single unit delivery trucks (FHWA Class 5 or 6) per day
  - Up to 2 full size truck (FHWA Class 9) per day
  - Up to 5 fuel deliveries by double tanker truck (FHWA Class 12 or 13) per week

- Based on the traffic loading noted above and a 2 percent annual growth rate, we estimate the 20-year design life equivalent single-axle loads (ESALs) to be approximately 100,000 for drive aisles.
- A resilient modulus of 8,000 pounds per square inch (psi) was estimated for a subgrade that has been moisture conditioned and compacted in conformance with *Section 8.0 Earthwork Recommendations* of this report.
- A resilient modulus of 25,000 psi was estimated for the base rock.
- Initial and terminal serviceability indices of 4.2 and 2.5, respectively.
- Reliability and standard deviation of 85 percent and 0.45, respectively.
- Structural coefficients of 0.45 and 0.12 for the AC and base rock layers, respectively.
- Minimum moduli of rupture and elasticity of 570 and 3,600,000 psi, respectively, for conventional PCC.
- Minimum compressive strength of 4,000 psi for conventional PCC.

Also, construction traffic should be limited to non-building, unpaved portions of the site or haul roads. Construction traffic should not be allowed on new pavements. If construction traffic is to be allowed on newly constructed road sections, an allowance for additional traffic will need to be made in the design pavement section.

7.3 Pavement Sections

The AC pavement sections in Table 5 are minimum recommended material thicknesses.

Table 5 –AC Pavement Sections

Traffic Basis	AC Thickness (inches)	Aggregate Base Thickness (inches)
Drive Aisles	3.0	8.0
Parking Stalls	2.5	6.0

The PCC pavement sections in Table 6 include both reinforced and unreinforced sections and are valid for all of the traffic levels. The unreinforced PCC pavement would most typically be used in areas that receive “pass through” traffic, such as decorative cross-walks, etc.; whereas, the reinforced PCC pavement would typically be used as areas with extensive vehicular braking and increased long-term performance requirements, such as at the fueling stations.

Table 6 –PCC Pavement Sections

PCC Pavement Type	PCC Thickness (inches)	Aggregate Base Thickness (inches)
Unreinforced	5.0	6.0
Reinforced	6.0	6.0

7.4 Pavement Materials

7.4.1 Flexible AC

Flexible AC should be 1/2-inch hot mix asphalt in conformance with the specifications provided in Washington State Department of Transportation (WSDOT) *Standard Specifications* (WSS) 5 04 – Hot Mix Asphalt and WSS 9 03.8 – Aggregates for Hot Mix Asphalt (WSDOT 2020). The AC binder should be PG 64-22 Performance Grade Asphalt Cement according to WSS 9-02.1(4) – Performance Graded Asphalt Binder. The AC should be placed with a minimum lift thickness of 1.5 inches and be compacted to at least 91 percent of Rice Density of the mix, as determined in accordance with American Society for Testing and Materials (ASTM) D 2041.

7.4.2 Rigid PCC

Rigid PCC pavement should meet the specifications provided in WSS 5 05 – Cement Concrete Pavement. The PCC should have a minimum compressive strength of 4,000 psi and nominal maximum aggregate size of 1.5 inches. The PCC should be constructed with a maximum joint spacing of 15 feet.

Unreinforced slabs should be interlocked at contraction joints (e.g., continuous slab with no dowels), although dowels should be used at construction and expansion joints. Reinforced PCC should have No. 4 bars at 24 inches on center, each way at the mid-depth of the PCC.

7.4.3 Aggregate Base

Imported granular material used as base aggregate (base rock) should meet the criteria specified in *Section 8.6 Structural Fill and Backfill* of this report. The base aggregate should be compacted to not less than 95 percent of the maximum dry density as determined by ASTM D 1557.

8.0 EARTHWORK RECOMMENDATIONS

8.1 General

Based on the sloping nature of the site, we anticipate that grading mass grading will be required with mass cuts on the order of 4 to 8 feet deep and mass fills of approximately 15 feet thick. Additionally, deeper excavations (up to approximately 20 feet) will be required for installation of fuel underground storage tanks (USTs) and utilities. We recommend that earthwork activities be conducted in accordance with the WSS (WSDOT 2021).

8.2 Site Preparation

8.2.1 Clearing and Grading

Initial site preparation and earthwork operations will include stripping and grading to establish subgrade elevation for improvements. We estimate the depth of soft, organic-rich material to be stripped is between 11 and 16 inches (average 14 inches). Actual stripping depths should be based on field observations at the time of construction. Stripped material should be transported off-site for disposal or stockpiled for use in landscaped areas.

Trees and their root balls should be grubbed out to the depth of significant roots, which could exceed 3 to 5 feet bgs for the tall conifer trees. Depending on the methods used to remove the 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.

Any cavities resulting from removal of the unsuitable soils and/or previous establishments shall be cleared of debris and backfilled with structural fill.

8.2.2 Subgrade Preparation and Evaluation

Following stripping, site preparation, and rough grading, the suitability of the subgrade should be evaluated by proof rolling with a fully loaded dump truck or similar heavy rubber-tired construction equipment to identify any remaining soft, loose, or unsuitable areas. The proof roll should be conducted prior to placing new fill. Proof rolling should be observed by a representative of Hart Crowser who would evaluate the suitability of the subgrade and identify areas of yielding that are indicative of soft or loose soil. During wet weather or when the exposed subgrade is wet or unsuitable for proof rolling, the prepared subgrade should be evaluated by observing excavation activity and probing with a steel foundation probe. Observations and probing should be performed by Hart Crowser.

If soft or loose zones are identified during proof rolling or probing, these areas should be excavated to the extent indicated by Hart Crowser and replaced with structural fill.

If site preparation activities cause excessive subgrade disturbance, replacement with imported structural fill may be necessary. Disturbance to the subgrade should be expected if site preparation and earthwork are conducted during periods of excessive wet weather and/or when the moisture content of the surficial soil exceeds optimum.

8.3 Wet Soil/Wet Weather Construction

The near-surface site soils generally consist of silt, silty clay and clay. These materials are highly susceptible to becoming disturbed when they are wet or heavily trafficked. If not carefully executed, site preparation, utility trench work, and pavement construction can create extensive soft areas, and significant repair costs can result. Earthwork planning should include considerations for minimizing subgrade disturbance.

We anticipate that wet soils/wet weather earthwork practices will need to be employed at all times, regardless of the season. However, if earthwork is completed during rainy weather or during periods of high moisture content in the soil, then significant difficulties in trafficking and placement of fill will occur. In that case, the subgrade may need to be stabilized by removal and replacement with geotextile stabilization fabric and granular fill or cement amending of the *in situ* soil.

8.3.1 Removal and Replacement

Soft and/or wet soils can be removed and replaced with imported granular fill. Stabilization rock and possibly select granular fill can be used as the initial lift of structural fill over a softer subgrade. The native soil subgrade would be covered by a stabilization geotextile fabric and then covered with 12 to 18 inches of stabilization rock/select granular fill, depending upon the subgrade condition, finished grades, and the contractor's means and methods. This may require partial removal and replacement to meet design grades.

8.3.2 Soil Amendment with Cement

As an alternative to the use of imported granular material for structural fill, the on-site soils can be amended with Portland cement to obtain suitable support properties. Portland cement-amended soils are hard and have low permeability. They should not be used if runoff during construction cannot be properly controlled.

Treatment depths for subgrades, haul roads, and staging areas are typically on the order of 12, 16, and 12 inches, respectively. To protect the cement-treated surfaces from abrasion or damage, the finished surface is typically covered with 4 to 6 inches of imported granular material.

The actual thickness of the amended material, percentage of added cement, and thickness of imported granular material will depend on the anticipated subgrade usage, as well as the contractor's means and methods, and accordingly, should be the contractor's responsibility.

If the subgrade is be amended with cement, refer to *Section 8.6.7 Soil Amendment with Cement* for additional discussion.

8.3.3 Haul Roads

One method for minimizing subgrade disturbance during construction is through the use of temporary haul roads and staging areas. Based on our experience, between 12 and 18 inches of imported granular material is generally required to construct staging areas and haul roads that will support typical construction traffic. However, the actual thickness will depend on the contractor's means and methods, and accordingly, should be the contractor's responsibility. Additionally, a geotextile fabric may be placed as a barrier between the subgrade and imported granular material in areas of repeated construction traffic to provide separation between the imported rock and native soils. The imported granular material and geotextile fabric should meet the specifications in *Section 8.6 Structural Fill and Backfill* of this report.

8.4 Excavation

8.4.1 General Excavations

Site soils are generally medium stiff to stiff within expected excavation depths. It is our opinion that conventional earthmoving equipment in proper working condition should be capable of making necessary general excavations for utilities, footings, and other earthwork. However, difficult excavation may be encountered where deeper, harder residual soils with cobbles or intact rock are encountered on site. The earthwork contractor should be responsible for providing equipment and following procedures as needed to excavate the site soils as described in this report.

Permanent slope excavations should have a maximum gradient of 2 horizontal to 1 vertical (2H:1V).

8.4.2 Temporary Excavation Stability

Due to the potential for perched water near the ground surface, even shallow excavations will have a high susceptibility to sloughing, raveling, or caving. Open excavation techniques may be used for temporary excavations above the groundwater table. For planning purposes only, we expect that temporary cut slopes (but not trench excavations) may be excavated at an angle of 1H:1V or flatter. However, because of the variables involved, actual slope angles required for stability in temporary cut areas can only be estimated before construction. We recommend that stability of the temporary slopes used for construction be the responsibility of the contractor, since the contractor is in control of the construction operation and is continuously at the site to observe the nature and condition of the subsurface.

