5. Geotechnical Soil Analysis Report (Includes Geologic Hazard Study)

Report of Geotechnical Engineering Services

Webberley Development

Camas, Washington

October 11, 2023

Geotechnical ▪ Environmental ▪ Special Inspections

Columbia West

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October 11, 2023

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Report of Geotechnical Engineering Services
Webberley Development
Camas, Washington
Columbia West Project: HSR-3-01-1

Columbia West is pleased to present this report of geotechnical engineering services for the Webberley Development in Camas, Washington. Our services were conducted in accordance with our proposal dated August 21, 2023.

We appreciate the opportunity to work on the project. Please contact us if you have any questions regarding this document.

Sincerely,

Columbia West

Daniel Lehto, PE
Principal Engineer

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Attachments
Document ID: Webberley Development Geotechnical Report.docx

EXECUTIVE SUMMARY

This executive summary presents the primary geotechnical considerations associated with the proposed Webberley Development project located in Camas, Washington. Our conclusions and recommendations are based upon the subsurface information presented in this report and proposed development information provided by the design team. Detailed discussion of the geotechnical considerations summarized here is presented in respective sections of the report.

- Based on subsurface exploration and testing, infiltration of concentrated stormwater is infeasible due to the presence of relatively shallow bedrock and slowly permeable soils. In our opinion, the behavior of site soils indicate they should be classified as WWHM Group 4 soils. See Section 6.0, Infiltration Testing for details.

- Based upon site research, surface reconnaissance, and subsurface exploration, specific Geologic Hazards, as defined by the *Camas Municipal Code, Section 16.59,* were not encountered at the site.

- Excavator refusal was encountered in several test pits in the proposed development area as shallow as 6 feet BGS and groundwater was encountered at 11.5 feet BGS in TP-2 in August 2023. Deep excavations at the site may require rock excavation techniques and/or dewatering systems to install necessary elements.

TABLE OF CONTENTS

LIST OF FIGURES		iii
LIST OF APPENDICES		iv
1.0	INTRODUCTION	1
	1.1 General Site Information	1
	1.2 Project Understanding	1
2.0	SCOPE OF SERVICES	1
3.0	REGIONAL GEOLOGY AND SOIL CONDITIONS	2
4.0	REGIONAL SEISMOLOGY	3
5.0	GEOTECHNICAL AND GEOLOGIC FIELD INVESTIGATION	4
	5.1 Surface Investigation and Site Description	4
	5.2 Subsurface Conditions	4
6.0	INFILTRATION TESTING	5
	6.1 General	5
	6.2 Results	5
7.0	GEOLOGIC HAZARDS	6
	7.1 Erosion Hazards	6
	7.2 Landslide Hazards	6
	7.3 Seismic Hazard Area	6
8.0	DESIGN RECOMMENDATIONS	7
	8.1 Shallow Foundation Support	7
	8.2 Seismic Design Considerations	9
	8.3 Retaining Structures	9
	8.4 Pavement Design	10
	8.5 Drainage	10
9.0	CONSTRUCTION RECOMMENDATIONS	11
	9.1 Site Preparation and Grading	11
	9.2 Construction Traffic and Staging	12
	9.3 Cut and Fill Slopes	12
	9.4 Excavation	13
	9.5 Dewatering	13
	9.6 Materials	14
	9.7 Erosion Control Measures	17
10.0	OBSERVATION OF CONSTRUCTION	17
11.0	CONCLUSION AND LIMITATIONS	18

REFERENCES

TABLE OF CONTENTS CONTINUED

FIGURES

Site Location Map	Figure 1
Exploration Location Map	Figure 2
Preliminary Site Plan	Figure 2A
Surcharge-Induced Lateral Earth Pressures	Figure 3
Typical Perimeter Footing Drain Detail	Figure 4
Typical Perforated Drainpipe Trench Detail	Figure 5
Typical Drainage Mat Detail	Figure 6
Typical Cut and Fill Slope Cross-Section	Figure 7
Minimum Foundation Slope Setback Detail	Figure 8

APPENDICES

Appendix A	
Field Explorations	A-1
Exploration Legend	Table A-1
Soil Description and Classification	Table A-2
Test Pit Logs	Figures A-1 to A-8
Appendix B	
Laboratory Test Reports	B-1
Appendix C	
Report Limitations and Important Information	C-1

GEOTECHNICAL SITE INVESTIGATION
WEBBERLEY DEVELOPMENT
CAMAS, WASHINGTON

1.0 INTRODUCTION
Columbia West Engineering, Inc. (Columbia West) was retained by HSR Development to conduct a geotechnical site investigation for the proposed Webberley Development project located in Camas, Washington. The purpose of the investigation was to provide geotechnical engineering recommendations for use in design and construction of the proposed development. This report summarizes the investigation and provides field assessment documentation and laboratory analytical test reports. This report is subject to the limitations expressed in Section 11.0, *Conclusion and Limitations*, and Appendix C.

1.1 GENERAL SITE INFORMATION
As indicated on Figures 1 and 2, the subject site is located east and west of 921 SE Gardner Road in Camas, Washington. The site is comprised of tax parcels 178140000, 178159000, 178169000, and 178108000 totaling approximately 36.12 acres. The approximate latitude and longitude are N 45° 36′ 56″ and W 122° 23′ 53″, and the legal description is a portion of the NE ¼ of Section 35, T2N, R3E, Willamette Meridian. The regulatory jurisdictional agency is the City of Camas.

1.2 PROJECT UNDERSTANDING
Based on client correspondence and review of the preliminary site plan shown in Figure 2A, proposed development includes construction of a 156-lot residential subdivision and a 10-building multi-family residential and commercial development. Proposed development also includes paved access roads, paved parking lots, essential underground utilities, and stormwater management facilities. Grading plans were not available at the time this report was prepared.

We anticipate maximum loads for the buildings will be less than 40 kips per column and 5 kips per foot for perimeter footings. Allowable total and differential static settlement tolerances for the structures are 1 inch and 0.5 inch over a 50-foot span, respectively. We also anticipate that proposed structures will be Risk Category II with a fundamental period less than 0.5 second. We should be contacted to revise our recommendations if the assumptions stated above are incorrect.

2.0 SCOPE OF SERVICES
Columbia West's scope of services was outlined in a proposal dated August 21, 2023. In accordance with our proposal, we performed the following geotechnical services:

- Reviewed information available in our files from previous geological and geotechnical studies conducted at and in the vicinity of the site.
- Reviewed preliminary site plans and structural information provided by the design team.
- Conducted subsurface exploration at the site, to include:
    - Excavated 12 test pits to depths ranging from 6 to 13 feet BGS. Infiltration testing was conducted in eight test pits.
- Collected disturbed soil samples from test pits for laboratory analysis.
- Classified and logged observed soil and groundwater conditions.

- Prepared this geotechnical site investigation report for the proposed development, which includes:
  - Summary of soil index properties, regional geology, soil conditions, and observed groundwater conditions.
  - Summary of geologic and seismic literature research used to evaluate relevant seismic risks, including locations of faults, earthquake magnitudes, and seismic factors from the *2018 IBC* and *ASCE 7-16*
  - Infiltration test results
  - Fill- and load-induced settlement potential
  - Geotechnical design and construction recommendations for:
    - Shallow foundations
    - Lateral earth pressures
    - Site preparation and grading, organic stripping, fill placement and compaction, over-excavation, and construction monitoring and testing,
    - Structural fill materials, onsite soil suitability, and import aggregate specifications,
    - Utility trench excavation and backfill,
    - Drainage and management of groundwater conditions,
    - Asphaltic concrete pavement construction for access roads and parking lots
    - Seismic design parameters in accordance with *ASCE 7-16*

3.0 REGIONAL GEOLOGY AND SOIL CONDITIONS

The subject site lies within the Willamette Valley/Puget Sound Lowland, a wide physiographic depression flanked by the mountainous Coast Range on the west and the Cascade Range on the east. Inclined or uplifted structural zones within the Willamette Valley/Puget Sound Lowland constitute highland areas and depressed structural zones form sediment-filled basins. The site is located in the central eastern portion of the Portland/Vancouver Basin, an open, somewhat elliptical, northwest-trending syncline approximately 60 miles wide.

According to the *Geologic Map of the Camas Quadrangle, Clark County, Washington, and Multnomah County, Oregon* (US Geological Survey, Scientific Investigations Map 3017, 2008) and the *Geologic Map of the Washougal Quadrangle, Clark County, Washington, and Multnomah County, Oregon* (US Geological Survey, Scientific Investigations Map 3257, 2013), near-surface geology is expected to primarily consist of Pleistocene to Pliocene, unconsolidated to semiconsolidated, deeply weathered sedimentary deposits of the Unnamed conglomerate (QTc). The unnamed conglomerate is lithologically similar to the Pliocene or late Miocene Troutdale Formation, differing primarily in age of emplacement, degree of weathering, and the presence of hyaloclastite interbeds. Previously published geologic mapping has identified the QTc unit as the Troutdale Formation.

The *Web Soil Survey* (United States Department of Agriculture, Natural Resource Conservation Service [USDA NRCS], 2022 Website) identifies surface soils as Hesson clay loam with a small pocket of Washougal gravelly loam mapped in the northwest corner of Parcel 178140000. Hesson soils are generally fine-textured clays, silts, and sands with low permeability, moderate to high water capacity, and low shear strength. They are generally moisture sensitive, somewhat compressible, and described as having low to moderate shrink-swell potential. Washougal soils are generally clayey gravel soils that form on river terraces. They are generally moderately permeable, have moderate shear strength and minor shrink-swell potential.

4.0 REGIONAL SEISMOLOGY

Recent research and subsurface mapping investigations within the Pacific Northwest appear to suggest the historic potential risk for a large earthquake event with strong localized ground movement may be underestimated. Past earthquakes in the Pacific Northwest appear to have caused landslides and ground subsidence, in addition to severe flooding near coastal areas. Earthquakes may also induce soil liquefaction, which occurs when elevated horizontal ground acceleration and velocity cause soil particles to interact as a fluid as opposed to a solid. Liquefaction of soil can result in lateral spreading and temporary loss of bearing capacity and shear strength. Liquefaction is discussed later in Section 7.0, *Geologic Hazards*

Three scenario earthquakes are possible with the local seismic setting. Two of the possible earthquake sources are associated with the Cascadia Subduction Zone (CSZ), and the third event is a shallow, local crustal earthquake that could occur in the North American Plate. The three earthquake scenarios are discussed below.

<u>Cascadia Subduction Zone</u>

The Cascadia Subduction Zone is a potential source of strong earthquake activity in the Portland/Vancouver Basin. This phenomenon is the result of the earth's large tectonic plate movement. Geologic evidence indicates that volcanic ocean floor activity along the Juan de Fuca ridge in the Pacific Ocean causes the Juan de Fuca Plate to perpetually move east and subduct under the North American Continental Plate. The subduction zone results in historic volcanic and potential earthquake activity in proximity to the plate interface, believed to lie approximately 20 to 50 miles west of the general location of the Oregon and Washington coast (Geomatrix Consultants, 1995).

Evidence suggests that this subduction zone has generated eight great earthquakes in the last 4,000 years, with the most recent event occurring approximately 300 years ago (Weaver and Shedlock, 1991).

Two types of subduction zone earthquakes are possible and considered in this report:

1 An interface event earthquake on the seismogenic part of the interface between the Juan de Fuca Plate and the North American Plate on the CSZ. This source is capable of generating earthquakes with a moment magnitude of 9.0.

2 A deep intraplate earthquake on the seismogenic part of the subducting Juan de Fuca Plate. These events typically occur at depths of between 30 and 60 km. This source is capable of generating an event with a moment magnitude of up to 8.0.

<u>Crustal Events</u>

There are at least six major known fault zones in the vicinity of the site that may be capable of generating potentially destructive horizontal accelerations. These fault zones are described briefly in Table 1.

Table 1. Faults Within the Site Vicinity

Fault Name	Proximity to Site (km) per USGS	Mapped Length (km) per USGS
Beaverton fault zone	29	15
Helvetia fault zone	27	7
Oatfield fault zone	20	29
Portland Hills fault zone	15	49
East Bank fault	15	29
Lacamas Lake fault	1	24

5.0 GEOTECHNICAL AND GEOLOGIC FIELD INVESTIGATION

Subsurface conditions were explored by excavating 12 test pits (TP-1 through TP-12) using a track-mounted excavator at the approximate locations shown on Figure 2. The test pits were excavated on August 23, 2023 to a maximum depth of 13 feet BGS. Subsurface conditions were logged in accordance with the Unified Soil Classification System (USCS). Disturbed soil samples were collected at representative depth intervals. Test pit logs are presented in Appendix A. Analytical laboratory test results are presented in Appendix B. Soil descriptions and classification information are provided in Appendix A.

5.1 SURFACE INVESTIGATION AND SITE DESCRIPTION

The site is located just east and west of 921 SE Gardner Road in Camas, Washington. The site is comprised of tax parcels 178140000, 178159000, 178169000, and 178108000 totaling approximately 36.12 acres. The site is bound by SE Gardner Road to the west, Lacamas Heights Elementary School and Camas High School to the south, and acreage-parcel residential properties to the north and east. The site can be divided into two general areas for discussion: a developed, divided 10-acre portion to the west and a single 26.12-acre undeveloped parcel to the east. Multiple single-family homes exist on the generally cleared, divided 10-acre portion adjacent to SE Gardner Road and no structures are built on the forested eastern parcel. A Bonneville Power Authority (BPA) easement exists on the western parcel and trends northwest-southeast in the northern third of the property. Most site terrain is relatively flat to gently rolling and characterized by grades of 5 to 10 percent.

5.2 SUBSURFACE CONDITIONS

The test pits were excavated through grass surface and a 3- to 4-inch-thick root zone. An organic top soil zone extended to approximately 12 inches BGS. Underlying the surface vegetation, fine-grained residual soil and sedimentary conglomerate were encountered to the maximum explored depth of 13 feet BGS. Subsurface lithology may generally be described by the soil units identified in the following text.

5.2.1 Residual Soil

Underlying the surface vegetation, medium dense (stiff) to dense (stiff) silt, silty or clayey sand, and silty or clayey gravel was observed to depths from 6 feet to the maximum depth explored of 13 feet BGS. The moisture content of the residual soil ranged from 11 to 41 percent at the time of exploration. Atterberg limits analysis indicates that the residual soils exhibit moderate plasticity behavior. Soils at the site are interpreted as residual soils derived from weathered sedimentary conglomerate bedrock.

5.2.2 Sedimentary Conglomerate

Underlying the fine-grained alluvium, sedimentary conglomerate of dense to very dense silty gravel with clay and cobbles.

5.2.3 Groundwater

Groundwater was observed at a depth of 11.5 feet BGS in test pits TP-2. Note that groundwater levels are subject to seasonal variance and may rise during extended periods of increased precipitation. Perched groundwater is typical in the Camas area, generally present near the surface during the wet season and dropping below depths of 10 to 15 feet in the dry season.

6.0 INFILTRATION TESTING

6.1 GENERAL

Infiltration potential of site soils was evaluated through in situ infiltration testing within test pits TP-1, TP-2, TP-3, TP-5, TP-7, TP-8, TP-9 and TP-10. Single-ring, falling head infiltration testing was performed by embedding a 3-inch standpipe into undisturbed native soil, filling the apparatus with water, and measuring time relative to changes in hydraulic head. Using Darcy's Law for saturated flow in homogenous media, the coefficient of permeability (k) was then calculated. Representative soil samples were collected from select test locations and submitted for laboratory analysis. Results of in situ infiltration testing are presented in Table 2.

6.2 RESULTS

Results of in situ infiltration testing are presented in Table 2.