All temporary soil cuts associated with site excavations should be adequately sloped back to prevent sloughing and collapse, in accordance with Department of Occupational Safety and Health (DOSH) Chapter 296-155 Washington Administrative Code (WAC) Part N Excavation, Trenching and Shoring Occupational Safety and Health Administration (OSHA) guidelines.

The stability and safety of cut slopes depend on a number of factors, including:

- The type and density of the soil
- The presence and amount of any seepage
- Depth of cut
- Proximity and magnitude of the cut to any surcharge loads, such as stockpiled material, traffic loads, or structures
- Duration of the open excavation
- Care and methods used by the contractor

According to DOSH guidelines, we interpret the existing site soils as Type B.

It is the responsibility of the contractor to ensure that the excavation is properly sloped or braced for worker protection, in accordance with DOSH guidelines. To assist with this effort, for planning purposes only, we make the following recommendations regarding temporary excavation slopes.

- Protect the slope from erosion with plastic sheeting for the duration of the excavation to minimize surface erosion and raveling.
- Limit the maximum duration of open excavation to the shortest time period practicable.
- Place no surcharge loads (equipment, materials, etc.) within 10 feet of the top of any excavation or slope.

More restrictive requirements may apply depending on specific site conditions, which should be continuously assessed by the contractor.

If temporary sloping is not feasible due to site spatial constraints, excavations could be supported by internally braced shoring systems, such as a trench box, slide rail, or other temporary shoring. There are a variety of options available. We recommend that the contractor be responsible for selecting the type of shoring system to use. We note that trench boxes are a safety feature used to protect workers and do not prevent caving. If the excavations are left open for extended periods of time, then caving of the sidewalls may occur. The presence of caved material will limit the ability to properly backfill and compact the trenches. The voids between the trench boxes and the sidewalls of the trenches should be filled with sand or gravel before caving occurs.

8.4.3 Dewatering

As noted previously, groundwater seepage was observed at the site and is therefore likely within the expected depth of excavations and slope cuts at the project. Construction of utilities and mass excavations that extend below groundwater levels will require dewatering or water control systems. Pumping from sumps may be effective in removing water from the bases of trenches and open excavations, but will not prevent or reduce the greater risk of trench wall caving, sloughing, or basal instability caused by seepage.

Excavation or hauling equipment should not track below the groundwater table without dewatering systems in place. Also, fill, topsoil, treatment media, trench backfill, etc. should not be placed in ponded water. Therefore, dewatering points or trenches may be required to prevent water from ponding in excavations during construction. The contractor should be made responsible for temporary drainage of surface water and groundwater as necessary to prevent standing water and/or erosion at the working surface or in excavations.

The bases of excavations may be soft and/or unstable if groundwater is present. If that is the case, then stabilization material may need to be placed at the base of the excavations. Stabilization material should be placed to a minimum thickness of 12 inches or as needed to provide an adequate working surface and should meet the criteria discussed in *Section 8.6 – Structural Fill and Backfill*. The use of a geotextile separation fabric may be necessary below any stabilization material to help prevent the stabilization material from pushing into the unstable base materials.

8.5 Permanent Slopes

Permanent slopes should be completed in accordance with the specifications provided in WSS 2-03 – Roadway Excavation and Embankment and City standards. Permanent slopes up to approximately 6 feet tall are anticipated to be required for the project.

Permanent slopes should not exceed a gradient of 2H:1V. Where soft surficial soils are encountered in the exposed face of cut slopes, they may need to be excavated and replaced with structural fill, as described in *Section 8.2 – Site Preparation.* Also, where seepage is encountered at the face of cut slopes, it will be necessary to install a subdrain to collect the water, as discussed in *Section 6.0 – Drainage*.

Hardscape improvements (e.g., curbs) should be located at least 3 feet from the crest of slopes. The setback for the toe of building and wall foundations should be no less than 10 feet, unless the foundation design and construction takes into consideration soil creep and slope stability. Depending upon the slope gradient and structure setback from the crest of the slope, this requirement will affect the foundation embedment.

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.

8.6 Structural Fill and Backfill

Structural fill should be considered to include subgrade soils beneath buildings, foundations, slabs, and pavements and in other areas intended to support structures or within the influence zone of structures.

Fill should only be placed over a subgrade that has been prepared in conformance with the prior sections of this report. 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 materials and should meet specifications provided in the WSS (WSDOT 2021). A brief characterization of some of the acceptable materials and our recommendations for their use as structural fill are provided below. All materials should be placed and compacted in lifts with maximum uncompacted thicknesses and relative densities as recommended in the tables that follow.

8.6.1 On-Site Soils

On-site, near-surface soils that might be used for fill generally consist of fine-grained, cohesive silt and clay. The materials have variable moisture contents, Atterberg limits, and organic contents. This material is not ideal for use of structural fill and should only be considered if earthwork is being completed during periods of extended dry weather, where the material can be aerated (dried).

If used as structural fill, the on-site materials will need to be moisture conditioned; free of debris, organic materials, and particles over 6 inches in diameter; and meet the specifications provided in WSS 9 03.14(3) – Common Borrow. Topsoil and organic material are not suitable for structural fill

8.6.2 Imported Select Structural Fill

Imported granular material used as structural fill 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. However, the imported granular material should also have a maximum size of 2 inches, be angular and fairly well graded between coarse and fine material, have less than 5 percent by dry weight passing the U.S. Standard No. 200 Sieve, and have at least two mechanically fractured faces.

If soft or loose materials are present, imported fill material may need to be separated from the native subgrade with a layer of subgrade geotextile that meets the specifications provided in WSDOT SS 9-33.2(1) Table 3 – Geotextile for Separation or Soil Stabilization. The geotextile should be installed in conformance with the specifications provided in WSS 2-12 – Construction Geosynthetic.

8.6.3 Aggregate Base

Imported granular material used as aggregate base (base rock) beneath pavements should be clean, crushed rock or crushed gravel and sand that is fairly well graded between coarse and fine. The base aggregate should meet the specifications provided in WSS 9 03.9 – Aggregates for Ballast and Crushed Surfacing, depending upon application. For use beneath building slabs, the base rock should also meet the gradation of WSS 9 03.9(3) – Crushed Surfacing for “Base Course,” although should have less than 5 percent by dry weight passing a U.S. Standard No. 200 Sieve.

For use beneath conventional pavements or footings, the aggregate base should have a maximum particle size of 1 or 1.5 inches, while for use beneath buildings or sidewalk slabs should have a maximum particle size of 0.75 or 1 inch.

Aggregate base should be separated from the native subgrade with a layer of subgrade geotextile that meets the specifications provided in WSS 9-33.2(1) Table 3 – Geotextile for Separation or Soil Stabilization. The geotextile should be installed in conformance with the specifications provided in WSS 2-12 – Construction Geosynthetic. (A separation fabric is not needed where the aggregate base bears on imported fill.)

8.6.4 Drain Rock

Drain rock used for subslab capillary breaks or subsurface drainage systems should consist of clean, crushed drain rock that meets the gradation specifications provided in WSS 9 03.12(4) – Gravel Backfill for Drains or WSS 9 03.1254) – Gravel Backfill for Drywells. However, the materials should have a maximum particle size of 1 inch.

The drain rock should be wrapped in a geotextile fabric that meets the specifications provided in WSS 9 33.2 for drainage geotextiles. The geotextile should be installed in conformance with the specifications provided in WSS 2 12 – Construction Geosynthetic.

8.6.5 Trench Backfill

Trench backfill placed beneath, adjacent to, and for at least 12 inches above utility lines (i.e., the pipe zone) should consist of well graded granular material with a maximum particle size of 1 inch and should meet the specifications provided in WSS 9 03.12(3) – Gravel Backfill for Pipe Zone Bedding and the pipe manufacturer.

Within pavement and slab subgrades, the remainder of the trench backfill up to the subgrade elevation can consist of the above 1-inch material or of granular material with a maximum particle size of 2.5 inches, less than 10 percent by dry weight passing the U.S. Standard No. 200 Sieve, and meeting the specifications provided in WSS 9 03.19 – Bank Run Gravel for Trench Backfill.

8.6.6 Stabilization Material

Imported material that is placed as a stabilization layer for haul roads or staging area should consist of a clean, angular, crushed rock, such as ballast or quarry spalls. The material should have a maximum particle size of 4 inches, a nominal size between 2 and 4 inches, less than 5 percent by dry weight passing the U.S.

Standard No. 4 Sieve, and at least two mechanically fractured faces. The material should be free of organic matter and other deleterious material.

Material meeting the gradations of WSS 9-03.9(2) – Shoulder Ballast, WSS 9-03.12(1)B – Gravel Backfill for Foundations (Class B), WSS 9-03.12(5) – Gravel Backfill for Drains, WSS 9-13.1(2) – Light Loose Riprap, WSS 9-03.12(5) – Gravel Backfill for Drywells, or WSS 9-13.6 – Quarry Spalls is generally acceptable for use. Stabilization material should be placed in lifts between 12 and 18 inches thick and be compacted to a well-keyed condition with a smooth drum roller without using vibratory action.

Stabilization material should be separated from the subgrades with a layer of subgrade geotextile that meets the specifications provided in WSDOT SS 9-33.2(1) Table 3 – Geotextile for Separation or Soil Stabilization. The geotextile should be installed in conformance with the specifications provided in WSS 2-12 – Construction Geosynthetic.

8.6.7 Soil Amendment with Cement

As an alternative to the use of imported granular material for structural fill, an experienced contractor may be able to amend the on-site soils with Portland cement to obtain suitable support properties. Successful use of soil amendment depends on the use of correct mixing techniques, soil moisture content, and amendment quantities. Specific recommendations for soil amending, based on exposed site conditions, can be provided if necessary.

Portland cement-amended soils are hard and have low permeability. These soils do not drain well nor are they suitable for planting. Future planted areas should not be cement amended, if practical, or accommodations should be made for drainage and planting. Moreover, cement amending soil within building areas must be done carefully to avoid trapping water under floor slabs. We should be contacted if this approach is considered. Cement amendment should not be used if runoff during construction cannot be directed away from off-site drainage facilities.

We recommended a 7-day unconfined compressive strength of at least 80 psi. To protect the cement-treated surfaces from abrasion or damage, the finished surface should be covered with 4 to 6 inches of imported granular material. The crushed rock typically becomes contaminated with soil during construction. Contaminated base rock should be removed and replaced with clean rock in pavement areas.

For preliminary planning purposes, we estimate that 4 percent cement (by dry weight) will be required for amending of on-site soils for use as general structural fill. However, where amended soils will be used in the upper 12 to 18 inches of roadway subgrades, haul roads, or staging areas, we estimate the cement may need to be increased to 6 percent particularly during rainy periods. Actual percentages of cement will need to be based on *in situ* soil moisture contents and other field conditions at the time of amendment. The contractor should assuming an *in situ* soil unit weight of 110 pcf when estimating cement volumes.

The actual thickness of the amended material and imported granular material will depend on the anticipated traffic, as well as the contractor's means and methods, and accordingly, should be the contractor's responsibility.

It is not possible to amend soils during heavy or continuous rainfall. Work should be completed during suitable conditions. To prevent strength loss during curing, cement-amended soil should be allowed to cure for a minimum of 4 days prior to access by construction traffic.

In order to use wet on-site soils that would not otherwise be suitable for structural fill, they may be amended and placed as fill over a stabilized subgrade. Consecutive lifts of fill may be treated immediately after the previously lift has been amended and compacted (e.g., the 4-day wait period does not apply). However, where the final lift of fill is a building or roadway subgrade, then the 4-day wait period is in effect.

8.7 Fill Placement and Compaction

Structural fill should be placed and compacted in accordance with the following guidelines.

- Place fill and backfill on a prepared subgrade that consists of firm, inorganic native soils or approved structural fill.

- Place fill or backfill in uniform horizontal lifts with a thickness appropriate for the material type and compaction equipment. Table 7 provides general guidance for lift thicknesses.

Table 7 – Guidelines for Uncompacted Lift Thickness

Compaction Equipment	Guidelines for Uncompacted Lift Thickness (inches)		
	On-Site Soil	Granular and Crushed Rock Maximum Particle Size ≤ 1½ inch	Crushed Rock Maximum Particle Size > 1½ inch
Plate Compactors and Jumping Jacks	4 – 8	4 – 8	Not Recommended
Rubber-Tire Equipment	6 – 8	10 – 12	6 – 8
Light Roller	8 – 10	10 – 12	8 – 10
Heavy Roller	10 – 12	12 – 18	12 – 16
Hoe Pack Equipment	12 – 16	18 – 24	12 – 16

Note: The above table is based on our experience and is intended to serve as a guideline. The information provided in this table should not be included in the project specifications.

- Use appropriate operating procedures to attain uniform coverage of the area being compacted.

- Place fill at a moisture content within approximately 3 percent of optimum as determined in accordance with ASTM D 1557. Moisture condition fill soil to achieve uniform moisture content within the specified range before compacting. Compact fill to the percent of maximum dry densities as noted in Table 8 below.

- Do not place, spread, or compact fill soils during freezing or unfavorable weather conditions. Frozen or disturbed lifts should be removed or properly recompacted prior to placement of subsequent lifts of fill soils.

Table 8 – Fill Compaction Criteria

Fill Type	Percent of Maximum Dry Density Determined in Accordance with ASTM D 1557		
	0 – 2 Feet Below Subgrade	>2 Feet Below Subgrade	Pipe Bedding and Pipe Zone
Mass Fill: fine-grained soils	92	90	-----
Mass Fill: granular materials	95	90	-----
Aggregate Base	95	95	-----
Trench Backfill	95	92	90
Nonstructural Trench Backfill	90	88	-----
Nonstructural Zones	90	88	90

Note: "Nonstructural" areas are only located in landscaping zones, where the potential for localized trench settlement is acceptable to the owner.

During structural fill placement and compaction, a sufficient number of in-place density tests should be completed by Hart Crowser to verify that the specified degree of compaction is being achieved. For structural fill with more than 30 percent retained on the 3/4-inch sieve, Hart Crowser should visually verify proper compaction with a proof roll or other methods.

9.0 CONSTRUCTION OBSERVATIONS

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

We recommend that Hart Crowser be retained to monitor construction at the site to confirm that subsurface conditions are consistent with the site explorations and to confirm that the intent of project plans and specifications relating to earthwork and foundation construction are being met. In particular, we recommend that the foundation and building subgrades; subgrade beneath fills and pavements; and compaction of structural fills and aggregate bases be observed and/or tested by Hart Crowser.

10.0 LIMITATIONS

We have prepared this report for the exclusive use of MAJ Development Corporation and their authorized agents for the proposed Camas Station project in Camas, Washington. Our work was completed in general accordance with our Services Agreement dated November 30, 2021. Our report is intended to provide our opinion of geotechnical parameters for design and construction of the proposed project based on exploration locations that are believed to be representative of site conditions. However, conditions can vary significantly between exploration locations and our conclusions should not be construed as a warranty or guarantee of subsurface conditions or future site performance.

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

Any electronic form, facsimile, or hard copy of the original document (email, text, table, and/or figure), if provided, and any attachments are only a copy of the original document. The original document is stored by Hart Crowser and will serve as the official document of record.

11.0 REFERENCES

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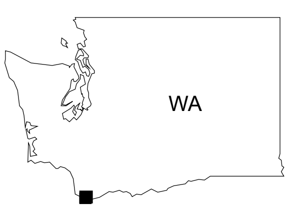
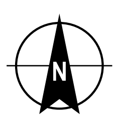
\\haleyaldrich.com\share\pdx_data\Notebooks\0202499-006_MAJ_Camas_Station\Deliverables\Reports\Geotechnical Report\2022_0207_HCHA_136th MAJ Camas Station Geotech Report_F.docx





MAP SOURCE: ESRI
SITE COORDINATES: 45°35'28"N, 122°27'12"W

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CAMAS STATION DEVELOPMENT
NW 16TH AVENUE AND NW BRADY ROAD
CAMAS, WASHINGTON

VICINITY MAP

APPROXIMATE SCALE: 1 IN = 2000 FT
FEBRUARY 2022

FIGURE 1

GIS: \\haleyaldrich.com\share\pdx_data\Notebooks\0202499-006_MAJ_Camas_Station\GIS\Maps\2022_02\202499_006_0001_VICINITY_MAP.mxd - mschweitzer - 2/2/2022 7:32:34 PM

LEGEND

- INFILTRATION TEST
- TEST PIT
- PROPOSED BUILDING FOOTPRINT
- SITE BOUNDARY

NOTES

1. ALL LOCATIONS AND DIMENSIONS ARE APPROXIMATE.
2. BASE MAP SOURCE: "2021-154_SP11.DWG" PROVIDED BY TILAND SCHMIDT, 22 DECEMBER 2021
3. AERIAL IMAGERY SOURCE: NEARMAP, 28 AUGUST 2021

N

0 50 100

SCALE IN FEET

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CAMAS STATION DEVELOPMENT
NW 16TH AVENUE AND NW BRADY ROAD
CAMAS, WASHINGTON

SITE PLAN

FEBRUARY 2022

FIGURE 2

GIS:\\haleyaldrich.com\share\pdx_data\Notebooks\0202499-006_MAJ_Camas_Station\GIS\Maps\2022_02\202499_006_0002_SITE_PLAN.mxd - mschweitzer - 2/2/2022 7:33:56 PM

APPENDIX A
Field Explorations

APPENDIX A
FIELD EXPLORATIONS

General

We evaluated subsurface conditions at the site by advancing six test pits and two infiltration test holes on December 15, 2021. The field explorations were coordinated and overseen by geotechnical staff from Hart Crowser who classified the various soil units encountered, obtained representative soil samples for geotechnical testing, recorded groundwater conditions, and maintained a detailed log of each exploration. Logs of the test pits are included in this appendix. Results of the laboratory testing are indicated on the exploration logs and are included in Appendix B.

Materials encountered in the explorations were classified in the field in general accordance with ASTM Standard Practice D 2488 "Standard Practice for the Classification of Soils (Visual-Manual Procedure)." Disturbed ("grab") samples were collected from sidewalls or excavation spoils during test pit explorations and from the core samples in the push probe boring. Sampling intervals are shown on the exploration logs included in this appendix.

The exploration logs in this appendix show our interpretation of the exploration, sampling, and testing data. The logs indicate the depth where the soils change. Note that the change may be gradual. In the field, we classified the samples taken from the explorations according to the methods presented on the *Figure A- 1 Key to Exploration Logs*. This figure also provides a legend explaining the symbols and abbreviations used in the logs.

The approximate locations of the explorations are shown on Figure 2 of the report. Explorations were located in the field using a mapping grade Trimble GPS unit.

Test Pits

Six test pit explorations, designated TP-1 through TP-6, were performed on December 15, 2021. Test pit explorations were completed using a mini-trackhoe operated by Stratus Corporation of Gaston, Oregon. The explorations were continuously observed by geotechnical staff members from Hart Crowser, and detailed field logs of the test pits were prepared. Disturbed ("grab") samples were collected from sidewalls or excavation spoils during test pit explorations. Sampling intervals are shown on the exploration logs included in this appendix.

Infiltration Testing

We conducted two infiltration tests designated IT-1 and IT-2 at the site adjacent to two test pits. IT-1 was conducted adjacent to TP5, and IT-2 adjacent to TP-3. The tests consisted of single-ring falling head infiltration tests, as referenced in and conducted in general accordance with the procedures in Camas (2016) and as briefly described below.