Table 2. Infiltration Test Results

Test Number	Location	Depth (feet BGS)	Passing No. 200	Depth to Groundwater (feet BGS)	Saturated Hydraulic Conductivity (in/hr)	Recommended WWHM Soil Group
IT-1.1	TP-1	3	-	Not Observed	<0.02	4
IT-1.2		6	-	Not Observed	<0.02	4
IT-2.1	TP-2	3	31	11.5	2.5	4
IT-2.2		6	19	11.5	2.5	4
IT-3.1	TP-3	3	-	Not Observed	0.5	4
IT-5.1	TP-5	3	-	Not Observed	0.3	4
IT-5.2		6	-	Not Observed	<0.2	4
IT-7.1	TP-7	3	-	Not Observed	0.3	4
IT-7.2		6	-	Not Observed	1.0	4
IT-8.1	TP-8	3	-	Not Observed	0.2	4
IT-9.1	TP-9	4	40	Not Observed	2.0	4
IT-10.1	TP-10	4	40	Not Observed	0.4	4

6.2.1 Soil Group Classification

Based on results of infiltration testing and the presence of near-surface relatively impermeable conglomerate (QTc) site geology, infiltration potential is considered minimal at the site. The presence of near-surface seeps and springs may become evident in cut areas during earthwork activities. Columbia West classified near-surface soils into a representative soil group based upon site-specific infiltration test results and review of published literature. As indicated in Table 2, observed near-surface infiltration rates ranged from less than 0.02 to 2.5 inches per hour in the tested locations. Based upon review of USDA hydrologic soil group criteria (USDA, 2007), Appendix 2-A of the *2021 Clark County Stormwater Manual,* and the *Clark County WWHM Soil Groupings Memorandum* (Otak, 2010), the behavior of site soils generally meet the criteria for *Western Washington Hydrology Model WWHM* Soil Group 4 as presented in Table 2.

7.0 GEOLOGIC HAZARDS

Camas Municipal Code, Section 16.59 defines geologic hazard requirements for proposed development in areas subject to City of Camas jurisdiction. Three potential geologic hazards are identified: (1) erosion hazard areas, (2) landslide hazard areas, and (3) seismic hazard areas.

Columbia West conducted a geologic hazard review to assess whether these hazards are present at the subject property proposed for development, and if so, to provide mitigation recommendations. The geologic hazard review was based upon physical and visual reconnaissance, subsurface exploration, laboratory analysis of collected soil samples, and review of maps and other published technical literature. The results of the geologic hazard review are discussed in the following sections.

7.1 EROSION HAZARDS

Camas Municipal Code, Section 16.59.020.A defines an erosion hazard as areas where slope grades meet or exceed 40 percent. Based upon review of slope grade mapping published by *Clark County Maps Online,* maximum slope grades of 0 to 15 percent are mapped in the southwestern portion of the site. Therefore, site slopes do not meet the definition of an erosion hazard according to *Camas Municipal Code*.

7.2 LANDSLIDE HAZARDS

Columbia West conducted a review of available mapping, *Clark County GIS data*, and site reconnaissance to evaluate the potential presence of a landslide hazard on or near the subject site. There are no landslide hazards mapped on the property nor did Columbia West observe any slope stability hazards during site reconnaissance.

7.3 SEISMIC HAZARD AREAS

Seismic hazards include areas subject to severe risk of earthquake-induced damage. Damage may occur due to soil liquefaction, dynamic settlement, ground shaking amplification, or surface faulting rupture. These seismic hazards are discussed below.

7.3.1 Soil Liquefaction and Dynamic Settlement

According to *the Liquefaction Susceptibility Map of Clark County, Washington* (Washington State Department of Natural Resources, 2004), the site is mapped as very low susceptibility for liquefaction. Liquefaction, defined as the transformation of the behavior of a granular material from a solid to a liquid due to increased pore-water pressure and reduced effective stress, may occur when granular materials quickly compact under cyclic stresses caused by a seismic event. The effects of liquefaction may include immediate ground settlement, lateral spreading, and differential compaction.

Soils most susceptible to liquefaction are recent geologic deposits, such as river and floodplain sediments. These soils are generally saturated, cohesionless, loose to medium dense sands within 50 feet of ground surface. Potentially liquefiable soils located above the existing, historic, or expected ground water levels do not generally pose a liquefaction hazard. It is important to note that changes in perched ground water elevation may occur due to project development or other factors not observed at the time of investigation.

Based upon the results of subsurface exploration, literature review, and laboratory analysis, the above-mentioned criteria were not observed during the geotechnical site investigation. Therefore, the potential for soil liquefaction is considered to be very low.

7.3.2 Ground Shaking Amplification
Review of the *Site Class Map of Clark County, Washington* (Washington State Department of Natural Resources, 2004), indicates that site soils may be represented by Site Class C as defined in *2018 IBC Section 1613.3.2*. A designation of Site Class C indicates that minor amplification of seismic energy may occur during a seismic event due to subsurface conditions. However, this is typical for many areas within Clark County, does not represent a geologic hazard in Columbia West's opinion, and will not prohibit development if properly accounted for during the design process. Additional seismic information is presented in Section 8.2, *Seismic Design Considerations*.

7.3.3 Fault Rupture
Because there are no known geologic seismic faults within the site boundaries, fault rupture is unlikely.

8.0 DESIGN RECOMMENDATIONS
The geotechnical site investigation suggests the proposed development is generally compatible with surface and subsurface soils, provided the recommendations presented in this report are incorporated in design and implemented during construction. Design and construction recommendations are presented in the following sections.

8.1 Shallow Foundation Support
Proposed residential structures may be supported by conventional spread footings bearing on firm native soil or engineered structural fill.

Any loose or disturbed soil should be improved or removed and replaced with structural fill. If footing subgrade soils are above their optimum moisture content, we recommend that a minimum of 6 inches of compacted aggregate be placed over exposed subgrade soils. The aggregate pad should extend 6 inches beyond the edge of the foundations and consist of imported granular material as described in Section 9.6.1, *Structural Fill*. Columbia West should observe exposed subgrade conditions prior to placement of crushed aggregate to verify adequate subgrade support.

8.1.1 Footing Dimensions and Bearing Capacity
Continuous perimeter wall and isolated spread footings should have minimum width dimensions of 18 and 24 inches, respectively. The base of exterior footings should bear at least 18 inches below

the lowest adjacent exterior grade. The base of interior footings should bear at least 12 inches below the base of the floor.

Footings bearing on subgrade prepared as recommended above should be sized based on an allowable bearing pressure of 2,000 psf. As the allowable bearing pressure is a net bearing pressure, the weight of the footing and associated backfill may be ignored when calculating footing sizes. The recommended allowable bearing pressure applies to the total of dead plus long-term live loads and may be increased by 50 percent for transient lateral forces such as seismic or wind.

8.1.2 Shallow Foundation Settlement

Foundation settlement is a significant structural design consideration. Provided subgrade soils are prepared as described above and in Section 9.1, *Site Preparation and Grading*, we anticipate that post-construction static foundation settlement will be less than approximately 1 inch. Differential settlement between comparably loaded foundations is not expected to exceed approximately 0.5 inch over a distance of 50 feet.

8.1.3 Resistance to Sliding

Lateral foundation loads can be resisted by passive earth pressure on the sides of the footing and by friction at the base of the footings. Recommended passive earth pressure for footings confined by native soil or engineered structural fill is 250 pcf. The upper 12 inches of soil should be neglected when calculating passive pressure resistance. Adjacent floor slabs and pavement, if present, should also be neglected from the analysis. The recommended passive pressure resistance assumes that a minimum horizontal clearance of 10 feet is maintained between the footing face and adjacent downgradient slopes.

The estimated coefficient of friction between in situ native soil or engineered structural fill and in-place poured concrete is 0.35. The estimated coefficient of friction between compacted crushed aggregate and in-place poured concrete is 0.45.

8.1.4 Subgrade Observation

Upon completion of stripping and prior to the placement of structural fill or pavement improvements, exposed subgrade soil should be evaluated by proof rolling with a fully-loaded dump truck or similar heavy, rubber tire construction equipment. When the subgrade is too wet for proof rolling, a foundation probe may be used to identify areas of soft, loose, or unsuitable soil. Subgrade evaluation should be performed by Columbia West. If soft or yielding subgrade areas are identified during evaluation, we recommend the subgrade be over-excavated and backfilled with compacted imported granular fill.

8.1.5 Floor Slabs

Floor slabs can be supported on firm, competent, native soil or engineered structural fill prepared as described in this report. Disturbed soils and unsuitable fills in proposed slab locations, if encountered, should be removed and replaced with structural fill.

To provide a capillary break, slabs should be underlain by at least 6 inches of compacted crushed aggregate that contains less than 5 percent by weight passing the No. 200 Sieve. Geotextile may be used below the crushed aggregate layer to increase subgrade support. Recommendations for floor slab base aggregate and subgrade geotextile are discussed in Section 9.6, *Materials*.

Floor slabs with maximum floor load of 100 psf may be designed assuming a modulus of subgrade reaction, k, of 125 pci.

8.2 SEISMIC DESIGN CONSIDERATIONS

Seismic design for proposed structures is prescribed by *ASCE 7-16*. Based on literature review and results of subsurface exploration conducted by Columbia West, site soils meet the criteria for Site Class C. Seismic design parameters for Site Class C are presented in Table 3.

Table 3. ASCE 7-16 Seismic Design Parameters[1]

	Short Period	1 Second Period
MCE Spectral Acceleration	0.787	0.345
Site Class	C	
Site Coefficient	Fa = 1.2	Fv = 1.5
Adjusted Spectral Response Acceleration	S_{MS} = 0.944	S_{M1} = 0.517
Design Spectral Response Acceleration	S_{DS} = 0.629	S_{D1} = 0.345

1. The structural engineer should evaluate *ASCE 7-16* code requirements and exceptions to determine if these parameters are valid for design.

As discussed in Section 7.3, *Seismic Hazards Area*, liquefaction and lateral spreading are not design considerations for the site.

8.3 RETAINING STRUCTURES

Lateral earth pressures should be considered during design of retaining walls and below-grade structures. Hydrostatic pressure and additional surcharge loading should also be considered. Wall foundation construction and bearing capacity should adhere to specifications provided previously in Section 8.1, *Shallow Foundation Support*.

Permanent retaining walls that are not restrained from rotation and are retaining undisturbed native soil should be designed for active earth pressures using an equivalent fluid pressure of 39 pcf. Walls retaining undisturbed native soils that are restrained from rotation should be designed for an at-rest equivalent fluid pressure of 64 pcf. For walls with imported well-drained granular backfill meeting WSDOT 9.03.12(2), an equivalent fluid pressure of 34 pcf is applicable for active and 60 pcf for at rest is applicable.

The recommended earth pressures assume a maximum wall height of 10 feet with level backfill. These values also assume that adequate drainage is provided behind retaining walls to prevent hydrostatic pressures from developing. Lateral earth pressures induced by surcharge loads may be estimated using the criteria presented on Figure 3.

Seismic forces may be calculated by superimposing a uniform lateral force of $10H^2$ pounds per lineal foot of wall, where H is the total wall height in feet. The force should be applied as a distributed load with the resultant located at 0.6H from the base of the wall.

8.3.1 Wall Drainage and Backfill

A minimum 6-inch-diameter, perforated collector pipe should be placed at the base of retaining walls. The pipe should be embedded in a minimum 2-foot-wide zone of angular drain rock that is wrapped in a drainage geotextile fabric and extends up the back of the wall to within 1 foot of finished grade. The drain rock and geotextile drainage fabric should meet the specifications provided in Section 9.6, *Materials*. 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 drainage systems, unless measures are taken to prevent backflow into the drainage system of the wall.

Backfill material placed behind the walls and extending a horizontal distance of ½ H, where H is the height of the retaining wall, should consist of select granular material placed and compacted as described in Section 9.6.1, *Structural Fill*.

Settlement of up to 1 percent of the wall height commonly occurs immediately adjacent to the wall as the wall rotates and develops active lateral earth pressures. Consequently, we recommend that construction of flatwork adjacent to retaining walls be delayed at least four weeks after placement of wall backfill, unless survey data indicates that settlement is complete prior to that time.

8.4 PAVEMENT RECOMMENDATIONS

We understand that public roadways for the subdivision will be constructed in accordance with City of Camas standards. For dry weather construction, pavement surface sections should bear upon competent subgrade consisting of scarified and compacted native soil or engineered structural fill. Wet weather construction may require an increased thickness of base aggregate as discussed later in Section 9.2, *Construction Traffic and Staging*.

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

If AC paving must take place during cold-weather construction as defined in this section, the contractor and design team should discuss options for minimizing risk to pavement serviceability.

8.5 DRAINAGE

At a minimum, site drainage should include surface water collection and conveyance to properly designed stormwater management structures and facilities. Drainage design in general should conform to City of Camas regulations. Finished site grading should be conducted with positive drainage away from structures at a minimum 2 percent slope for a distance of at least 10 feet. Depressions or shallow areas that may retain ponding water should be avoided.

Recommendations for foundation drains and subdrains are presented in the following sections. Drain rock and geotextile drainage fabric should meet the requirements presented in Section 9.6, *Materials*. Drains should be closely monitored after construction to assess their effectiveness. If additional surface or shallow subsurface seeps become evident, the drainage provisions may require modification or additional drains. We should be consulted to provide appropriate recommendations.

8.5.1 Foundation Drains

Roof drains are recommended for all structures. Perimeter building foundation drains should be considered for shallow foundations constructed below existing site grades but are not necessary for the functionality of the buildings.

Foundation and roof drains, where installed, should consist of separate systems that gravity flow away from foundations to an approved discharge location. Perimeter foundation drains should consist of 4-inch perforated PVC pipe surrounded by a minimum 2-foot-wide zone of clean, washed drain rock wrapped with geotextile drainage fabric. The wrapped drain rock zone should extend up the sides of embedded walls to within 12 inches of proposed finished grade. Foundation drains should be constructed with a minimum slope of ½ percent. The drainpipe's invert elevation should be at least 18 inches below the elevation of the floor slab. Figure 4 presents a typical foundation drain detail.

8.5.2 Subdrains

Subdrains should be considered if portions of the site are cut below surrounding grades. Shallow groundwater or seeps should be conveyed via drainage channel or perforated pipe into an approved discharge. Recommendations for design and installation of perforated drainage pipe may be performed on a case-by-case basis by Columbia West during construction. Failure to provide adequate surface and sub-surface drainage may result in soil slumping or unanticipated settlement of structures exceeding tolerable limits. A typical perforated drainpipe trench detail is presented in Figure 5.

9.0 CONSTRUCTION RECOMMENDATIONS

9.1 SITE PREPARATION AND GRADING

Site vegetation primarily consisted of grass and a 3- to 4-inch-thick root zone at the time of our exploration. Thicker root zones may be present in areas of mature trees and shrub growth. Moderately organic topsoil was observed to a depth of approximately 12 inches BGS. Pavement, vegetation, organic material, unsuitable fill, and deleterious material should be cleared from areas identified for structures and site grading. Vegetation, root zones, organic material, and debris should be removed from the site. Stripped topsoil should also be removed or used only as landscape fill in nonstructural areas with slopes less than 25 percent. The post-construction maximum depth of landscape fill placed or spread at any location onsite should not exceed one foot. Actual stripping depths should be determined based upon visual observations made during construction when soil conditions are exposed.

9.1.1 Subgrade Evaluation

Upon completion of stripping and prior to the placement of structural fill or pavement improvements, exposed subgrade soil should be evaluated by proof rolling with a fully-loaded dump truck or similar heavy, rubber tire construction equipment. When the subgrade is too wet for proof rolling, a foundation probe may be used to identify areas of soft, loose, or unsuitable soil. Subgrade evaluation should be performed by Columbia West. If soft or yielding subgrade areas are identified during evaluation, we recommend the subgrade be over-excavated and backfilled with compacted imported granular fill.