The primary test pits were excavated to a depth of approximately 6 feet or more below the base of the tests to verify subsurface conditions below the base of the test. The adjacent infiltration test pits were advanced adjacent to each primary test pit and cuttings/or grab samples generated from infiltration holes/pits were observed to verify that subsurface conditions were relatively consistent with the primary test pit excavation.

- At each test location, a 6-inch diameter PVC pipe was placed in the bottom of the test pit. The tip of the pipe was pushed into the soil approximately 6 or more inches to form a seal around the base of the pipe.
- The pipes were filled with water depths roughly corresponding to the anticipated inundation depth of potential infiltration systems and were allowed to saturate. The tests were allowed to saturate for a minimum of approximately four hours or until the draw-down rates had sufficiently stabilized, as described in the test procedure.
- After the saturation period, the infiltration rate was monitored until the rate stabilized.

The results of our infiltration tests are provided in Table 1 of the report. Please refer to the body of the report for a discussion of our findings and recommendations regarding the design of infiltration systems.

Sample Description

Identification of soils in this report is based on visual field and laboratory observations which include density/consistency, moisture condition, grain size, and plasticity estimates and should not be construed to imply field nor laboratory testing unless presented herein. ASTM D 2488 visual-manual identification methods were used as a guide. Where laboratory testing confirmed visual-manual identifications, then ASTM D 2487 was used to classify the soils.

Relative Density/Consistency

Soil density/consistency in borings is related primarily to the standard penetration resistance (N). Soil density/consistency in test pits and probes is estimated based on visual observation and is presented parenthetically on the logs.

SAND or GRAVEL Relative Density	N (Blows/Foot)	SILT or CLAY Consistency	N (Blows/Foot)
Very loose	0 to 4	Very soft	0 to 1
Loose	5 to 10	Soft	2 to 4
Medium dense	11 to 30	Medium stiff	5 to 8
Dense	31 to 50	Stiff	9 to 15
Very dense	>50	Very stiff	16 to 30
		Hard	>30

Moisture

Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, usually soil is below water table

USCS Soil Classification Chart (ASTM D 2487)

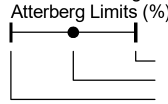
Major Divisions			Symbols: Graph	Symbols: USCS	Typical Descriptions
Coarse Grained Soils More than 50% of Material Retained on No. 200 Sieve	Gravel and Gravelly Soils More than 50% of Coarse Fraction Retained on No. 4 Sieve	Clean Gravels *(<5% fines)*		GW	Well-Graded Gravel; Well-Graded Gravel with Sand
				GP	Poorly Graded Gravel; Poorly Graded Gravel with Sand
		Gravels *(5-12% fines)*		GW-GM	Well-Graded Gravel with Silt; Well-Graded Gravel with Silt and Sand
				GW-GC	Well-Graded Gravel with Clay; Well-Graded Gravel with Clay and Sand
				GP-GM	Poorly Graded Gravel with Silt; Poorly Graded Gravel with Silt and Sand
				GP-GC	Poorly Graded Gravel with Clay; Poorly Graded Gravel with Clay and Sand
		Gravels with Fines *(>12% fines)*		GM	Silty Gravel; Silty Gravel with Sand
				GC	Clayey Gravel; Clayey Gravel with Sand
	Sand and Sandy Soils More than 50% of Coarse Fraction Passing No. 4 Sieve	Sands with few Fines *(<5% fines)*		SW	Well-Graded Sand; Well-Graded Sand with Gravel
				SP	Poorly Graded Sand; Poorly Graded Sand with Gravel
		Sands *(5-12% fines)*		SW-SM	Well-Graded Sand with Silt Well-Graded Sand with Silt and Gravel
				SW-SC	Well-Graded Sand with Clay; Well-Graded Sand with Clay and Gravel
				SP-SM	Poorly Graded Sand with Silt; Poorly Graded Sand with Silt and Gravel
				SP-SC	Poorly Graded Sand with Clay; Poorly Graded Sand with Clay and Gravel
		Sands with Fines *(>12% fines)*		SM	Silty Sand; Silty Sand with Gravel
				SC	Clayey Sand; Clayey Sand with Gravel
Fine Grained Soils More than 50% of Material Passing No. 200 Sieve	Silts			ML	Silt; Silt with Sand or Gravel; Sandy or Gravelly Silt
				MH	Elastic Silt; Elastic Silt with Sand or Gravel; Sandy or Gravelly Elastic Silt
	Silty Clay *(based on Atterberg Limits)*			CL-ML	Silty Clay; Silty Clay with Sand or Gravel; Gravelly or Sandy Silty Clay
	Clays			CL	Lean Clay; Lean Clay with Sand or Gravel; Sandy or Gravelly Lean Clay
				CH	Fat Clay; Fat Clay with Sand or Gravel; Sandy or Gravelly Fat Clay
	Organics			OL/OH	Organic Soil; Organic Soil with Sand or Gravel; Sandy or Gravelly Organic Soil
Highly Organic *(>50% organic material)*				PT	Peat - Decomposing Vegetation - Fibrous to Amorphous Texture

Minor Constituents

	Estimated Percentage
Sand, Gravel	
Trace	<5
Few	5 - 15
Cobbles, Boulders	
Trace	<5
Few	5 - 10
Little	15 - 25
Some	30 - 45

Soil Test Symbols

%F	Percent Passing No. 200 Sieve
AL	Atterberg Limits (%)



CA	Chemical Analysis
CAUC	Consolidated Anisotropic Undrained Compression
CAUE	Consolidated Anisotropic Undrained Extension
CBR	California Bearing Ratio
CIDC	Consolidated Drained Isotropic Triaxial Compression
CIUC	Consolidated Isotropic Undrained Compression
CK0DC	Consolidated Drained k0 Triaxial Compression
CK0DSS	Consolidated k0 Undrained Direct Simple Shear
CK0UC	Consolidated k0 Undrained Compression
CK0UE	Consolidated k0 Undrained Extension
CRSCN	Constant Rate of Strain Consolidation
DS	Direct Shear
DSS	Direct Simple Shear
DT	In Situ Density
GS	Grain Size Classification
HYD	Hydrometer
ILCN	Incremental Load Consolidation
K0CN	k0 Consolidation
kc	Constant Head Permeability
kf	Falling Head Permeability
MD	Moisture Density Relationship
OC	Organic Content
OT	Tests by Others
P	Pressuremeter
PID	Photoionization Detector Reading
PP	Pocket Penetrometer
SG	Specific Gravity
TRS	Torsional Ring Shear
TV	Torvane
UC	Unconfined Compression
UUC	Unconsolidated Undrained Triaxial Compression
VS	Vane Shear
WC	Water Content (%)

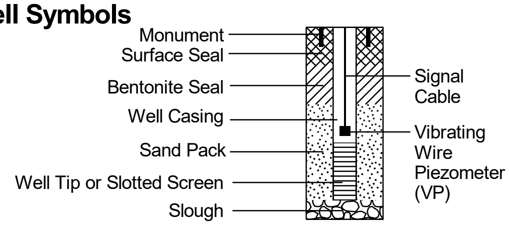
Groundwater Indicators

- Groundwater Level on Date or At Time of Drilling (ATD)
- Groundwater Level on Date Measured in Piezometer
- Groundwater Seepage (Test Pits)

Sample Symbols

1.5" I.D. Split Spoon	Rock Core Run	Grab
3.0" I.D. Split Spoon	Sonic Core	Cuttings
Modified California Sampler	Thin-walled Sampler	Push Probe

Well Symbols



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Project: Camas Station Development
Location: Camas, Oregon
Project No.: 0202499-006

Key to Exploration Logs

Figure **A-1**
Sheet **1 of 1**

Date Started: 12/15/2021 | Date Completed: 12/15/2021 | Contractor/Crew: Stratus Corporation
Logged by: M. Parks | Checked by: L. Kevan | Rig Model/Type: Cat® 312E / Excavator
Location: Lat: 45.590912 Long: -122.452991 (WGS 84) | Total Depth: 9 feet | Depth to Seepage: Not Encountered
Ground Surface Elevation: 523.04 feet (NAVD 88)
Comments:

Elevation (feet)	Depth (feet)	Sample Data: Type / Length (inches) / Number Tests	Graphic Log	Material Description	PL / WC / LL; Fines Content (%) (10, 20, 30, 40)
	0			SILT WITH SAND (ML), (soft to stiff), moist, dark brown, fine sand, roots and organics. [TOPSOIL]	
				LEAN CLAY (CL), (stiff), moist, yellow-brown. [RESIDUAL SOIL]	
		S-1 WC			
520					
		S-2 AL, WC		grades to very stiff, fissured clay	
	5				
		S-3			
515		S-4 GS, WC		GRAVELLY LEAN CLAY WITH SAND (CL), few cobbles, (very stiff to hard), moist, yellow brown, fine to coarse gravel, black angular basalt rock fragments, friable gravels and cobbles.	68
				Bottom of Test Pit at 9.0 feet.	
	10				
510					
	15				
505					

General Notes:
1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Material stratum lines are interpretive and actual changes may be gradual. Solid lines indicate distinct contacts and dashed lines indicate gradual or approximate contacts.
3. USCS designations are based on visual-manual identification (ASTM D 2488), unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling/excavation (ATD) or for date specified. Level may vary with time.
5. Location and ground surface elevations are approximate.