9.2 CONSTRUCTION TRAFFIC AND STAGING

Near-surface clay will be easily disturbed during construction. If not carefully executed, site preparation, excavation, and grading can create extensive soft areas resulting in significant repair costs. Earthwork planning should include considerations for minimizing subgrade disturbance, particularly during wet-weather conditions.

If construction occurs during wet-weather conditions, or if the moisture content of the surficial soil is more than a few percentage points above optimum, site stripping and cutting may need to be accomplished using track-mounted equipment. Under these conditions, granular haul roads and staging areas will also be necessary to provide a firm support base and sustain construction equipment.

The recommended base aggregate thickness for pavement sections is intended to support post-construction design traffic loads and will not provide adequate support for construction traffic. Staging areas and haul roads will require an increased base thickness during wet weather conditions. The configuration of staging and haul road areas, as well as the required thickness of granular material, will vary with the contractor's means and methods. Therefore, design and construction of staging areas and haul roads should be the responsibility of the contractor. Based on our experience, between 12 and 18 inches of imported granular material is generally required in staging areas and between 18 and 24 inches in haul road areas. In areas of heavy construction traffic, geotextile separation fabric may be placed between the subgrade soil and imported granular material to increase subgrade support and minimize fines migration into the base aggregate layer.

Project stakeholders should understand that wet weather construction is risky and costly. Proper construction methods and techniques are critical to overall project integrity and should be observed and documented by Columbia West.

9.3 CUT AND FILL SLOPES

Fill slopes should consist of structural fill material as discussed in Section 9.6.1, *Structural Fill*. Fill placed on existing grades steeper than 5H:1V should be horizontally benched at least 10 feet into the slope. Fill slopes greater than six feet in height should be vertically keyed into existing subsurface soil. A typical fill slope cross-section is shown in Figure 7. Drainage implementations, including subdrains or perforated drainpipe trenches, may also be necessary in proximity to cut and fill slopes if seeps or springs are encountered. Drainage design may be performed on a case-by-case basis. Extent, depth, and location of drainage may be determined in the field by Columbia West during construction when soil conditions are exposed. Failure to provide adequate drainage may result in soil sloughing, settlement, or erosion.

Final cut or fill slopes at the site should not exceed 2H:1V or 10 feet in height without individual slope stability analysis. The values above assume a minimum horizontal setback for loads of 10 feet from top of cut or fill slope face or overall slope height divided by three (H/3), whichever is greater. A minimum slope setback detail for structures is presented in Figure 8.

Concentrated drainage or water flow over the face of slopes should be prohibited, and adequate protection against erosion is required. Fill slopes should be overbuilt, compacted, and trimmed at least two feet horizontally to provide adequate compaction of the outer slope face. Proper cut and fill slope construction is critical to overall project stability and should be observed and documented by Columbia West.

9.4 EXCAVATION

The site was explored to a maximum depth of 13 feet BGS with an excavator. Weathered conglomerate bedrock was encountered as shallow as 2 feet BGS (TP-6) and several test pits terminated by refusal on competent bedrock as shallow as 6 feet in the area proposed for development. If significant excavation depths are required to install utilities or other structural elements, additional exploration may be warranted to determine if conventional earthmoving equipment in proper working condition is capable of making necessary site excavations. Blasting or pecking may be required for significant excavation depths.

Groundwater was observed at a depth of 11.5 feet BGS in test pit TP-2. Recommendations as described in Section 9.5, *Dewatering*, should be considered where subsurface construction activities intersect the shallow groundwater table.

Temporary excavation sidewalls should maintain a vertical cut to a depth of approximately 4 feet in the near-surface clay, provided groundwater seepage is not present in the sidewalls. In sandy soil, excavations will likely slough and cave, even at shallow depths. Open-cut excavation techniques may be used to excavate trenches between 4 and 8 feet deep, provided the walls of the excavation are cut at a maximum slope of 1H:1V and groundwater seepage is not present. Excavation slopes should be reduced to 1.5H:1V or 2H:1V if excessive sloughing or raveling occurs.

Shoring may be required if open-cut excavations are infeasible or if excavations are proposed adjacent to existing infrastructure. Typical methods for stabilizing excavations consist of soldier piles and timber lagging, sheet pile walls, tiebacks and shotcrete, or prefabricated hydraulic shoring. As a wide variety of shoring and dewatering systems are available, we recommend that the contractor be responsible for selecting the appropriate shoring and dewatering systems.

The contractor should be held responsible for site safety, sloping, and shoring. All excavation activity should be conducted in accordance with applicable OSHA requirements. Columbia West is not responsible for contractor activities and in no case should excavation be conducted in excess of applicable local, state, and federal laws.

9.5 DEWATERING

Groundwater was observed as shallow as 11.5 feet BGS at the time of our drilling. Based on this observation, groundwater will likely be encountered in utility trench excavations and in areas of cut. Generalized recommendations for temporary construction dewatering are presented in the following section.

9.5.1 Construction Dewatering

The contractor should be responsible for temporary drainage of surface water, perched water, and groundwater. Dewatering should be performed to the extent necessary to prevent standing water and/or erosion of exposed site soils. During rough and finished grading of building pad areas, the contractor should keep all footing excavations and slab subgrade soils free of standing water.

The contractor's proposed dewatering plan should be capable of maintaining groundwater levels at least two feet below the base of proposed trench excavations. Without adequate trench dewatering, running soil, caving, and sloughing will increase backfill volumes and may result in damage to adjacent structures or utilities. Significant pumping and dewatering may be required to temporarily reduce the groundwater elevation to the recommended depth. Dewatering via a sump within excavation zones may be insufficient to control groundwater and provide excavation side slope stability. Dewatering may be more feasibly conducted by installing a system of temporary well points and pumps around proposed excavation areas or utility trenches. Depending on proposed utility depths, a site-specific dewatering plan may be necessary.

If groundwater is present at the base of utility excavations, we recommend placing 18 to 24 inches of stabilization material at the base of the excavation. Subgrade geotextile placed directly over trench subgrade soils may reduce the required thickness of the stabilization material. The actual thickness of stabilization material should be determined at the time of construction based on observed field conditions. Trench stabilization material should be placed in one lift and compacted until well keyed. Stabilization material and geotextile fabric should meet the requirements presented in Section 9.6, *Materials*.

9.6 MATERIALS

9.6.1 Structural Fill

Areas proposed for fill placement should be appropriately prepared as described in Section 9.1, *Site Preparation and Grading*. Engineered fill placement should be observed by Columbia West. Compaction of engineered structural fill should be verified by nuclear gauge field compaction testing performed in accordance with *ASTM D6938*. Field compaction testing should be performed for each vertical foot of engineered fill placed.

Various materials may be acceptable for use as structural fill. Structural fill should be free of organic material or other unsuitable material and meet specifications provided in the following sections. Representative samples of proposed engineered structural fill should be submitted for laboratory analysis and approval by Columbia West prior to placement.

9.6.1.1 Onsite Soil

Most onsite soil will be suitable for use as structural fill if adequately dried or moisture-conditioned to achieve recommended compaction specifications. Native clay soil with a plasticity index greater than 25, if encountered, should be evaluated and approved by Columbia West prior to use as structural fill. Laboratory analysis indicated that the moisture content of the near-surface clay was above optimum at the time of exploration. Moisture conditioning will likely be necessary to dry the soil prior to applying compaction effort. In addition, the near-surface clay will be moisture sensitive and difficult, if not impossible, to compact during wet weather conditions. Therefore, structural fill placement using onsite soil should be performed during dry summer months if possible. Onsite soil may also require addition of moisture during extended periods of dry weather.

Onsite soil used as structural fill should be placed in loose lifts not exceeding 8 inches in depth and compacted using standard conventional compaction equipment. The soil moisture content should be within a few percentage points of optimum conditions. The soil should be compacted to at least

95 percent of maximum dry density as determined by the modified Proctor moisture-density relationship test *(ASTM D1557)*. Compacted onsite fill soils should be covered shortly after placement.

9.6.1.2 Imported Granular Material

Imported granular material should consist of pit- or quarry-run rock, crushed rock, or crushed gravel and sand. The imported granular material should also be durable, angular, and fairly well graded between coarse and fine material; should have less than 5 percent fines (material passing the U.S. Standard No. 200 sieve) by dry weight; and should have at least two mechanically fractured faces. Imported granular material should be placed in loose lifts not exceeding 12 inches in depth and compacted to at least 95 percent of maximum dry density as determined by the modified Proctor moisture-density relationship test *(ASTM D1557)*. During wet-weather conditions or where wet subgrade conditions are present, the initial loose lift of granular fill should be approximately 18 inches thick and should be compacted with a smooth-drum roller operating in static mode.

9.6.1.3 Stabilization Material

Stabilization material should consist of durable, 4- or 6-inch-minus pit- or quarry-run rock, crushed rock, or crushed gravel and sand that is free of organics and other deleterious material. The material should have a maximum particle size of 6 inches with less than 5 percent by dry weight passing the U.S. Standard No. 4 sieve. The material should have at least two mechanically fractured faces.

Stabilization material should be placed in loose lifts between 12 and 24 inches thick and be compacted to a firm, unyielding condition. Equipment with vibratory action should not be used when compacting stabilization material over wet, fine-textured soils. If stabilization material is used to stabilize soft subgrade below pavement or construction haul roads, a subgrade geotextile should be placed as a separation barrier between the soil subgrade and the stabilization material.

9.6.1.4 Trench Backfill

Trench backfill placed below, adjacent to, and up to at least 12 inches above utility lines (i.e., the pipe zone) should consist of well-graded granular material meeting *WSDOT 9-03.12(3)* specifications for *Gravel Backfill for Pipe Zone Bedding*. Pipe zone backfill should be compacted to at least 90 percent of maximum dry density, as determined by the modified Proctor moisture-density relationship test *(ASTM D1557),* or as required by the local jurisdictional agency or pipe manufacturer.

Within structural areas (below pavement and building pads), trench backfill above the pipe zone should consist of *WSDOT 9-03.19 Bank Run Gravel for Trench Backfill* or *WSDOT 9-03.14(2) Select Borrow* with a maximum particle size of 2 ½-inches. Trench backfill material within 18 inches of the top of utility pipes should be hand compacted (i.e., no heavy compaction equipment). Remaining trench backfill should be compacted to at least 95 percent of the maximum dry density as determined by the modified Proctor moisture-density relationship test *(ASTM D1557),* or as required by the local jurisdictional agency or pipe manufacturer.

Outside of structural areas, trench backfill placed above the pipe zone should be compacted to at least 90 percent of the maximum dry density as determined by the modified Proctor moisture-density relationship test *(ASTM D1557),* or as required by the local jurisdictional agency or pipe manufacturer.

9.6.1.5 Floor Slab Base Aggregate

Base aggregate for building floor slabs should consist of 1 ¼"-minus crushed aggregate meeting *WSDOT 9-03.9(3)* specifications for *Crushed Surfacing*. Slab base aggregate should be compacted to at least at least 95 percent of the maximum dry density as determined by the modified Proctor moisture-density relationship test *(ASTM D1557)*.

9.6.2 Pavement Base Aggregate

Base aggregate for pavement should consist of 1 ¼"-minus crushed aggregate meeting *WSDOT 9-03.9(3)* specifications for *Crushed Surfacing*. Pavement base aggregate should be compacted to at least at least 95 percent of the maximum dry density as determined by the modified Proctor moisture-density relationship test *(ASTM D1557)*.

9.6.2.1 Retaining Wall Backfill

Backfill material placed behind retaining walls and extending a horizontal distance of ½ H, where H is the height of the retaining wall, should consist of free-draining granular material meeting *WSDOT 9-03.12(2)* specifications for *Gravel Backfill for Walls*. The wall backfill should be separated from structural fill, native soil, and/or topsoil using a geotextile fabric that meets the specifications provided below for drainage geotextiles.

Wall backfill located within a horizontal distance of 3 feet from the face of a retaining wall should be compacted to 90 percent of the maximum dry density, as determined by *ASTM D1557*. Backfill placed within 3 feet of the wall should be compacted in loose lifts less than 6 inches thick using hand-operated tamping equipment (such as a jumping jack or vibratory plate compactor). Remaining wall backfill should be compacted to at least 95 percent of the maximum dry density, as determined by *ASTM D1557*.

9.6.2.2 Retaining Wall Leveling Pad

Crushed aggregate used as a leveling pad for retaining wall footings should consist of 1 ¼"-minus crushed aggregate meeting *WSDOT 9-03.9(3)* specifications for *Crushed Surfacing*. The leveling pad material should be compacted to at least 95 percent of the maximum dry density as determined by the modified Proctor moisture-density relationship test *(ASTM D1557)*.

9.6.2.3 Drain Rock

Drain rock should consist of angular, granular material with a maximum particle size of 2 inches and less than 2 percent by weight passing the No. 200 sieve. Drain rock should be free of roots, organic debris, and other unsuitable material and should have at least two mechanically fractured faces. Drain rock should be compacted to a firm, unyielding condition. Drain rock should be completely wrapped in a geotextile drainage fabric meeting the requirements presented below.

9.6.3 Geotextile Fabric

9.6.3.1 Subgrade Geotextile

Subgrade geotextile should meet the specifications provided in *WSDOT 9-33.2(1), Table 3, Geotextile for Separation or Soil Stabilization*. The geotextile should be installed in accordance with the manufacturer's recommendations. A minimum initial aggregate base lift of 6 inches is required over geotextiles. All stabilization material should be underlain by a subgrade geotextile.

9.6.3.2 *Drainage Geotextile*

Subgrade geotextile should meet the specifications provided in *WSDOT 9-33.2(1), Table 2, Geotextile for Underground Drainage Filtration Properties*. The AOS should be between the No. 70 and No. 100 sieve. The water permittivity should be greater than 1.5/sec. The geotextile should be installed in accordance with the manufacturer's recommendations. A minimum initial aggregate base lift of 6 inches is required over geotextiles.

9.6.4 Geotextile Fabric

9.6.4.1 *Subgrade Geotextile*

Subgrade geotextile should meet the specifications provided in *WSDOT 9-33.2(1), Table 3, Geotextile for Separation or Soil Stabilization*. The geotextile should be installed in accordance with the manufacturer's recommendations. A minimum initial aggregate base lift of 6 inches is required over geotextiles. All stabilization material should be underlain by a subgrade geotextile.

9.6.4.2 *Drainage Geotextile*

Subgrade geotextile should meet the specifications provided in *WSDOT 9-33.2(1), Table 2, Geotextile for Underground Drainage Filtration Properties*. The AOS should be between the No. 70 and No. 100 sieve. The water permittivity should be greater than 1.5/sec. The geotextile should be installed in accordance with the manufacturer's recommendations. A minimum initial aggregate base lift of 6 inches is required over geotextiles.

9.6.5 Pavement

9.6.5.1 *Asphaltic Concrete*

Asphaltic concrete should consist of HMA Class ½" adhering to *WSDOT 9-03.8(6), HMA Proportions of Materials*. The asphalt binder should consist of PG 58-22 meeting *WSDOT 9-02.1(4), Performance Graded (PG) Asphalt Binder*. Asphalt should be compacted to 91 percent of the theoretical maximum density as determined by *ASTM D2041*. Minimum and maximum asphalt lift thicknesses should be 2 and 3 inches, respectively. Nuclear gauge density testing should be conducted to verify adherence to recommended specifications. Testing frequency should be in accordance with WSDOT and City of Camas specifications.