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Project: Camas Station Development
Location: Camas, Oregon
Project No.: 0202499-006

Test Pit Log
TP-1

Figure **A-2**
Sheet **1 of 1**

Date Started: 12/15/2021 | Date Completed: 12/15/2021 | Contractor/Crew: Stratus Corporation

Logged by: M. Parks | Checked by: L. Kevan | Rig Model/Type: Cat® 312E / Excavator

Location: Lat: 45.590869 Long: -122.453546 (WGS 84) | Total Depth: 8.5 feet | Depth to Seepage: 1.33 feet

Ground Surface Elevation: 522.08 feet (NAVD 88)

Comments:

Elevation (feet)	Depth (feet)	Sample Data: Number / Tests	Material Description	Water Level
	0	S-1	SILT WITH SAND (ML), (soft to stiff), moist, dark brown, fine sand, roots and organics. [TOPSOIL]	
520		S-2 / WC	LEAN CLAY (CL), (stiff), moist, gray-brown mottled red-yellow, iron oxide staining, pinhole porosity. [RESIDUAL SOIL]	12/15/2021
	5			
		S-3	LEAN CLAY WITH SAND (CL), (stiff), moist, red-yellow mottled gray-brown, fine sand, iron oxide staining.	
515		S-4 / WC	GRAVELLY LEAN CLAY WITH SAND (CL), (stiff), moist, yellow-red, clasts of black angular basalt rock fragments, friable gravel.	

Bottom of Test Pit at 8.5 feet.

Sample Data columns: Type, Length (inches), Number / Tests. Graphic Log. WC plot scale: 10, 20, 30, 40. Depth (feet): 0, 5, 10, 15. Elevation (feet): 520, 515, 510, 505.

General Notes:
1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Material stratum lines are interpretive and actual changes may be gradual. Solid lines indicate distinct contacts and dashed lines indicate gradual or approximate contacts.
3. USCS designations are based on visual-manual identification (ASTM D 2488), unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling/excavation (ATD) or for date specified. Level may vary with time.
5. Location and ground surface elevations are approximate.

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Project: Camas Station Development
Location: Camas, Oregon
Project No.: 0202499-006

Test Pit Log
TP-2

Figure **A-3**
Sheet **1 of 1**

Date Started: 12/15/2021 | Date Completed: 12/15/2021 | Contractor/Crew: Stratus Corporation
Logged by: M. Parks | Checked by: L. Kevan | Rig Model/Type: Cat® 312E / Excavator
Location: Lat: 45.591216 Long: -122.453256 (WGS 84) | Total Depth: 9.25 feet | Depth to Seepage: 1.17 feet
Ground Surface Elevation: 517.77 feet (NAVD 88)
Comments: Infiltration test conducted at 3 feet. See text for additional details.

Elevation (feet)	Depth (feet)	Sample Data: Type	Length (inches)	Number / Tests	Graphic Log	Material Description	Water Level	PL / WC / LL	Depth (feet)
	0					SILT WITH SAND (ML), (soft to stiff), moist, dark brown, fine sand, roots and organics. [TOPSOIL]			0
						LEAN CLAY (CL), few sand, trace gravel, (very stiff), moist to wet, yellow-brown mottled gray-brown, iron oxide staining, pinhole porosity. [RESIDUAL SOIL]	12/15/2021		
515				S-1					
							12/15/2021		
	5			S-2 AL, WC					5
				S-3		LEAN CLAY WITH SAND (CL), (very stiff), wet, yellow-brown mottled gray-brown, fine sand. trace gravel, trace cobbles			
510				S-4		GRAVELLY LEAN CLAY WITH SAND (CL), (hard), moist, yellow-red, clasts of black angular basalat rock fragments, friable gravel.			
						Bottom of Test Pit at 9.3 feet.			
	10								10
505									
	15								15
500									

Atterberg chart scale: 10, 20, 30, 40

General Notes:
1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Material stratum lines are interpretive and actual changes may be gradual. Solid lines indicate distinct contacts and dashed lines indicate gradual or approximate contacts.
3. USCS designations are based on visual-manual identification (ASTM D 2488), unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling/excavation (ATD) or for date specified. Level may vary with time.
5. Location and ground surface elevations are approximate.

Date Started: 12/15/2021	Date Completed: 12/15/2021	Contractor/Crew: Stratus Corporation	
Logged by: M. Parks	Checked by: L. Kevan	Rig Model/Type: Cat® 312E / Excavator	
Location: Lat: 45.591532 Long: -122.453053 (WGS 84)		Total Depth: 7 feet	Depth to Seepage: 1.25 feet
Ground Surface Elevation: 509.45 feet (NAVD 88)			
Comments:			

Elevation (feet)	Depth (feet)	Sample Number / Tests	Material Description	Water Level	WC / Fines Content (%)
	0		SILT WITH SAND (ML), (soft to stiff), moist, dark brown, fine sand, roots and organics. [TOPSOIL]		
			LEAN CLAY (CL), few fine sand, (medium stiff), moist, gray-brown mottled red-brown, fissured clay, iron oxide staining, clasts of black rock fragments. [RESIDUAL SOIL]	12/15/2021	
		S-1 GS, WC			Fines 92
505	5			12/15/2021	
		S-2 GS, WC	grades to stiff to very stiff, yellow-brown, less mottled		Fines 90
		S-3 WC	GRAVELLY LEAN CLAY WITH SAND (CL), (very stiff to hard), moist, yellow-red, clasts of black angular basalt rock fragments, friable gravel.		
			Bottom of Test Pit at 7.0 feet.		
500	10				
495	15				
490					

General Notes:
1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Material stratum lines are interpretive and actual changes may be gradual. Solid lines indicate distinct contacts and dashed lines indicate gradual or approximate contacts.
3. USCS designations are based on visual-manual identification (ASTM D 2488), unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling/excavation (ATD) or for date specified. Level may vary with time.
5. Location and ground surface elevations are approximate.

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A division of Haley & Aldrich

Project: Camas Station Development
Location: Camas, Oregon
Project No.: 0202499-006

Test Pit Log
TP-4

Figure **A-5**
Sheet **1 of 1**

Date Started: 12/15/2021 | Date Completed: 12/15/2021 | Contractor/Crew: Stratus Corporation
Logged by: M. Parks | Checked by: L. Kevan | Rig Model/Type: Cat® 312E / Excavator
Location: Lat: 45.591587 Long: -122.453789 (WGS 84) | Total Depth: 13.2 feet | Depth to Seepage: 5.5 feet
Ground Surface Elevation: 522.06 feet (NAVD 88)
Comments: Infiltration test conducted at 1 foot. See text for additional details.

Depth (feet)	Sample Number / Tests	Material Description
0	S-1	SILT WITH SAND (ML), (soft to stiff), moist, dark brown, fine sand, roots and organics. [TOPSOIL]
		LEAN CLAY WITH SAND (CL), trace gravel, (medium stiff), moist, yellow-brown mottled red-brown, fine sand. [RESIDUAL SOIL]
	S-2 / WC	
	S-3 / WC	grades to stiff, less mottled,fissured
5		
	S-4	pinhole porosity
		grades to stiff to very stiff, moist to wet, yellow-brown
	S-5 / AL, WC	
10		
	S-6	grades to few gravel, moist, clasts of black rock fragments
	S-7	
13.2		Bottom of Test Pit at 13.2 feet.

Water Level: 12/1/2021 (at 5.5 feet)

Elevation (feet): 520, 515, 510, 505

PL WC LL (10, 20, 30, 40)

Depth (feet): 0, 5, 10, 15

General Notes:
1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Material stratum lines are interpretive and actual changes may be gradual. Solid lines indicate distinct contacts and dashed lines indicate gradual or approximate contacts.
3. USCS designations are based on visual-manual identification (ASTM D 2488), unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling/excavation (ATD) or for date specified. Level may vary with time.
5. Location and ground surface elevations are approximate.

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A division of Haley & Aldrich

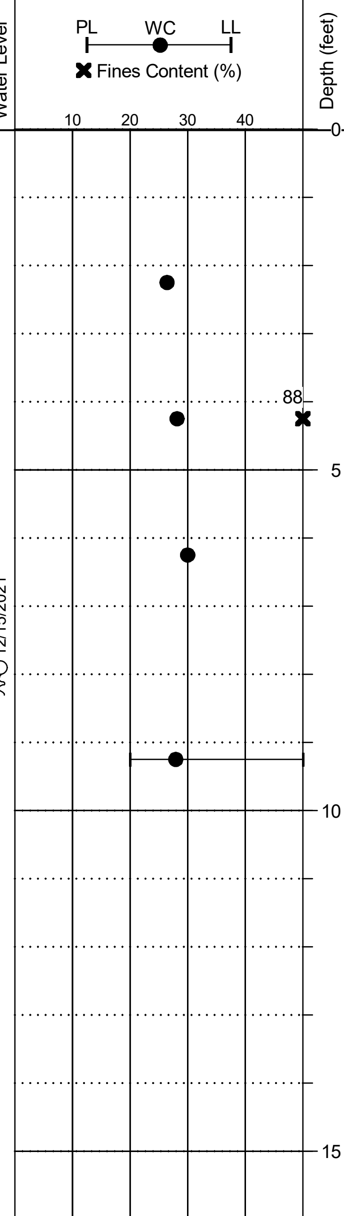
Project: Camas Station Development
Location: Camas, Oregon
Project No.: 0202499-006

Test Pit Log
TP-5

Figure **A-6**
Sheet **1 of 1**

Date Started: 12/15/2021	Date Completed: 12/15/2021	Contractor/Crew: Stratus Corporation	
Logged by: M. Parks	Checked by: L. Kevan	Rig Model/Type: Cat® 312E / Excavator	
Location: Lat: 45.591216 Long: -122.453576 (WGS 84)		Total Depth: 16 feet	Depth to Seepage: 8 feet
Ground Surface Elevation: 523.15 feet (NAVD 88)			
Comments:			

Elevation (feet)	Depth (feet)	Sample Number / Tests	Graphic Log	Material Description	Water Level
	0			SILT WITH SAND (ML), (soft to stiff), moist, dark brown, fine sand, roots and organics. [TOPSOIL]	
				LEAN CLAY (CL), few fine sand, (stiff), moist, yellow-brown mottled brown, fissured clay, iron oxide staining, pinhole porosity, roots. [RESIDUAL SOIL]	
		S-1 / WC			
520					
		S-2 / GS, WC		grades to yellow-red mottled brown, fine sand, fissured clay, iron oxide staining, roots	
	5				
		S-3 / WC		less mottled	
				grades to moist to wet, cemented, blocky	
515				FAT CLAY WITH GRAVEL (CH), few sand, (very stiff), moist, yellow-red, fine gravel, friable gravel.	12/15/2021
		S-4 / AL, WC			
	10				
510					
		S-5		grades to red	
	15				
				Bottom of Test Pit at 16.0 feet.	
505					



General Notes:
1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Material stratum lines are interpretive and actual changes may be gradual. Solid lines indicate distinct contacts and dashed lines indicate gradual or approximate contacts.
3. USCS designations are based on visual-manual identification (ASTM D 2488), unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling/excavation (ATD) or for date specified. Level may vary with time.
5. Location and ground surface elevations are approximate.