9.7 EROSION CONTROL MEASURES

Soil at this site is susceptible to erosion by wind and water; therefore, erosion control measures should be carefully planned and installed before construction begins. Surface water runoff should be collected and directed away from sloped areas to prevent water from running down the slope face. Measures that can be employed to reduce erosion include the use of silt fences, hay bales, buffer zones of natural growth, sedimentation ponds, and granular haul roads. All erosion control methods should be in accordance with local jurisdiction standards.

10.0 OBSERVATION OF CONSTRUCTION

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

11.0 CONCLUSIONS AND LIMITATIONS

This geotechnical site investigation report was prepared in accordance with accepted standard conventional principles and practices of geotechnical engineering. This investigation pertains only to material tested and observed as of the date of this report and is based upon proposed site development as described in the text herein. This report is a professional opinion containing recommendations established by engineering interpretations of subsurface soils based upon conditions observed during site exploration. Soil conditions may differ between tested locations or over time. Slight variations may produce impacts to the performance of structural facilities if not adequately addressed. This underscores the importance of diligent QA/QC construction observation and testing to verify soil conditions are as anticipated in this report.

Therefore, this report contains several recommendations for field observation and testing by Columbia West personnel during construction activities. Columbia West cannot accept responsibility for deviations from recommendations described in this report. Future performance of structural facilities is often related to the degree of construction observation by qualified personnel. These services should be performed to the full extent recommended.

This report is not an environmental assessment and should not be construed as a representative warranty of site subsurface conditions. The discovery of adverse environmental conditions, or subsurface soils that deviate from those described in this report, should immediately prompt further investigation. The above statements are in lieu of all other statements expressed or implied.

♦ ♦ ♦

Sincerely,
Columbia West Engineering, Inc.

Daniel E. Lehto, PE, GE
Principal

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PROFESSIONAL ENGINEER

10-11-23

Expires: 6-5-25

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Document ID: Webberly Development Geotechnical Report

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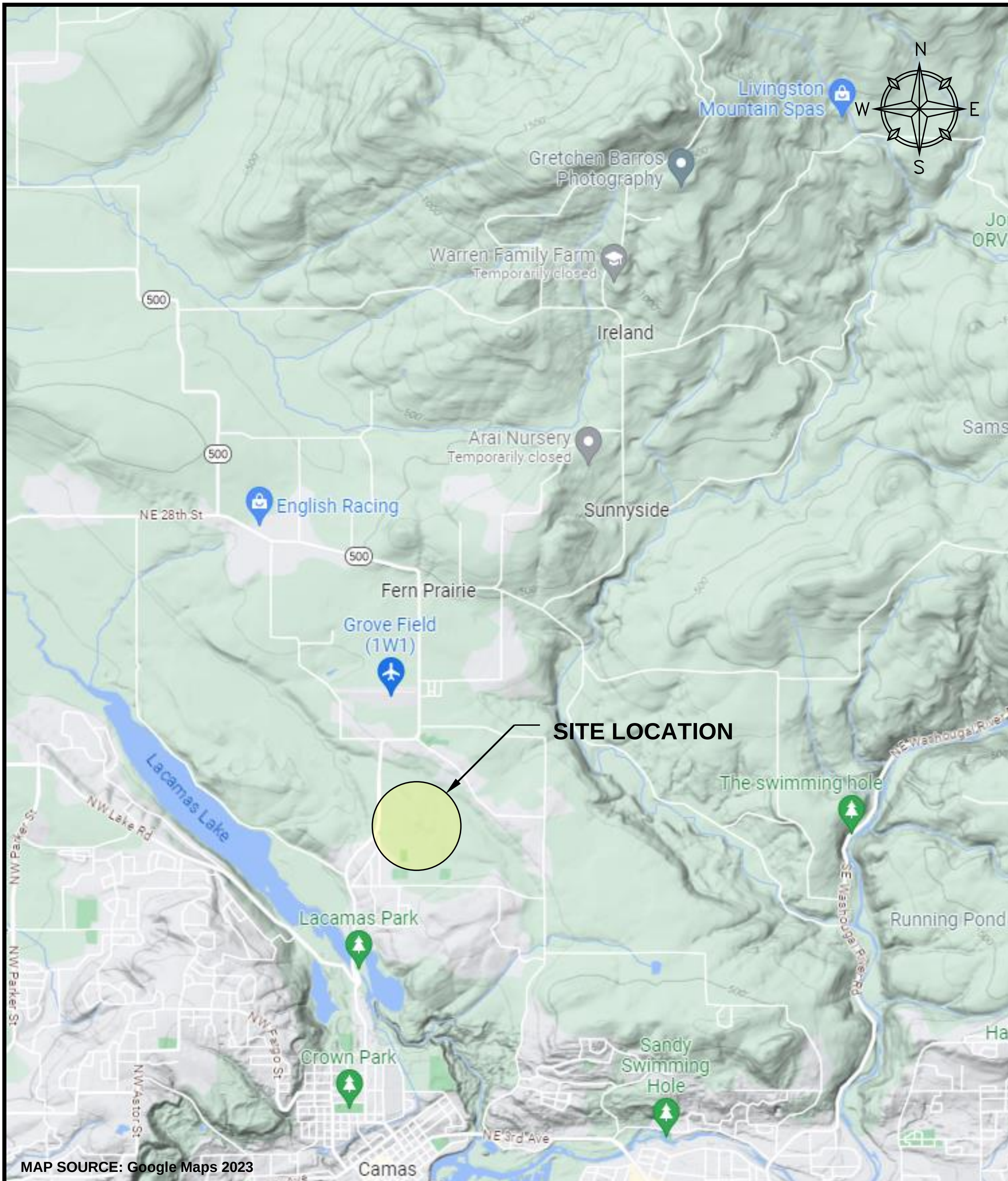

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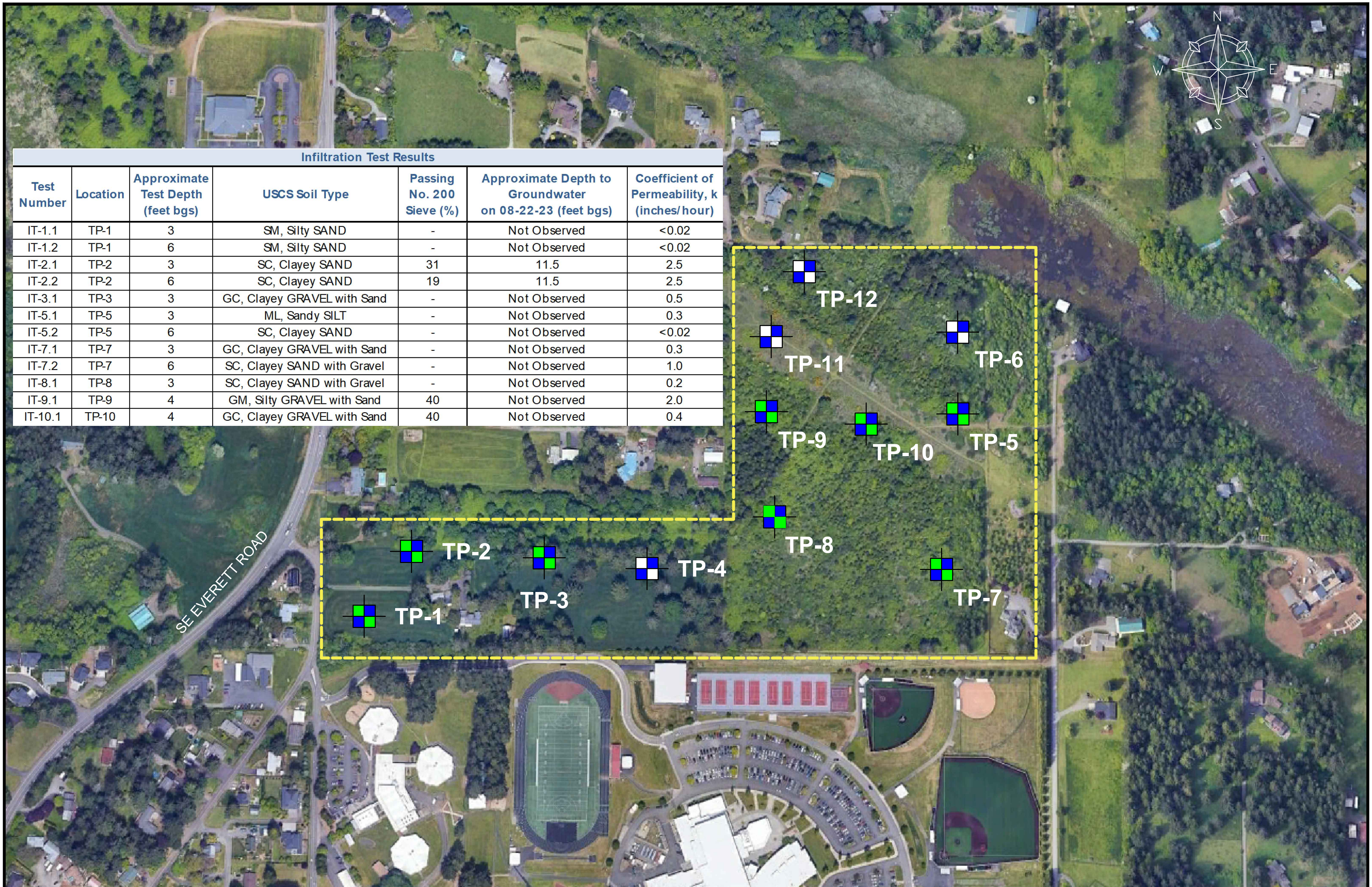
Job No: HSR-3-01-1
Date: 09/19/23
Drawn: SSC
Checked: ASR

SITE LOCATION MAP
WEBBERLEY DEVELOPMENT
CAMAS, WASHINGTON

FIGURE 1



Infiltration Test Results

Test Number	Location	Approximate Test Depth (feet bgs)	USCS Soil Type	Passing No. 200 Sieve (%)	Approximate Depth to Groundwater on 08-22-23 (feet bgs)	Coefficient of Permeability, k (inches/hour)
IT-1.1	TP-1	3	SM, Silty SAND	-	Not Observed	<0.02
IT-1.2	TP-1	6	SM, Silty SAND	-	Not Observed	<0.02
IT-2.1	TP-2	3	SC, Clayey SAND	31	11.5	2.5
IT-2.2	TP-2	6	SC, Clayey SAND	19	11.5	2.5
IT-3.1	TP-3	3	GC, Clayey GRAVEL with Sand	-	Not Observed	0.5
IT-5.1	TP-5	3	ML, Sandy SILT	-	Not Observed	0.3
IT-5.2	TP-5	6	SC, Clayey SAND	-	Not Observed	<0.02
IT-7.1	TP-7	3	GC, Clayey GRAVEL with Sand	-	Not Observed	0.3
IT-7.2	TP-7	6	SC, Clayey SAND with Gravel	-	Not Observed	1.0
IT-8.1	TP-8	3	SC, Clayey SAND with Gravel	-	Not Observed	0.2
IT-9.1	TP-9	4	GM, Silty GRAVEL with Sand	40	Not Observed	2.0
IT-10.1	TP-10	4	GC, Clayey GRAVEL with Sand	40	Not Observed	0.4

APPROXIMATE SITE BOUNDARY

APPROXIMATE LOCATION OF TEST PIT

APPROXIMATE LOCATION OF TEST PIT WITH INFILTRATION

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Date: 08/23/23
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Checked: DEL

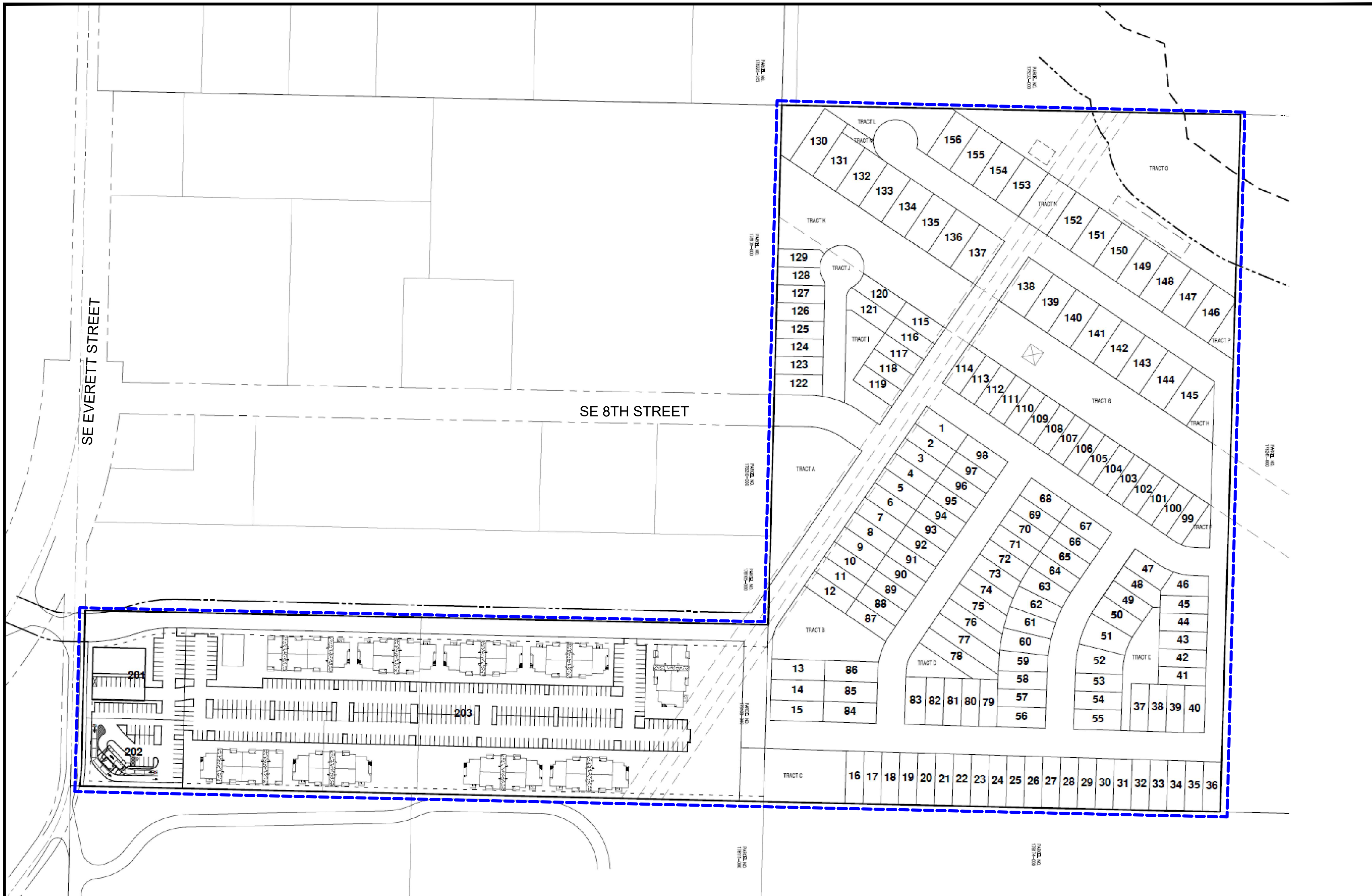
EXPLORATION LOCATION MAP

WEBBERLEY DEVELOPMENT

NOTES:
1. APPROXIMATE SITE LOCATION: EAST AND WEST OF 921 SE GARDNER ROAD IN CAMAS, WASHINGTON 98607.
2. SITE CONSISTS OF TAX PARCEL NOS. 178159000,178169000, 178108000, AND 178140000 TOTALING APPROXIMATELY 36.12 ACRES.
3. AERIAL PHOTO SOURCED FROM GOOGLE EARTH.
4. EXPLORATION LOCATIONS ARE APPROXIMATE AND NOT SURVEYED.
5. TEST PITS BACKFILLED WITH ONSITE SOILS ON AUGUST 23, 2023.