HARTCROWSER A division of Haley & Aldrich

Project: Camas Station Development
Location: Camas, Oregon
Project No.: 0202499-006

Test Pit Log
TP-6

Figure **A-7**
Sheet **1 of 1**

APPENDIX B
Laboratory Testing

APPENDIX B
LABORATORY TESTING

General

Soil samples obtained from the explorations were transported to our laboratory and to a subcontracted laboratory (Northwest Testing, Inc.) and evaluated to confirm or modify field classifications, as well as to assess engineering properties of the soils encountered. Representative samples were selected for laboratory testing. The tests were performed in general accordance with the test methods of the ASTM or other applicable procedures. The test results are included in this appendix, and where noted, included on the exploration log in Appendix A. A summary of the test results is included on Figure B-1. The specific tests conducted are outlined below. (We note that the test results from Northwest Testing, Inc. have been incorporated with test results from our lab into the attached figures.)

Visual Classifications

Soil samples obtained from the explorations were visually classified in the field and in our geotechnical laboratory based on the Unified Soil Classification System (USCS) and ASTM classification methods. ASTM Test Method D 2488 was used to classify soils using visual and manual methods. ASTM Test Method D 2487 was used to classify soils based on laboratory test results.

Laboratory Test Results

Moisture Content

Moisture contents of samples were obtained in general accordance with ASTM Test Method D 2216. The results of the moisture content tests completed on samples from the explorations are presented on the exploration logs included in Appendix A and on Figure B-1 in this appendix.

Percent Fines

Fines content analyses were performed to determine the percentage of soils finer than the No. 200 sieve—the boundary between sand size particles and silt size particles. The tests were performed in general accordance with ASTM Test Method D 1140. The test results are indicated on the exploration logs included in Appendix A and on Figure B-1 in this appendix.

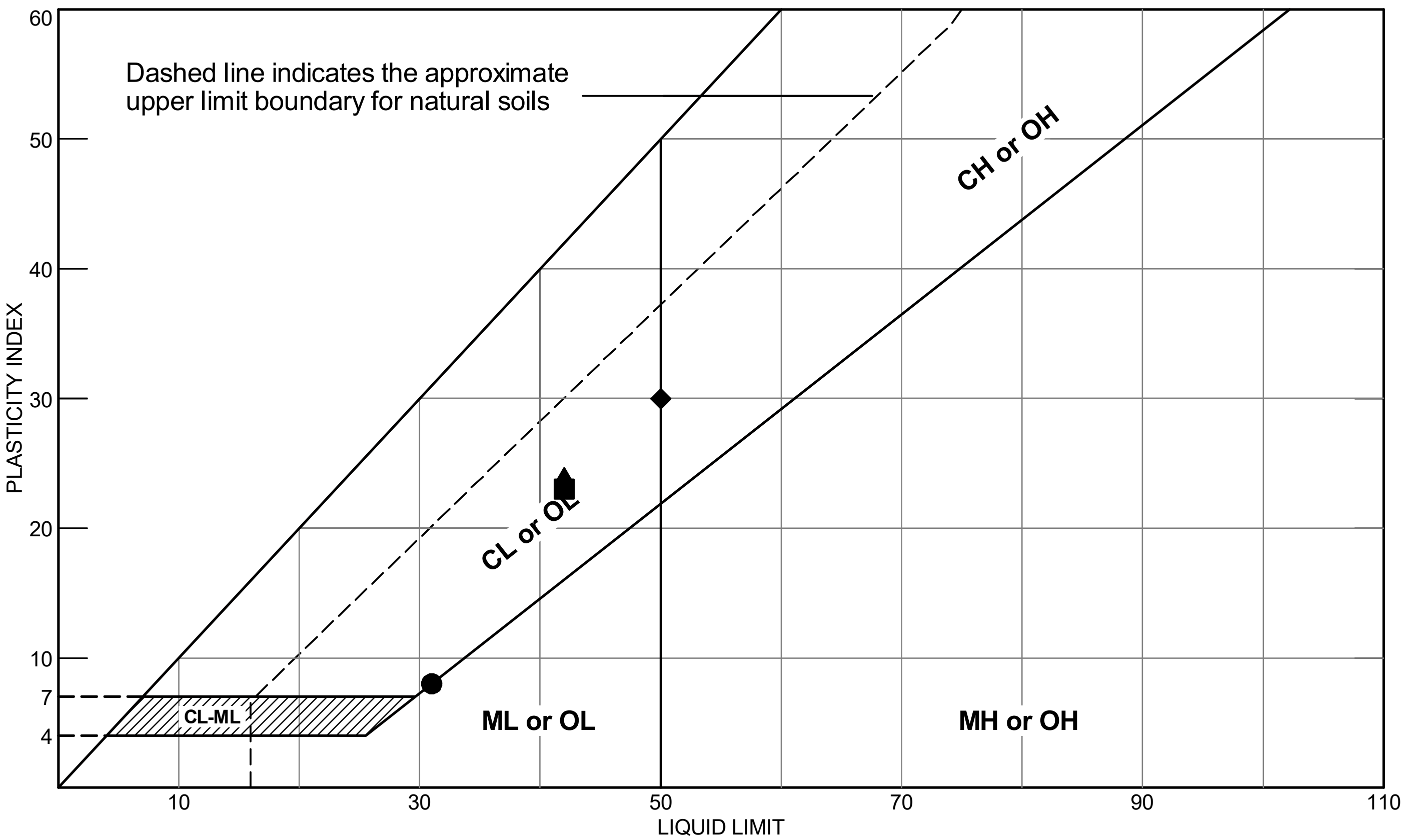
Grain Size Distribution

Grain size distribution analyses were conducted to determine the quantitative distribution of particle sizes in different soil samples. Fines content analyses were performed to determine the percentage of soils finer than the No. 200 sieve—the boundary between sand size particles and silt size particles. The tests were performed in general accordance with ASTM D 422, D 6913, and D 1140. The fines content test results are indicated on the exploration logs included in Appendix A. The test results are summarized on Figure B-1 in this appendix and the full grain size distribution test results are shown on Figure B-3 in this appendix.

Atterberg Limits Testing

Atterberg limits (liquid limit, plastic limit and plasticity index) were obtained in general accordance with ASTM Test Method D 4318. The results of the Atterberg limits test completed from the explorations is presented on the exploration logs included in Appendix A, summarized on Figure B-1 in this appendix, and shown in detail on Figure B-2 in this appendix.

Exploration	Sample ID	Depth	Gravel (%)	Sand (%)	Fines (%)	Liquid Limit	Plastic Limit	Water Content (%)	USCS Group Symbol	Soil Description
IT-1	S-1	1.0	0.5	14.8	84.7			30.9	CL	LEAN CLAY WITH SAND
IT-2	S-1	3.0	0.0	6.8	93.2			28.2	CL	LEAN CLAY
TP-1	S-1	2.0						23.8		
TP-1	S-2	4.0				31	23	24.4	CL	LEAN CLAY
TP-1	S-3	6.0								
TP-1	S-4	7.5	20.0	12.4	67.6			20.2	CL	LEAN CLAY WITH GRAVEL
TP-2	S-1	0.5								
TP-2	S-2	2.5						24.1		
TP-2	S-3	6.0								
TP-2	S-4	8.0						25.8		
TP-3	S-1	2.0								
TP-3	S-2	5.0				42	19	22.3	CL	LEAN CLAY
TP-3	S-3	7.0								
TP-3	S-4	8.5								
TP-4	S-1	2.0	0.0	8.0	92.0			23.9	CL	LEAN CLAY
TP-4	S-2	5.5	0.0	9.7	90.3			25.5	CL	LEAN CLAY
TP-4	S-3	6.5						24.8		
TP-5	S-1	0.0								
TP-5	S-2	2.0						27.1		
TP-5	S-3	4.0						25.8		
TP-5	S-4	6.0								
TP-5	S-5	8.0				42	18	23.8	CL	LEAN CLAY
TP-5	S-6	10.5								
TP-5	S-7	13.0								
TP-6	S-1	2.0						26.4		
TP-6	S-2	4.0	0.0	11.7	88.3			28.2	CL	LEAN CLAY
TP-6	S-3	6.0						30.0		
TP-6	S-4	9.0				50	20	27.9	CH	FAT CLAY
TP-6	S-5	14.0								



	Location and Description			LL	PL	PI	-#200	MC%	USCS
●	**Source:** TP-1 LEAN CLAY	**Sample No.:** S-2	**Depth:** 4.0 to 4.5	31	23	8	NT	24	CL
■	**Source:** TP-3 LEAN CLAY	**Sample No.:** S-2	**Depth:** 5.0 to 5.5	42	19	23	NT	22	CL
▲	**Source:** TP-5 LEAN CLAY	**Sample No.:** S-5	**Depth:** 8.0 to 8.5	42	18	24	NT	24	CL
◆	**Source:** TP-6 FAT CLAY	**Sample No.:** S-4	**Depth:** 9.0 to 9.5	50	20	30	NT	28	CH