FIGURE

2



APPROXIMATE SITE BOUNDARY

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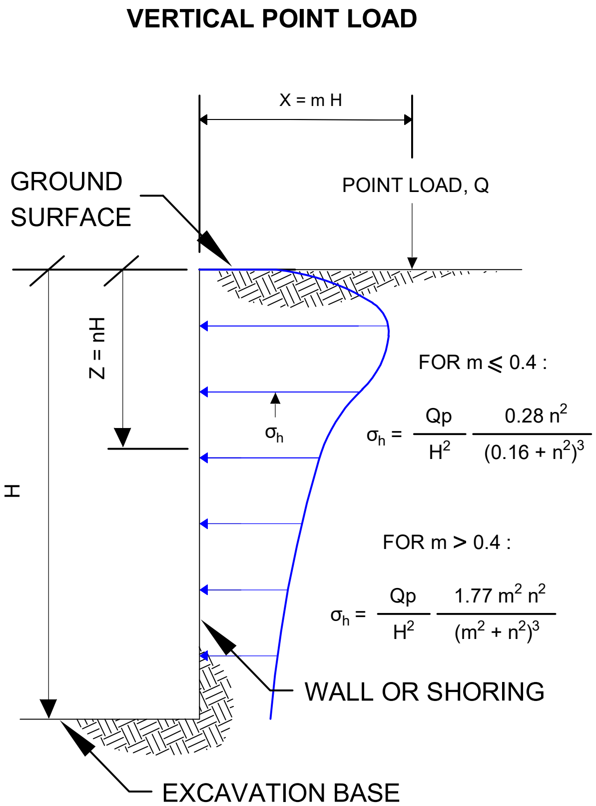
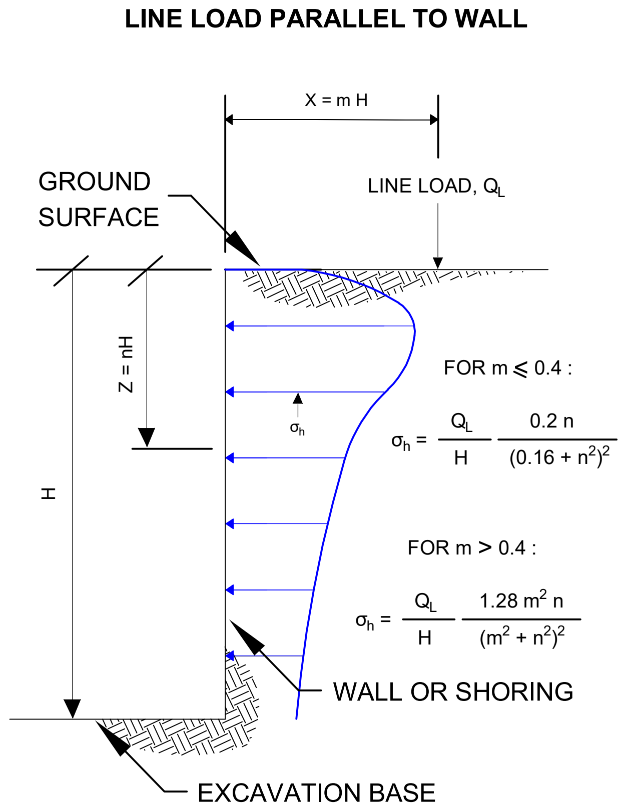
Checked: ASR

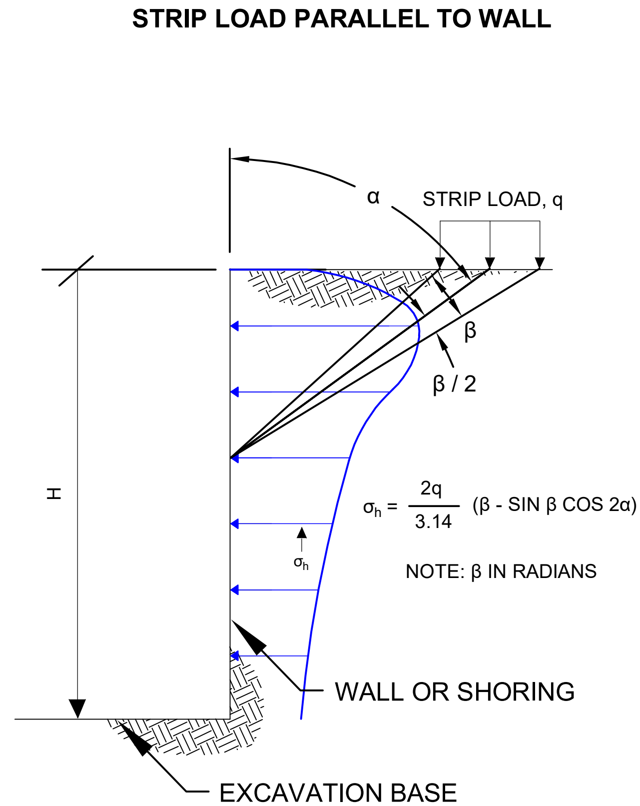
PRELIMINARY SITE PLAN
WEBBERLEY DEVELOPMENT

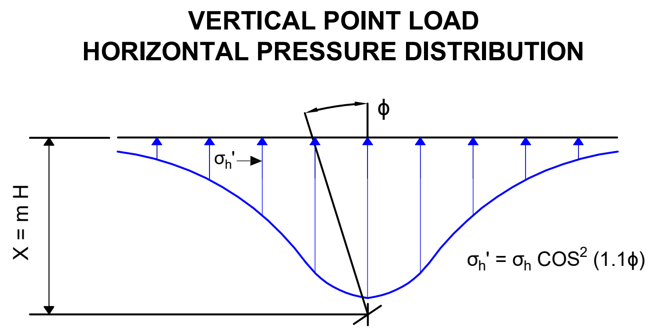
NOTES:
1. APPROXIMATE SITE LOCATION: EAST AND WEST OF 921 SE GARDNER ROAD IN CAMAS, WASHINGTON 98607.
2. SITE CONSISTS OF TAX PARCEL NOS. 178159000,178169000, 178108000, AND 178140000 TOTALING APPROXIMATELY 36.12 ACRES.
3. SITE PLAN PROVIDED BY AKS ENGINEERING.

FIGURE
2A









NOTES:
1. FIGURE SHOULD BE USED JOINTLY WITH RECOMMENDATIONS PRESENTED IN THE REPORT TEXT.
2. LATERAL EARTH PRESSURES ASSUME RIGID WALLS WITH BACKFILL MATERIALS HAVING A POISSON'S RATIO OF 0.5.
3. TOTAL LATERAL EARTH PRESSURES RESULTING FROM COMBINED LOADS MAY BE CALCULATED USING SUPERPOSITION.
4. DRAWING IS NOT TO SCALE.

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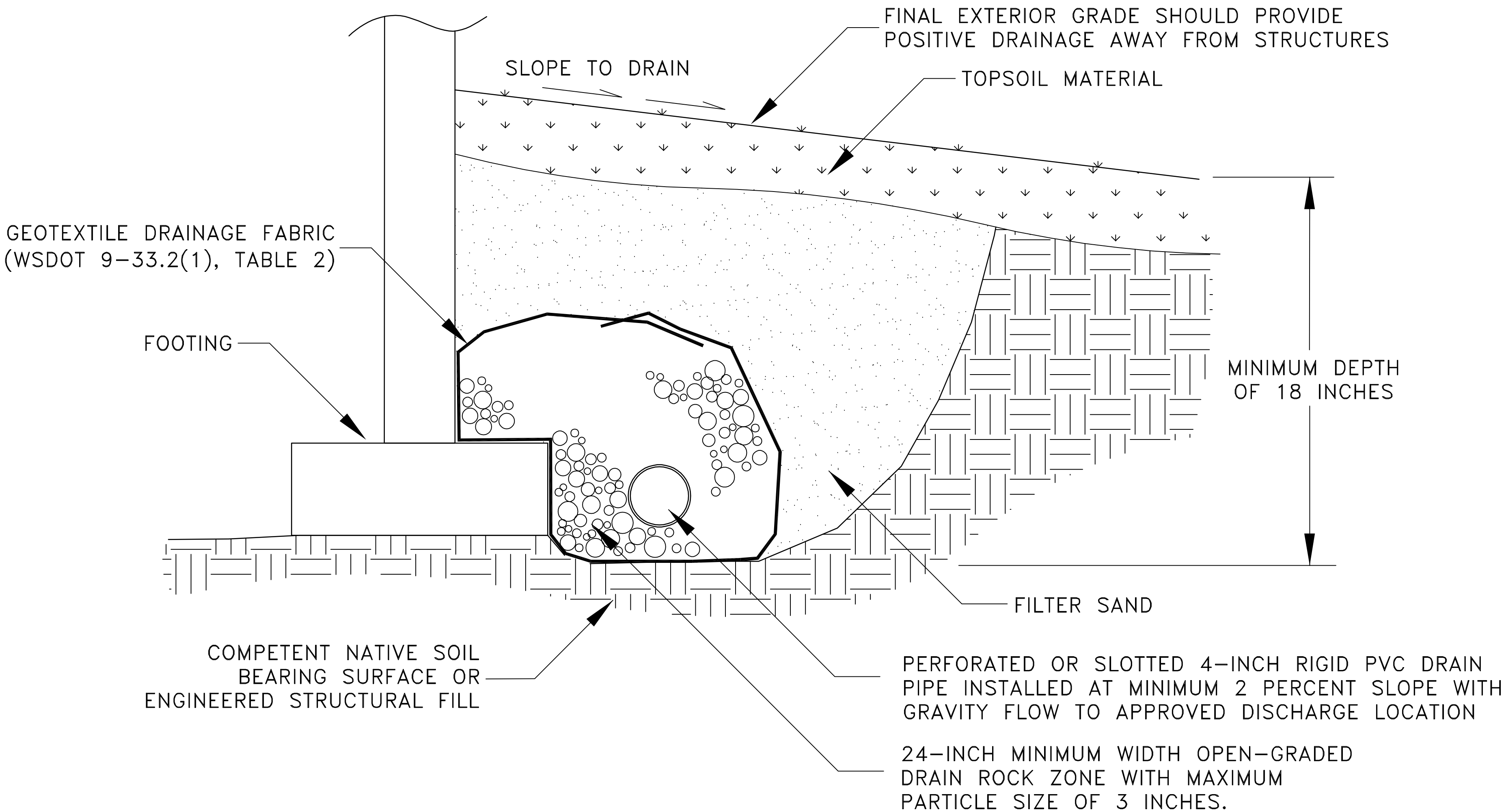
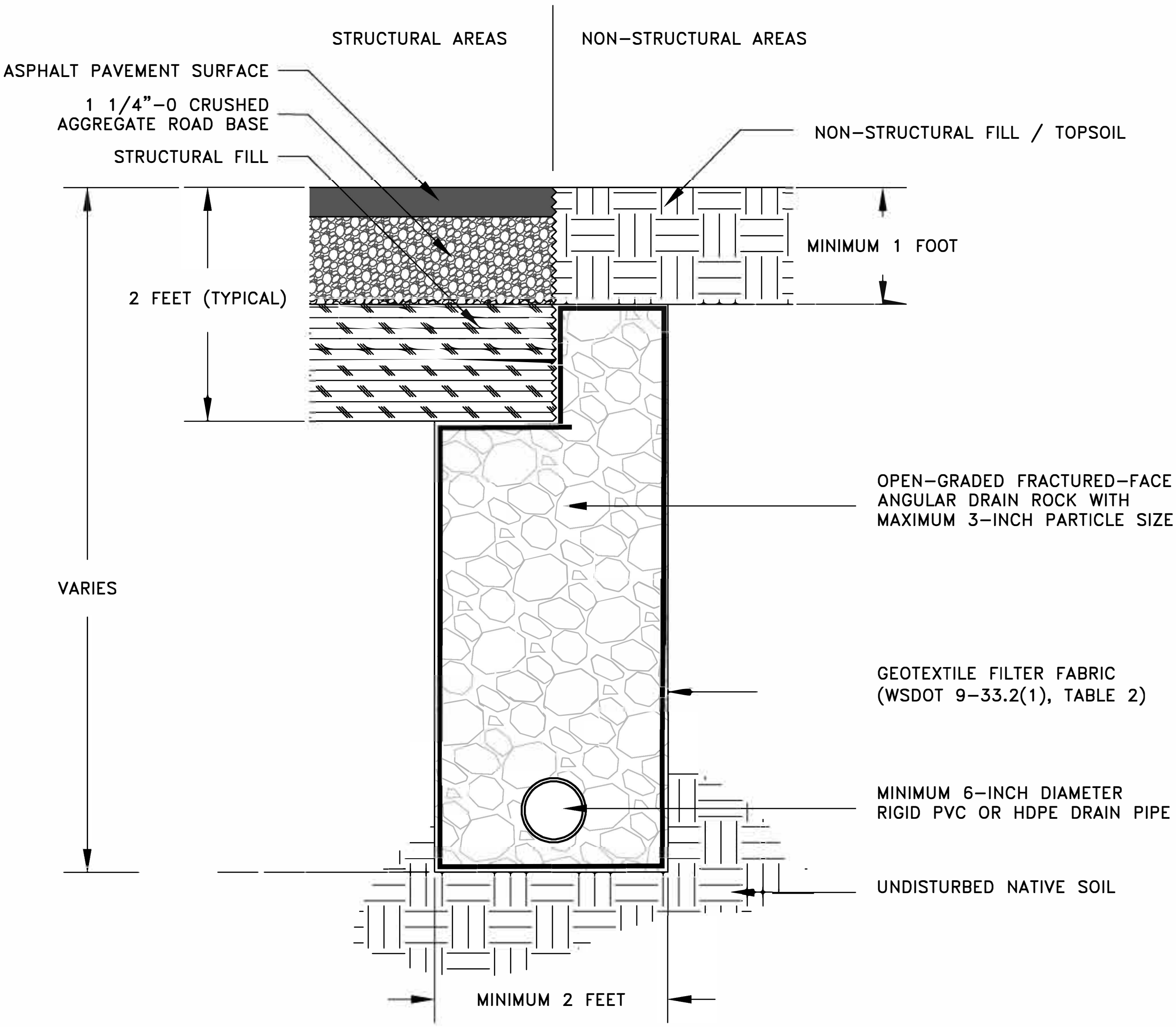
SURCHARGE–INDUCED LATERAL EARTH PRESSURES