Remarks:

● Borderline CL/ML

■

▲

◆ Borderline CH/CL

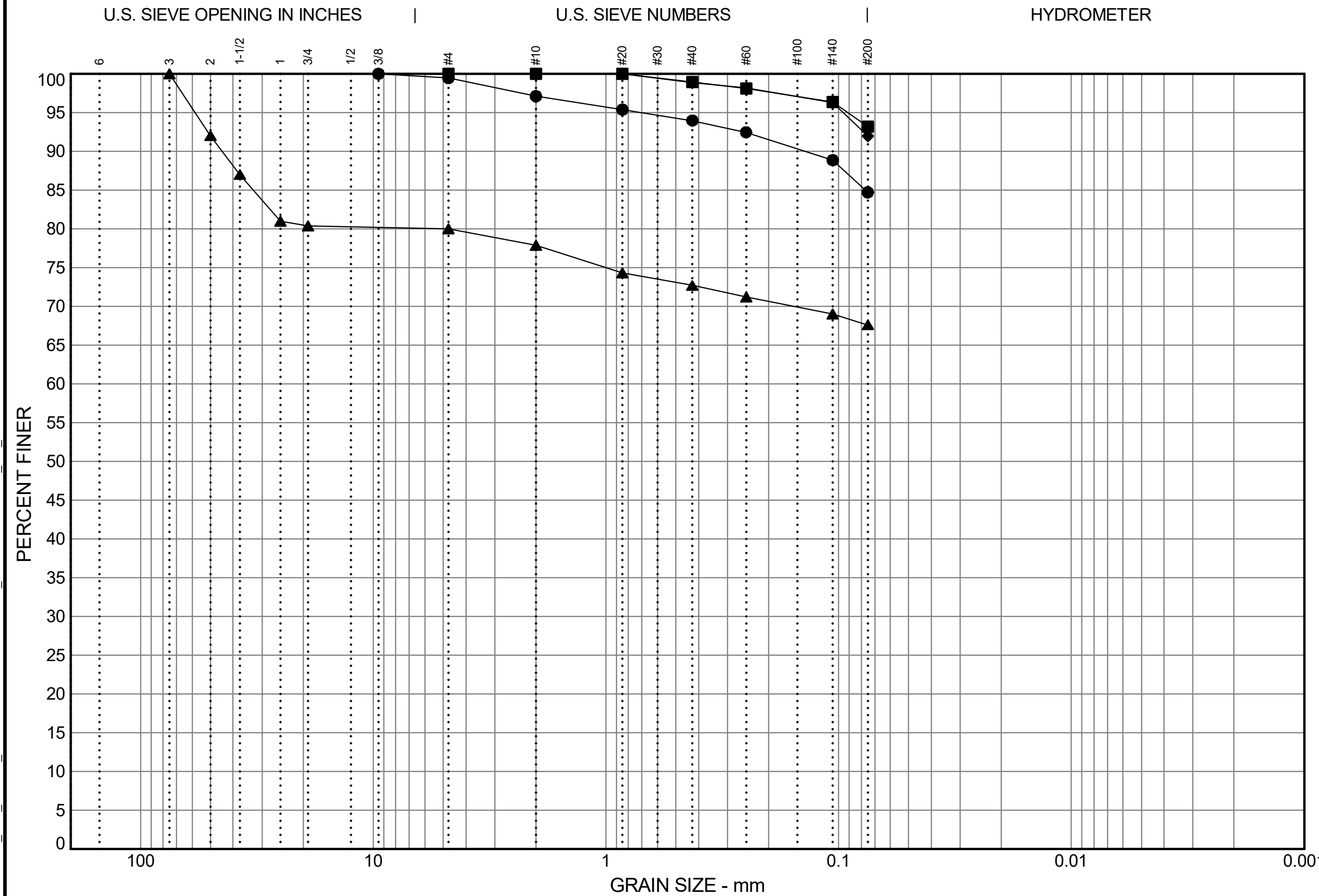
HC ATTERBERG LIMITS - \\HALEYALDRICH.COM\SHARE\SEA_DATA\GINT\HC_LIBRARY.GLB - 1/14/22 14:04 - \\HALEYALDRICH.COM\SHARE\PDX_DATA\NOTEBOOKS\0202499-006_MAJ_CAMAS_STATION\FIELD DATA\PERM_GINT FILES\202499_006_EXPLORATIONS.GPJ - ahultz

HARTCROWSER
A division of Haley & Aldrich

Project: Camas Station Development
Location: Camas, Oregon
Project No.: 0202499-006

Liquid Limit, Plastic Limit, and Plasticity Index

Figure **B-2**
Sheet **1 of 1**



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Location and Description	% Cobbles	% Gravel	% Sand	% Silt	% Clay	MC%	USCS
● **Source:** IT-1 **Sample No.:** S-1 **Depth:** 1.0 to 2.0 LEAN CLAY WITH SAND	0.0	0.5	14.8	84.7		31	CL
■ **Source:** IT-2 **Sample No.:** S-1 **Depth:** 3.0 to 4.0 LEAN CLAY	0.0	0.0	6.8	93.2		28	CL
▲ **Source:** TP-1 **Sample No.:** S-4 **Depth:** 7.5 to 8.0 LEAN CLAY WITH GRAVEL	0.0	20.0	12.4	67.6		20	CL
◆ **Source:** TP-4 **Sample No.:** S-1 **Depth:** 2.0 to 2.5 LEAN CLAY	0.0	0.0	8.0	92.0		24	CL

	LL	PI	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
●			0.077							
■										
▲			32.759							
◆										

Remarks:

● Scattered organics

■

▲ Note: Bulk sample contained 16.2 % +3-inch material (dry wt. basis). The +3-inch material was omitted for fractional classification.

◆

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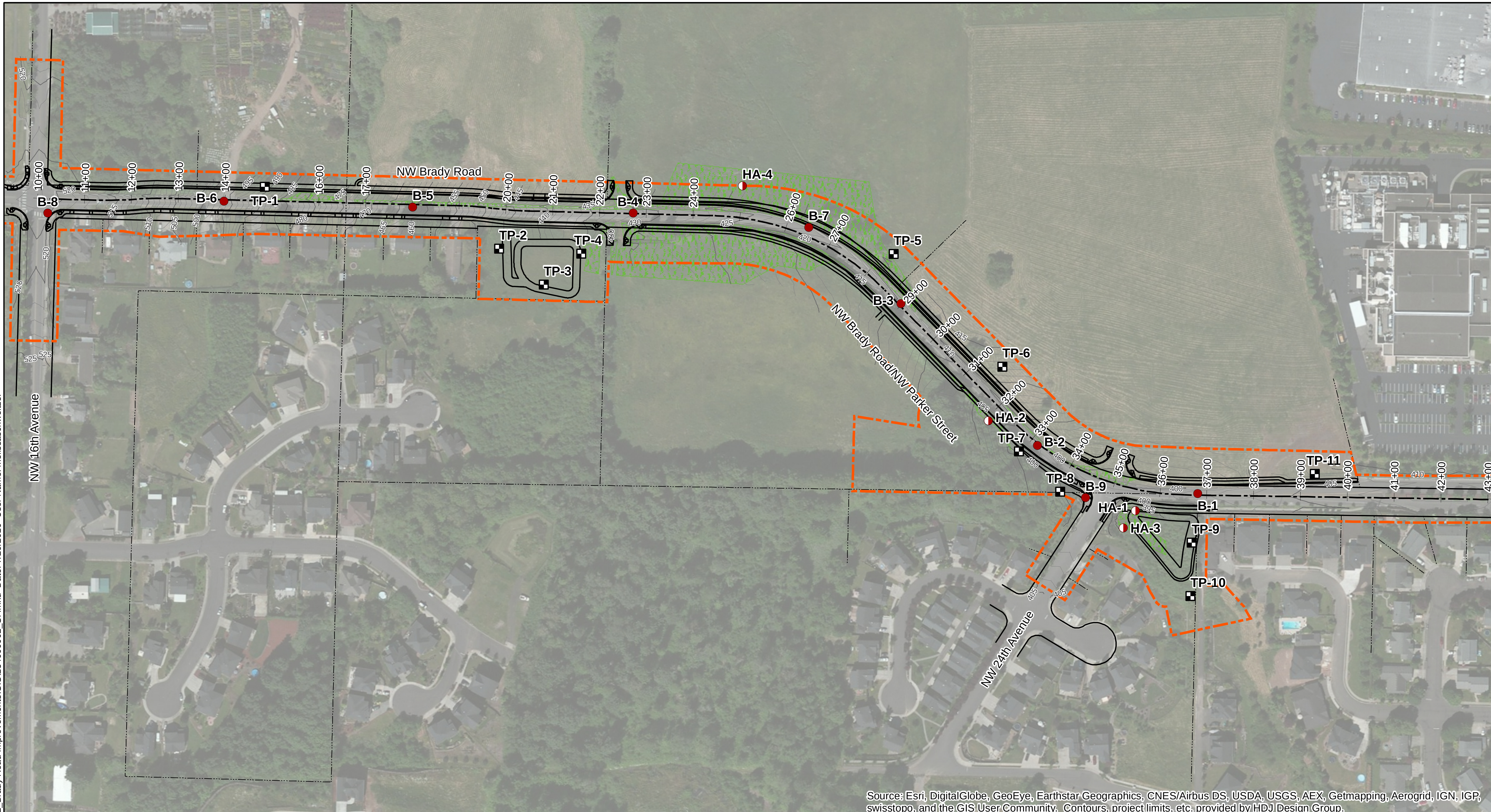
Project: Camas Station Development Location: Camas, Oregon Project No.: 0202499-006	**Particle-Size Analysis**	Figure **B-3** Sheet **1 of 1**