FIGURE 3



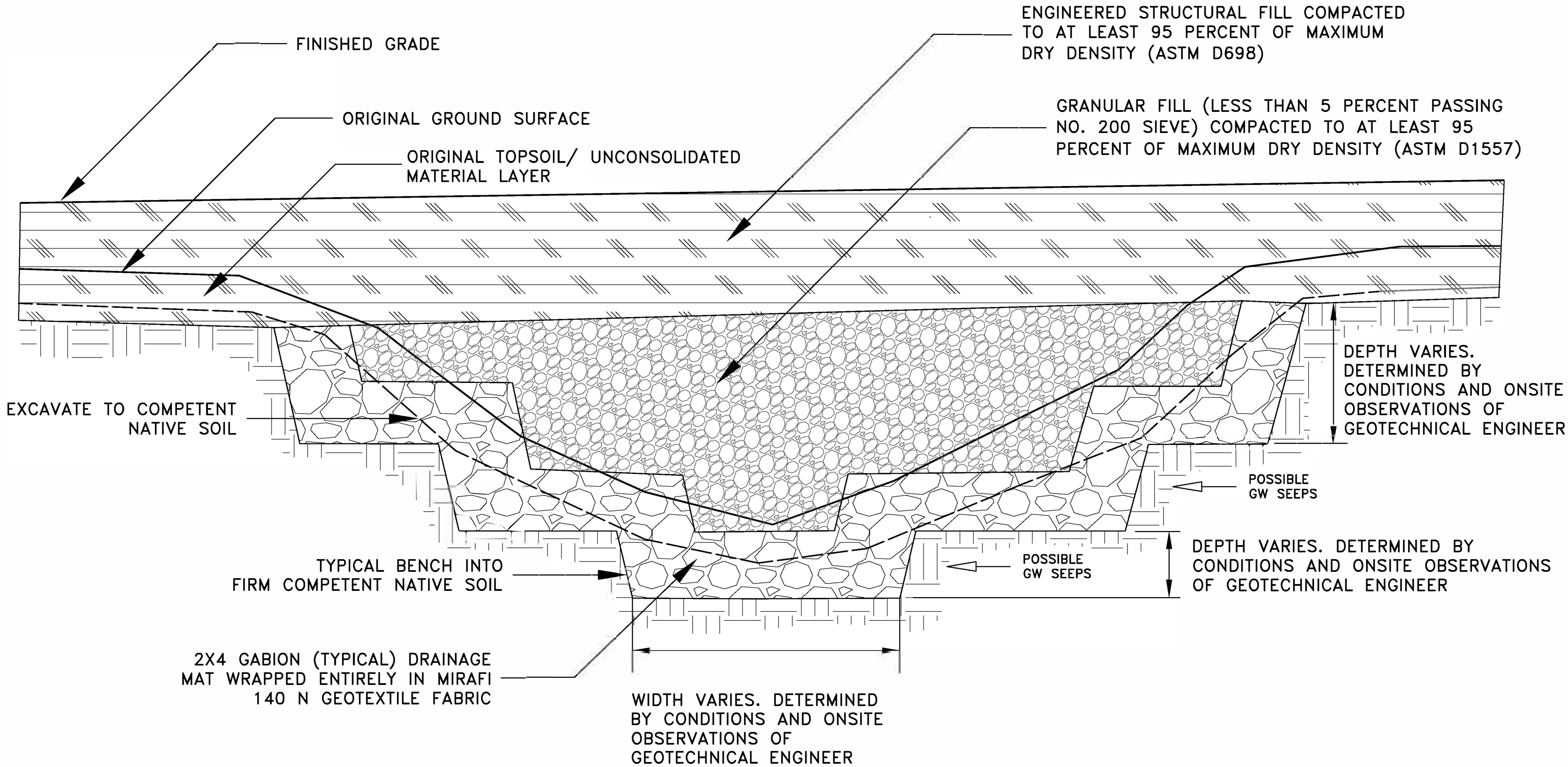
Geotechnical ▪ Environmental ▪ Special Inspections **Columbia West** Engineering, Inc	TYPICAL PERIMETER FOOTING DRAIN DETAIL	NOTES: 1. DRAWING IS NOT TO SCALE. 2. DRAWING REPRESENTS TYPICAL PERIMETER FOOTING DRAIN DETAIL AND MAY NOT BE SITE-SPECIFIC.	FIGURE 4



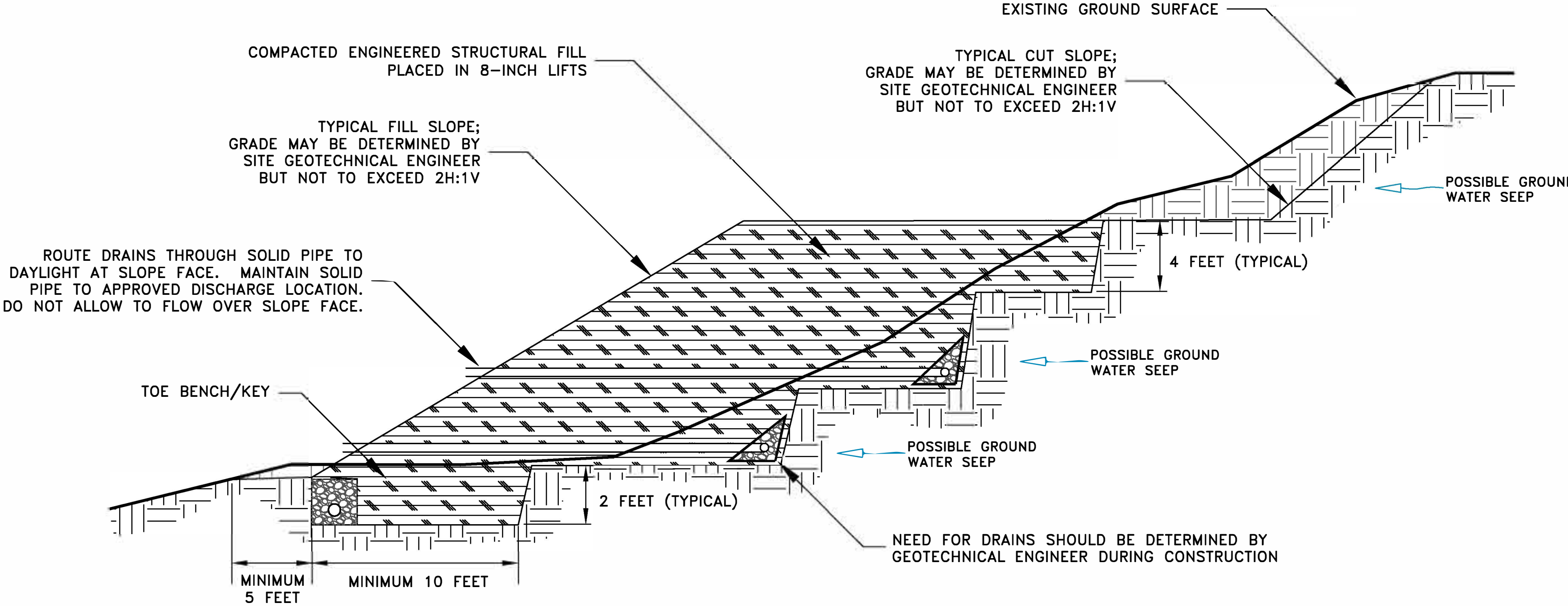
NOTE: LOCATION, INVERT ELEVATION, DEPTH OF TRENCH, AND EXTENT OF PERFORATED PIPE REQUIRED MAY BE MODIFIED BY THE GEOTECHNICAL ENGINEER DURING CONSTRUCTION BASED UPON FIELD OBSERVATION AND SITE-SPECIFIC SOIL CONDITIONS.

Geotechnical ■ Environmental ■ Special Inspections Columbia West Engineering, Inc	TYPICAL PERFORATED DRAINPIPE TRENCH DETAIL	NOTES: 1. DRAWING IS NOT TO SCALE. 2. DRAWING REPRESENTS TYPICAL PERFORATED DRAIN PIPE TRENCH DETAIL AND MAY NOT BE SITE-SPECIFIC.	FIGURE 5

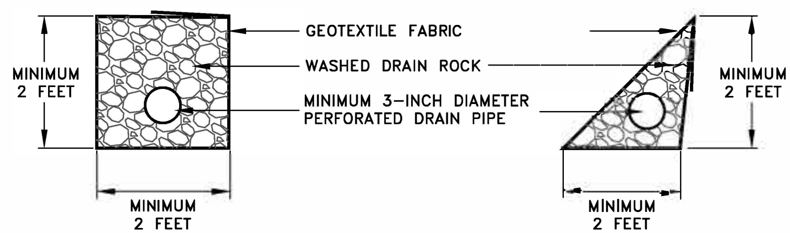
TYPICAL DRAINAGE MAT CROSS-SECTION



Geotechnical ▪ Environmental ▪ Special Inspections Columbia West Engineering, Inc	TYPICAL DRAINAGE MAT CROSS SECTION	NOTES: 1. DRAWING IS NOT TO SCALE. 2. DRAWING REPRESENTS TYPICAL DRAINAGE MAT SECTION AND MAY NOT BE SITE-SPECIFIC.	FIGURE 6



TYPICAL DRAIN SECTION DETAIL



DRAIN SPECIFICATIONS

GEOTEXTILE FABRIC SHALL MEET WSDOT 9-33.2(1), TABLE 2, GEOTEXTILE FOR UNDERGROUND DRAINAGE FILTRATION PROPERTIES WITH AOS BETWEEN No. 70 AND No. 100 SIEVE. WATER PERMITIVITY SHOULD BE GREATER THAN 1.5/SEC.

WASHED DRAIN ROCK SHALL BE OPEN-GRADED ANGULAR DRAIN ROCK WITH LESS THAN 2 PERCENT PASSING THE No. 200 SIEVE AND A MAXIMUM PARTICLE SIZE OF 2 INCHES.

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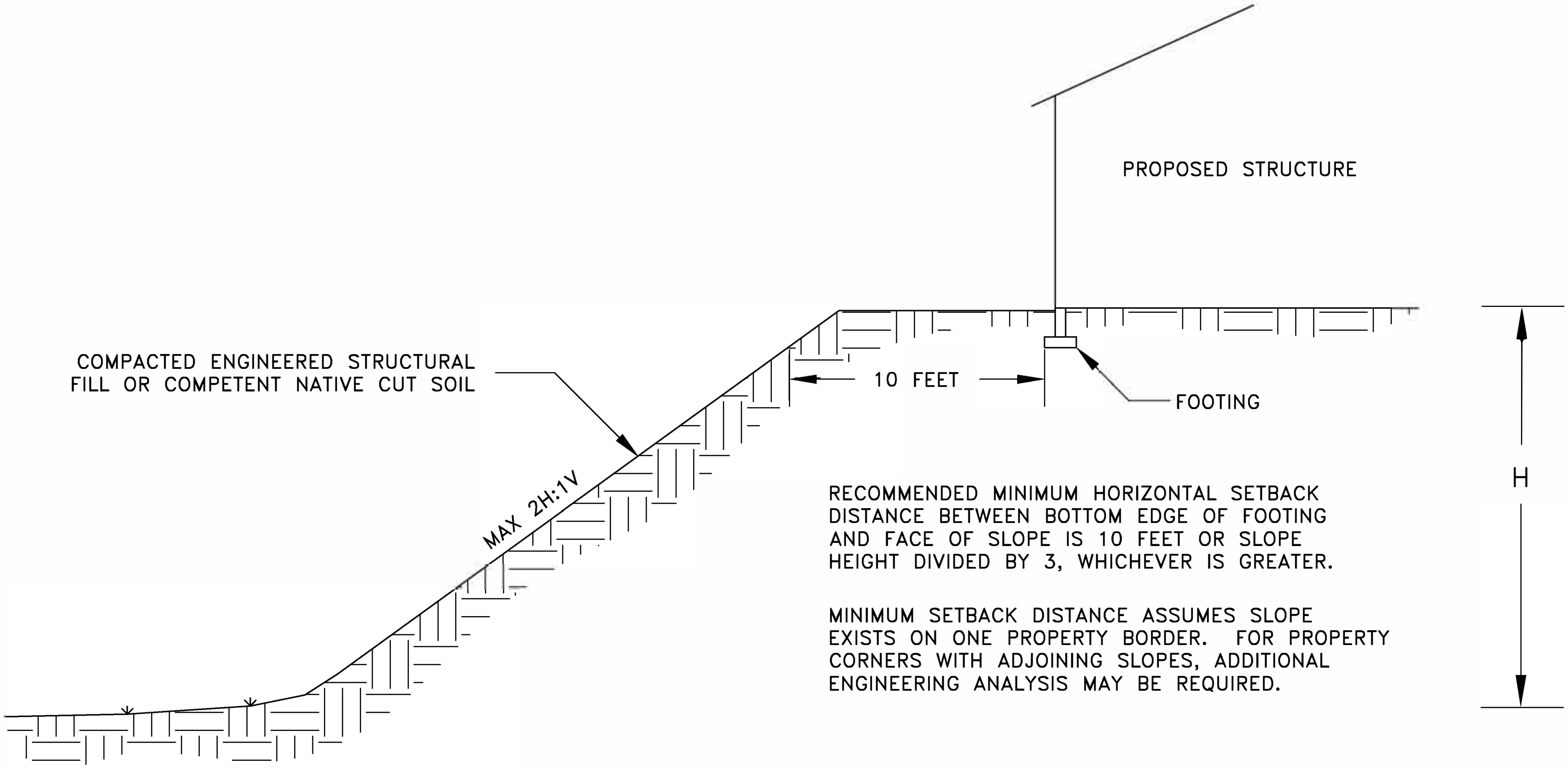
TYPICAL CUT AND FILL
SLOPE CROSS SECTION

NOTES:
1. DRAWING IS NOT TO SCALE.
2. DRAWING REPRESENTS TYPICAL CUT AND FILL SLOPE CROSS SECTION AND MAY NOT BE SITE-SPECIFIC.

FIGURE
7



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TYPICAL MINIMUM FOUNDATION
SLOPE SETBACK DETAIL

NOTES:
1. DRAWING IS NOT TO SCALE.
2. SLOPES AND PROFILES SHOWN ARE APPROXIMATE.
3. DRAWING REPRESENTS TYPICAL FOUNDATION SETBACK DETAIL AND MAY NOT BE SITE-SPECIFIC.

FIGURE 8

APPENDIX A
SUBSURFACE EXPLORATION PROGRAM
FIELD EXPLORATIONS

GENERAL

We explored subsurface conditions at the site by excavating eight test pits (TP-1 through TP-12) to depths between 6 and 13 feet BGS. Excavation services were provided by L&S Contractors of Battle Ground, Washington on August 23, 2023. The test pit locations are shown on Figures 2. The test pit logs are presented in this appendix.

SOIL SAMPLING

Representative grab samples of the soil observed in the test pit explorations were obtained from the walls and/or base of the test pits using the excavator bucket.

SOIL CLASSIFICATION

The soil samples were classified in accordance with the Unified Soil Classification System presented in Appendix C. The exploration log indicates the depths at which the soils or their characteristics change, although the change actually could be gradual. If the change occurred between sample locations, the depth was interpreted. Classifications are shown on the exploration log.