APPENDIX C
Historical Exploration Logs



LEGEND

- Boring
- Hand-Auger Boring
- Test Pit
- Project Limits
- Lot Boundary
- Edge of Right-of-Way
- Elevation Contour
- Wetland (Mapped by Others)

N

0 100 200 400 Feet

NW Brady Road Improvements
Camas, Washington

Site Plan

154-000-002 4/17

HARTCROWSER

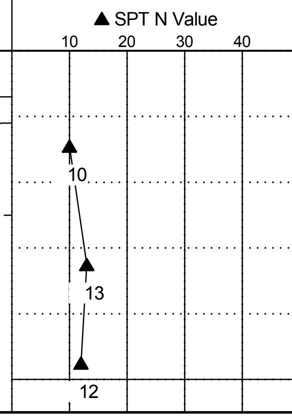
Figure 2

Document Path: F:\Notebooks\154000002_Brady Road Improvements\GIS\154000002_SP.mxd Date: 7/26/2016 User Name: melissaschweitzer

Date Started: 1/13/15	Date Completed: 1/13/15	Drilling Contractor/Crew: Dan J. Fischer Excavating, Inc.	
Logged by: R. Pirot	Checked by: D. Trisler	Drilling Method: Solid-stem Auger	
Location: N: 100,433.51 E: 1,140,527.40		Rig Model/Type: Big Beaver	
Ground Surface Elevation: 496 feet		Hammer Type: Manual	
Horizontal Datum: WA State Plane S, NAD 83, ft.		Hammer Weight: 140	Hammer Drop Height: 30
Vertical Datum: NGVD 29(47)		Hammer Efficiency (%): Measured: NA	Estimated: NA
Comments:		Auger Diameter: 3 inches	Casing Diameter: NA
		Total Depth: 5.5 feet	Depth to Ground Water: Not Identified

Elevation (feet)	Depth (feet)	Blow Count	Type Recovery	Length (inches)	Number Tests	Material Description
496	0					Asphalt pavement (9-inch thick)
						Aggregate base (4-inch thick)
495		10, 5, 5	13in.	18	S-1 SG	Stiff, moist, gray, SILT WITH SAND (ML), fine subangular sand. (Residual Soil)
		7, 7, 6	17in.	18	S-2 SG	Stiff, moist, gray-brown, SILT WITH SAND (ML), fine subangular sand, iron oxide staining, low plasticity.
	5	4, 5, 7	17in.	18	S-3 SG	

Bottom of Borehole at 5.5 feet.



General Notes:
1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual. Solid stratum lines indicate distinct contact between soil strata or geologic units. Dashed stratum lines indicate gradual or approximate change between soil strata or geologic units.
3. USCS designations are based on visual-manual identification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling/excavation (ATD) or for date specified. Level may vary with time.

HARTCROWSER

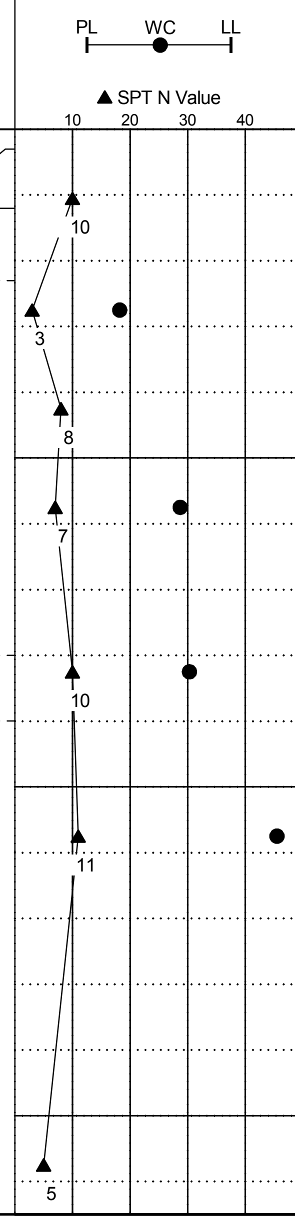
Project: Brady Road Improvements
Location: Camas, Washington
Project No.: 154-000-002

Boring Log
B-6

Figure **A-8**
Sheet **1 of 1**

Date Started: 1/14/15	Date Completed: 1/14/15	Drilling Contractor/Crew: Dan J. Fischer Excavating, Inc.	
Logged by: R. Pirot	Checked by: D. Trisler	Drilling Method: Solid-stem Auger	
Location: N: 100,057.05 E: 1,140,552.69		Rig Model/Type: Big Beaver	
Ground Surface Elevation: 520 feet		Hammer Type: Manual	
Horizontal Datum: WA State Plane S, NAD 83, ft.		Hammer Weight: 140	Hammer Drop Height: 30
Vertical Datum: NGVD 29(47)		Hammer Efficiency (%): Measured: NA	Estimated: NA
Comments:		Auger Diameter: 3 inches	Casing Diameter: NA
		Total Depth: 16.5 feet	Depth to Ground Water: Not Identified

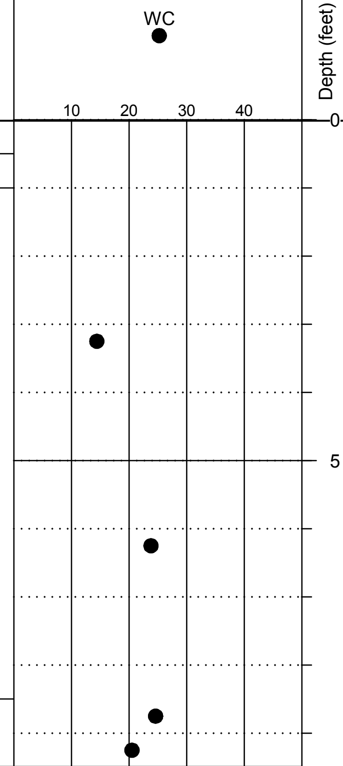
Elevation (feet)	Depth (feet)	Blow Count	Type / Recovery	Length (inches)	Number / Tests	Material Description
	0					Asphalt pavement (4-inch thick)
		6, 7, 3	13in.	18	S-1 / SG	Aggregate base (10-inch thick)
						Stiff, moist, yellow-brown, SILT WITH SAND (ML), fine sand, heavy iron oxide staining, low plasticity. (Residual Soil)
		2, 2, 1	11in.	18	S-2 / SG, WC	Soft to stiff, moist, gray-brown, SILT WITH SAND (ML), fine subangular sand, iron oxide staining, low plasticity.
		3, 4, 4	15in.	18	S-3 / SG	
515	5	3, 3, 4	18in.	18	S-4 / SG, WC	
						trace angular gravel/residual basalt fragments at 7.0 feet
		5, 4, 6	17in.	18	S-5 / AL, SG, WC	Stiff, moist, red-brown, SANDY LEAN CLAY (CL), fine to coarse subrounded to subangular sand, medium to high plasticity.
						Loose to medium dense, red to red-brown, CLAYEY SAND (SC), fine to coarse subrounded to subangular sand, heavy iron oxide staining, medium plasticity fines.
510	10	5, 6, 5	18in.	18	S-6 / SG, WC	
505	15	3, 3, 2	18in.	18	S-7 / SG	
						Bottom of Borehole at 16.5 feet.



General Notes:
1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual. Solid stratum lines indicate distinct contact between soil strata or geologic units. Dashed stratum lines indicate gradual or approximate change between soil strata or geologic units.
3. USCS designations are based on visual-manual identification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling/excavation (ATD) or for date specified. Level may vary with time.

Date Started: 6/28/16	Date Completed: 6/28/16	Excavation Contractor/Crew: Dan J. Fischer Excavating, Inc.	
Logged by: A. Jones	Checked by: D. Trisler	Excavation Method:	
Location: N: 100,520.34 E: 1,140,497.10		Rig Model/Type: John Deere 310E / Backhoe	
Ground Surface Elevation: 491 feet		Total Depth: 9.5 feet	Depth to Ground Water: Not Encountered
Horizontal Datum: WA State Plane S, NAD 83, ft.		Comments:	
Vertical Datum: NGVD 29(47)			

Elevation (feet)	Depth (feet)	Type / Recovery	Length (inches)	Number / Tests	Graphic Log	Material Description
	0					Topsoil (6-inch thick)
						(Soft), moist, dark brown, SILT WITH SAND (ML), fine sand. (Colluvium)
490						(Very stiff to stiff), moist, brown, SILT WITH SAND (ML), fine sand, relict rock structure, vesicles. (Residual Soil)
		6in.	6	S-1 PP, WC		(PP = 3.0 tsf)
						gray mottling and iron oxide staining below 4.0 feet
	5					
485		6in.	6	S-2 WC		
		6in.	6	S-3 WC		BASALTIC ANDESITE, moderately vesicular, moderately to highly weathered, moderately strong to strong (R3-R4). (Boring Lava Volcanics)
		6in.	6	S-4 WC		
						Bottom of Test Pit at 9.5 feet.
480	10					
475	15					



General Notes:
1. Refer to Figure A-1 for explanation of descriptions and symbols.
2. Soil descriptions and stratum lines are interpretive and actual changes may be gradual. Solid stratum lines indicate distinct contact between soil strata or geologic units. Dashed stratum lines indicate gradual or approximate change between soil strata or geologic units.
3. USCS designations are based on visual-manual identification (ASTM D 2488) unless otherwise supported by laboratory testing (ASTM D 2487).
4. Groundwater level, if indicated, is at time of drilling/excavation (ATD) or for date specified. Level may vary with time.

HARTCROWSER	Project: Brady Road Improvements Location: Camas, Washington Project No.: 154-000-002	Test Pit Log **TP-1**	Figure **A-16** Sheet **1 of 1**