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TEST PIT LOG

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PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Webberley Development	HSR Development		HSR-3-01-01	TP-1
PROJECT LOCATION Camas, Washington	CONTRACTOR L&S Contractors	EQUIPMENT Excavator	TECHNICIAN SSC/SMF	DATE 8/23/23
TEST PIT LOCATION See Figure 2	GROUNDWATER DEPTH Groundwater not observed.		START TIME 0820	FINISH TIME 1120

Depth (feet)	Sample Field ID	SCS Soil Survey Description	AASHTO Soil Type	USCS Soil Type	Graphic Log	LITHOLOGIC DESCRIPTION AND REMARKS	Moisture Content (%)	Passing No. 200 Sieve (%)	Liquid Limit	Plasticity Index	Infiltration Testing
0						Approximately 3 inches of root zone underlain by 6 to 8 inches of topsoil.					
	TP1.1			ML		Light brown sandy SILT with gravel, damp, medium stiff, low plasticity. Intermixed rounded 1/2 to 2 inch rounded gravel.					IT-1.1 Depth = 3 feet
5	TP1.2			SC		Red-brown clayey SAND with gravel, moist, medium dense, low to medium plasticity.					IT-1.2 Depth = 6 feet
				GC		WEATHERED CONGLOMERATE: Red and brown clayey GRAVEL with sand, dense, damp to moist, 1 to 6 inch subrounded gravel.					
10						Bottom of test pit at 11 feet BGS. Groundwater not observed.					
15											

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PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Webberley Development	HSR Development		HSR-3-01-01	TP-2
PROJECT LOCATION	CONTRACTOR	EQUIPMENT	TECHNICIAN	DATE
Camas, Washington	L&S Contractors	Excavator	SSC/SMF	8/23/23
TEST PIT LOCATION	GROUNDWATER DEPTH		START TIME	FINISH TIME
See Figure	Groundwater observed at 11.5 feet BGS.		0855	1125

Depth (feet)	Sample Field ID	SCS Soil Survey Description	AASHTO Soil Type	USCS Soil Type	Graphic Log	LITHOLOGIC DESCRIPTION AND REMARKS	Moisture Content (%)	Passing No. 200 Sieve (%)	Liquid Limit	Plasticity Index	Infiltration Testing
0						Approximately 3 inches of root zone underlain by 6 inches of topsoil.					
				SC		Light brown clayey SAND with gravel, damp, medium stiff, low plasticity. Intermixed rounded 1/2 to 2 inch rounded gravel.					
	TP2.1		A-2-7(2)			Intermixed rounded 1/2 to 2 inch rounded gravel.	22	31	53	25	IT-2.1 Depth = 3 feet
5											
	TP2.2					Red-brown, increase in plasticity.	32	19			IT-2.2 Depth = 6 feet
10				GC		WEATHERED CONGLOMERATE: Gray clayey GRAVEL with sand, moist to wet, medium dense, medium plasticity.					
						Bottom of test pit at 13 feet BGS. Groundwater observed 11.5 feet BGS.					
15											

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TEST PIT LOG

PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Webberley Development	HSR Development		HSR-3-01-01	TP-3
PROJECT LOCATION: Camas, Washington	CONTRACTOR: L&S Contractors	EQUIPMENT: Excavator	TECHNICIAN: SSC/SMF	DATE: 8/23/23
TEST PIT LOCATION: See Figure	GROUNDWATER DEPTH: Groundwater not observed.		START TIME: 0935	FINISH TIME: 1135

Depth (feet)	Sample Field ID	SCS Soil Survey Description	AASHTO Soil Type	USCS Soil Type	Graphic Log	LITHOLOGIC DESCRIPTION AND REMARKS	Moisture Content (%)	Passing No. 200 Sieve (%)	Liquid Limit	Plasticity Index	Infiltration Testing
0						Approximately 3 inches of root zone underlain by 6 inches of topsoil.					
				ML		Light brown sandy SILT with gravel, damp, stiff. 1/2 to 3 inch rounded gravels.					
	TP3.1			GC		Red and brown clayey GRAVEL with sand, moist, dense.					IT-3.1 Depth = 3 feet
5											
10											
						Excavator comment: digging like dense soil.					
						Bottom of test pit at 13 feet BGS. Groundwater not observed.					
15											

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TEST PIT LOG

PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Webberley Development	HSR Development		HSR-3-01-01	TP-4
PROJECT LOCATION	CONTRACTOR	EQUIPMENT	TECHNICIAN	DATE
Camas, Washington	L&S Contractors	Excavator	SSC/SMF	8/23/23
TEST PIT LOCATION	GROUNDWATER DEPTH		START TIME	FINISH TIME
See Figure	Groundwater not observed.		1030	1100

Depth (feet)	Sample Field ID	SCS Soil Survey Description	AASHTO Soil Type	USCS Soil Type	Graphic Log	LITHOLOGIC DESCRIPTION AND REMARKS	Moisture Content (%)	Passing No. 200 Sieve (%)	Liquid Limit	Plasticity Index	Infiltration Testing
0						Approximately 3 inches of root zone underlain by 6 inches of topsoil.					
				ML		Light brown sandy SILT with gravel, stiff, damp, with 1/2 to 3 inch rounded gravel.					
				GC		Red and brown clayey GRAVEL with sand, dense, damp to moist, 1 to 6 inch subrounded gravel.					
5											
10						18 inch rounded boulder at 10 feet.					
						Excavator comment: digging like dense soil.					
						Bottom of test pit at 13 feet BGS. Groundwater not observed.					
15											

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TEST PIT LOG

PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Webberley Development	HSR Development		HSR-3-01-01	TP-5
PROJECT LOCATION	CONTRACTOR	EQUIPMENT	TECHNICIAN	DATE
Camas, Washington	L&S Contractors	Excavator	SSC/SMF	8/23/23
TEST PIT LOCATION	GROUNDWATER DEPTH		START TIME	FINISH TIME
See Figure	Groundwater not observed.		1155	1600

Depth (feet)	Sample Field ID	SCS Soil Survey Description	AASHTO Soil Type	USCS Soil Type	Graphic Log	LITHOLOGIC DESCRIPTION AND REMARKS	Moisture Content (%)	Passing No. 200 Sieve (%)	Liquid Limit	Plasticity Index	Infiltration Testing
0						Approximately 4 inches of root zone underlain by 10 inches of topsoil.					
	TP5.1			ML		Brown sandy SILT with gravel, damp, stiff, low plasticity.					IT-5.1 Depth = 3 feet
5	TP5.2			SC		Red and brown clayey SAND with gravel, dense, damp to moist, 1 to 6 inch subrounded gravel.					IT-5.2 Depth = 6 feet
10				GC		WEATHERED CONGLOMERATE: Dark brown clayey GRAVEL with sand, moist, medium dense to dense.					
						Bottom of test pit at 12.5 feet BGS. Groundwater not observed.					
15											

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TEST PIT LOG

PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Webberley Development	HSR Development		HSR-3-01-01	TP-6
PROJECT LOCATION	CONTRACTOR	EQUIPMENT	TECHNICIAN	DATE
Camas, Washington	L&S Contractors	Excavator	SSC/SMF	8/23/23
TEST PIT LOCATION	GROUNDWATER DEPTH		START TIME	FINISH TIME
See Figure 2	Groundwater not observed.		1230	1300

Depth (feet)	Sample Field ID	SCS Soil Survey Description	AASHTO Soil Type	USCS Soil Type	Graphic Log	LITHOLOGIC DESCRIPTION AND REMARKS	Moisture Content (%)	Passing No. 200 Sieve (%)	Liquid Limit	Plasticity Index	Infiltration Testing
0						Approximately 4 inches of root zone underlain by 8 inches of topsoil.					
				ML		Brown sandy SILT, damp, medium stiff, low plasticity.					
				SC		WEATHERED CONGLOMERATE: Yellow and brown clayey SAND, moist, very dense, fines have medium plasticity.					
5											
						Bottom of test pit at 7 feet BGS due to practical refusal on competent conglomerate. Groundwater not observed.					
10											
15											

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TEST PIT LOG

PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Webberley Development	HSR Development		HSR-3-01-01	TP-7
PROJECT LOCATION Camas, Washington	CONTRACTOR L&S Contractors	EQUIPMENT Excavator	TECHNICIAN SSC/SMF	DATE 8/23/23
TEST PIT LOCATION See Figure	GROUNDWATER DEPTH Groundwater not observed.		START TIME 1305	FINISH TIME 1615

Depth (feet)	Sample Field ID	SCS Soil Survey Description	AASHTO Soil Type	USCS Soil Type	Graphic Log	LITHOLOGIC DESCRIPTION AND REMARKS	Moisture Content (%)	Passing No. 200 Sieve (%)	Liquid Limit	Plasticity Index	Infiltration Testing
0						Approximately 4 inches of root zone underlain by 8 inches of topsoil.					
				ML		Brown sandy SILT with gravel, damp, medium stiff, low plasticity, 1/2 to 3 inch rounded gravel.					
	TP7.1			GC		Red and brown clayey GRAVEL with sand, moist, dense.					IT-7.1 Depth = 3 feet
5	TP7.2			SC		WEATHERED CONGLOMERATE: Yellow and brown clayey SAND, moist, very dense, fines have medium plasticity, 1/2 to 6 inch gravel.					IT-7.2 Depth = 6 feet
10						Slow excavation observed from 10 to 11 feet.					
						Bottom of test pit at 11 feet BGS. Groundwater not observed.					
15											

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TEST PIT LOG

PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Webberley Development	HSR Development		HSR-3-01-01	TP-8
PROJECT LOCATION	CONTRACTOR	EQUIPMENT	TECHNICIAN	DATE
Camas, Washington	L&S Contractors	Excavator	SSC/SMF	8/23/23
TEST PIT LOCATION	GROUNDWATER DEPTH		START TIME	FINISH TIME
See Figure	Groundwater not observed.		1348	1630

Depth (feet)	Sample Field ID	SCS Soil Survey Description	AASHTO Soil Type	USCS Soil Type	Graphic Log	LITHOLOGIC DESCRIPTION AND REMARKS	Moisture Content (%)	Passing No. 200 Sieve (%)	Liquid Limit	Plasticity Index	Infiltration Testing
0						Approximately 4 inches of root zone underlain by 8 inches of topsoil.					
				CL		Light brown sandy lean CLAY with minor gravel, damp, stiff, minor rounded gravel.					
				SC		Brown clayey SAND with gravel, damp to moist, denst, low to medium plasticity, 1 to 6 inch sub rounded gravel.					
	TP8.1										IT-8.1 Depth = 3 feet
5											
	TP8.2		A-2-7(0)				19	18	50	22	
10											
						Excavator comment: digging like dense soil.					
						Bottom of test pit at 13 feet BGS. Groundwater not observed.					
15											

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TEST PIT LOG

PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Webberley Development	HSR Development		HSR-3-01-01	TP-9
PROJECT LOCATION	CONTRACTOR	EQUIPMENT	TECHNICIAN	DATE
Camas, Washington	L&S Contractors	Excavator	SSC/SMF	8/23/23
TEST PIT LOCATION	GROUNDWATER DEPTH		START TIME	FINISH TIME
See Figure	Groundwater not observed.		1420	1645

Depth (feet)	Sample Field ID	SCS Soil Survey Description	AASHTO Soil Type	USCS Soil Type	Graphic Log	LITHOLOGIC DESCRIPTION AND REMARKS	Moisture Content (%)	Passing No. 200 Sieve (%)	Liquid Limit	Plasticity Index	Infiltration Testing
0						Approximately 2 inches of root zone underlain by 4 inches of topsoil.					
				SM		Light brown silty SAND with gravel, moist, dense, 1 to 6 inch subrounded gravel.					
	TP9.1						28	40			IT-9.1 Depth = 4 feet
5											
				GC		WEATHERED CONGLOMERATE: Brown clayey GRAVEL with sand, dense, damp to moist, 1 to 6 inch subrounded gravel.					
						2-foot long boulder at 8 feet.					
10											
						Bottom of test pit at 13 feet BGS. Groundwater not observed.					
15											

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TEST PIT LOG

PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Webberley Development	HSR Development		HSR-3-01-01	TP-10
PROJECT LOCATION Camas, Washington	CONTRACTOR L&S Contractors	EQUIPMENT Excavator	TECHNICIAN SSC/SMF	DATE 8/23/23
TEST PIT LOCATION See Figure	GROUNDWATER DEPTH Groundwater not observed.		START TIME 1440	FINISH TIME 1645

Depth (feet)	Sample Field ID	SCS Soil Survey Description	AASHTO Soil Type	USCS Soil Type	Graphic Log	LITHOLOGIC DESCRIPTION AND REMARKS	Moisture Content (%)	Passing No. 200 Sieve (%)	Liquid Limit	Plasticity Index	Infiltration Testing
0						Approximately 4 inches of root zone underlain by 8 inches of topsoil.					
				GM		Brown silty GRAVEL with sand and cobbles, very dense, damp, 1 to 10 inch subangular gravel.					
	TP10.1		A-7-6(3)				11	40	47	19	
	TP10.2						20	41			IT-10.1 Depth = 4 feet
5											
10						Excavator comment: digging like dense soil.					
						Bottom of test pit at 13 feet BGS. Groundwater not observed.					
15											

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TEST PIT LOG

PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Webberley Development	HSR Development		HSR-3-01-01	TP-11
PROJECT LOCATION	CONTRACTOR	EQUIPMENT	TECHNICIAN	DATE
Camas, Washington	L&S Contractors	Excavator	SSC/SMF	8/23/23
TEST PIT LOCATION	GROUNDWATER DEPTH		START TIME	FINISH TIME
See Figure 2	Groundwater not observed.		1518	1538

Depth (feet)	Sample Field ID	SCS Soil Survey Description	AASHTO Soil Type	USCS Soil Type	Graphic Log	LITHOLOGIC DESCRIPTION AND REMARKS	Moisture Content (%)	Passing No. 200 Sieve (%)	Liquid Limit	Plasticity Index	Infiltration Testing
0						Approximately 2 inches of root zone underlain by 4 inches of topsoil.					
				GC		Brown clayey GRAVEL with sand, damp, dense, fines have low to medium plasticity, 1 to 10 inch subrounded gravel.					
5											
10						Excavator comment: digging like dense soil.					
						Bottom of test pit at 13 feet BGS. Groundwater not observed.					
15											

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TEST PIT LOG

PROJECT NAME	CLIENT		PROJECT NO.	TEST PIT NO.
Webberley Development	HSR Development		HSR-3-01-01	TP-12
PROJECT LOCATION: Camas, Washington	CONTRACTOR: L&S Contractors	EQUIPMENT: Excavator	TECHNICIAN: SSC/SMF	DATE: 8/23/23
TEST PIT LOCATION: See Figure 2	GROUNDWATER DEPTH: Groundwater not observed.		START TIME: 1545	FINISH TIME: 1605

Depth (feet)	Sample Field ID	SCS Soil Survey Description	AASHTO Soil Type	USCS Soil Type	Graphic Log	LITHOLOGIC DESCRIPTION AND REMARKS	Moisture Content (%)	Passing No. 200 Sieve (%)	Liquid Limit	Plasticity Index	Infiltration Testing
0						Approximately 2 inches of root zone underlain by 4 inches of topsoil.					
				SC		Brown clayey SAND, medium dense, fines have low to medium plasticity, sparse gravel.					
5				SC		WEATHERED CONGLOMERATE: Red and yellow and orange weathered clayey SAND wth gravel, moist, very dense, fines have low to medium plasticity, 1 to 3 inch gravel.					
						Bottom of test pit at 6 feet BGS due to practical refusal on competent bedrock. Groundwater not observed.					
10											
15											

APPENDIX B
LABORATORY TESTING

CLASSIFICATION

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

ATTERBERG LIMITS

Atterberg limits (plastic and liquid limits) testing was performed on select soil samples in general accordance with ASTM D4318. The plastic limit is defined as the moisture content where the soil becomes brittle. The liquid limit is defined as the moisture content where the soil begins to act similar to a liquid. The plasticity index is the difference between the liquid and plastic limits. The test results are presented in this appendix.

MOISTURE CONTENT

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

PARTICLE-SIZE ANALYSIS

We completed particle-size analysis on select soil samples in general accordance with ASTM D6913. This test is a quantitative determination of the soil particle size distribution expressed as a percentage of dry soil weight.

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MOISTURE CONTENT, PERCENT PASSING NO. 200 SIEVE BY WASHING

PROJECT	CLIENT	PROJECT NO.	REPORT DATE
Webberley Development Camas, Washington	HSR Development 500 E Broadway, Suite 120 Vancouver, Washington 98660	HSR-3-01-1	09/12/23
		DATE SAMPLED: 08/23/23	
		SAMPLED BY: SSC	

LABORATORY TEST DATA

TEST PROCEDURE
ASTM D2216 - Method A, ASTM D1140

LAB ID	CONTAINER MASS	MOIST MASS + PAN	DRY MASS + PAN	AFTER WASH DRY MASS + PAN	MATERIAL DESCRIPTION	FIELD ID	SAMPLE DEPTH	MOISTURE CONTENT	PASSING NO. 200 SIEVE
S23-1126	784.4	2,401.2	2,109.7	sieved sample	light brown Clayey SAND	TP2.1	3 feet	22%	31%
S23-1127	786.9	2,475.4	2,069.9	1,832.4	red/brown Clayey SAND	TP2.2	6 feet	32%	19%
S23-1128	853.1	3,834.0	3,349.7	sieved sample	brown Clayey SAND with Gravel	TP8.2	6 feet	19%	18%
S23-1129	773.0	2,309.2	1,969.4	1,487.0	light brown Silty SAND with Gravel	TP9.1	4 feet	28%	40%
S23-1130	1,744.3	51,010.0	46,200.1	sieved sample	brown Silty GRAVEL with Sand and Cobbles	TP10.1	2 feet	11%	40%
S23-1131	866.5	2,585.1	2,298.6	1,717.8	brown Clayey GRAVEL with Sand	TP10.2	4 feet	20%	41%

NOTES:
Sample weights received for Lab ID: S23-1126, 1128, and 1130 did not meet the minimum size requirements; entire sample used for analysis.

DATE TESTED: 09/05/23
TESTED BY: MRS/KMS

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PARTICLE-SIZE ANALYSIS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Webberley Development Camas, Washington	HSR Development 500 E Broadway, Suite 120 Vancouver, Washington 98660	HSR-3-01-1	S23-1126
		REPORT DATE: 09/12/23	FIELD ID: TP2.1
		DATE SAMPLED: 08/23/23	SAMPLED BY: SSC

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
light brown Clayey SAND	Test Pit TP-02 depth = 3 feet	SC, Clayey Sand

SPECIFICATIONS	AASHTO CLASSIFICATION
none	A-2-7(2)

LABORATORY TEST DATA

LABORATORY EQUIPMENT	TEST PROCEDURE
Rainhart "Mary Ann" Sifter, air-dried prep, hand washed, composite sieve - #4 split	ASTM D6913, Method A

ADDITIONAL DATA

initial dry mass (g) =	1325.3		
as-received moisture content =	22%	coefficient of curvature, C_C =	n/a
liquid limit =	53	coefficient of uniformity, C_U =	n/a
plastic limit =	28	effective size, $D_{(10)}$ =	n/a
plasticity index =	25	$D_{(30)}$ =	n/a
fineness modulus =	n/a	$D_{(60)}$ =	0.412 mm

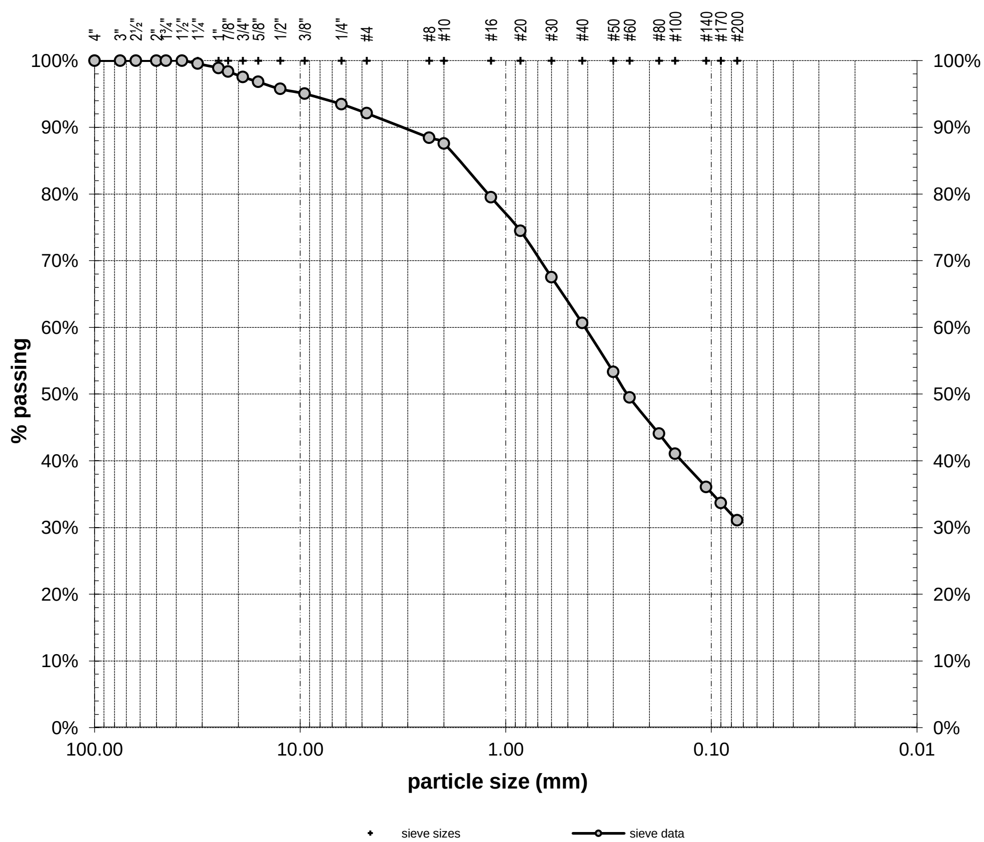
NOTES: Entire sample used for analysis; did not meet minimum size required.

SIEVE DATA

% gravel = 7.9%
% sand = 61.0%
% silt and clay = 31.1%

	SIEVE SIZE		PERCENT PASSING			
			SIEVE		SPECS	
	US	mm	act.	interp.	max	min
GRAVEL	6.00"	150.0		100%		
	4.00"	100.0		100%		
	3.00"	75.0		100%		
	2.50"	63.0		100%		
	2.00"	50.0		100%		
	1.75"	45.0		100%		
	1.50"	37.5	100%			
	1.25"	31.5		100%		
	1.00"	25.0	99%			
	7/8"	22.4		98%		
	3/4"	19.0	98%			
	5/8"	16.0		97%		
	1/2"	12.5	96%			
	3/8"	9.50	95%			
	1/4"	6.30	93%			
	#4	4.75	92%			
SAND	#8	2.36		88%		
	#10	2.00	88%			
	#16	1.18		80%		
	#20	0.850	74%			
	#30	0.600		68%		
	#40	0.425	61%			
	#50	0.300		53%		
	#60	0.250	49%			
	#80	0.180		44%		
	#100	0.150	41%			
	#140	0.106		36%		
	#170	0.090		34%		
	#200	0.075	31%			

GRAIN SIZE DISTRIBUTION



DATE TESTED	TESTED BY
09/07/23	MRS

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ATTERBERG LIMITS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Webberley Development Camas, Washington	HSR Development 500 E Broadway, Suite 120 Vancouver, Washington 98660	HSR-3-01-1	S23-1126
		REPORT DATE: 09/12/23	FIELD ID: TP2.1
		DATE SAMPLED: 08/23/23	SAMPLED BY: SSC

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
light brown Clayey SAND	Test Pit TP-02 depth = 3 feet	SC, Clayey Sand

LABORATORY TEST DATA

LABORATORY EQUIPMENT	TEST PROCEDURE
Liquid Limit Machine, Hand Rolled	ASTM D4318

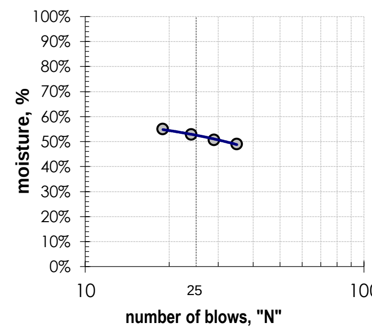
ATTERBERG LIMITS

liquid limit = 53
plastic limit = 28
plasticity index = 25

LIQUID LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	31.13	31.35	32.82	33.02
dry soil + pan weight, g =	27.79	27.86	28.69	28.60
pan weight, g =	20.98	20.97	20.88	20.56
N (blows) =	35	29	24	19
moisture, % =	49.1 %	50.7 %	52.9 %	55.0 %

LIQUID LIMIT



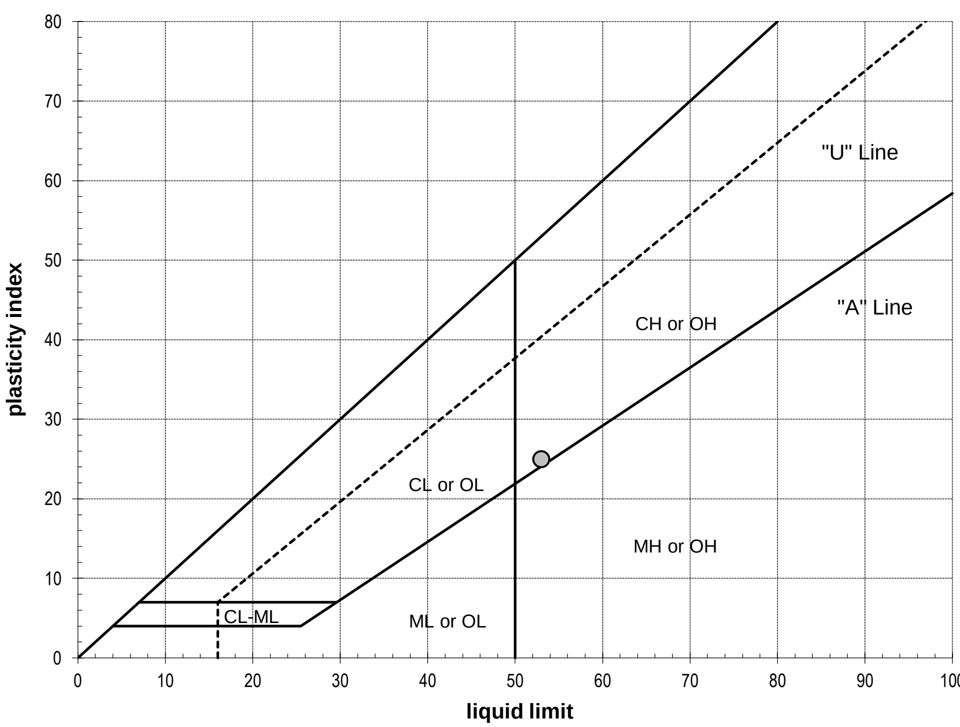
SHRINKAGE

shrinkage limit = n/a
shrinkage ratio = n/a

PLASTIC LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	27.68	27.07		
dry soil + pan weight, g =	26.20	25.75		
pan weight, g =	20.87	21.02		
moisture, % =	27.8 %	27.9 %		

PLASTICITY CHART



ADDITIONAL DATA

% gravel = 7.9%
% sand = 61.0%
% silt and clay = 31.1%
% silt = n/a
% clay = n/a
moisture content = 22%

DATE TESTED	TESTED BY
09/11/23	KMS

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PARTICLE-SIZE ANALYSIS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Webberley Development Camas, Washington	HSR Development 500 E Broadway, Suite 120 Vancouver, Washington 98660	HSR-3-01-1	S23-1128
		REPORT DATE: 09/12/23	FIELD ID: TP8.2
		DATE SAMPLED: 08/23/23	SAMPLED BY: SSC

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
brown Clayey SAND with Gravel	Test Pit TP-08 depth = 6 feet	SC, Clayey Sand with Gravel

SPECIFICATIONS	AASHTO CLASSIFICATION
none	A-2-7(0)

LABORATORY TEST DATA

LABORATORY EQUIPMENT	TEST PROCEDURE
Rainhart "Mary Ann" Sifter, air-dried prep, hand washed, composite sieve - #4 split	ASTM D6913, Method A

ADDITIONAL DATA

initial dry mass (g) =	2496.6		
as-received moisture content =	19%	coefficient of curvature, C_C =	n/a
liquid limit =	50	coefficient of uniformity, C_U =	n/a
plastic limit =	28	effective size, $D_{(10)}$ =	n/a
plasticity index =	22	$D_{(30)}$ =	0.210 mm
fineness modulus =	n/a	$D_{(60)}$ =	2.178 mm

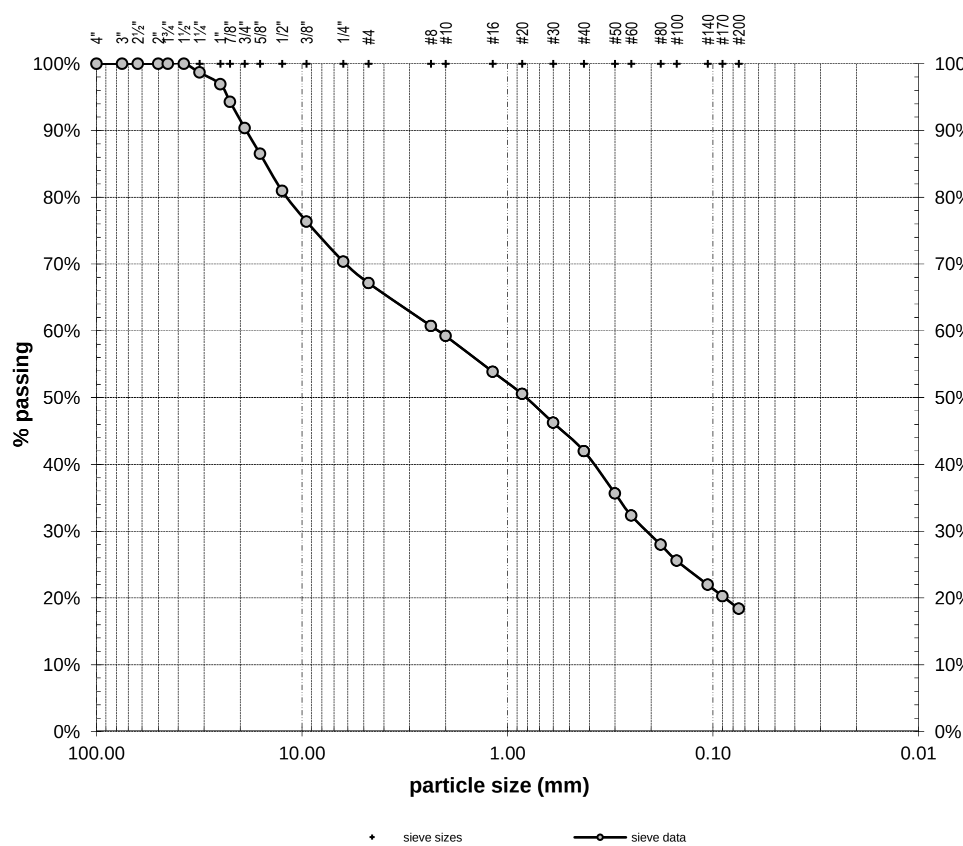
NOTES: Entire sample used for analysis; did not meet minimum size required.

SIEVE DATA

% gravel = 32.8%
% sand = 48.8%
% silt and clay = 18.4%

	SIEVE SIZE US	mm	PERCENT PASSING SIEVE act.	interp.	SPECS max	min
GRAVEL	6.00"	150.0		100%		
	4.00"	100.0		100%		
	3.00"	75.0		100%		
	2.50"	63.0		100%		
	2.00"	50.0		100%		
	1.75"	45.0		100%		
	1.50"	37.5	100%			
	1.25"	31.5		99%		
	1.00"	25.0	97%			
	7/8"	22.4		94%		
	3/4"	19.0	90%			
	5/8"	16.0		86%		
	1/2"	12.5	81%			
	3/8"	9.50	76%			
	1/4"	6.30	70%			
	#4	4.75	67%			
SAND	#8	2.36		61%		
	#10	2.00	59%			
	#16	1.18		54%		
	#20	0.850	51%			
	#30	0.600		46%		
	#40	0.425	42%			
	#50	0.300		36%		
	#60	0.250	32%			
	#80	0.180		28%		
	#100	0.150	26%			
	#140	0.106		22%		
	#170	0.090		20%		
	#200	0.075	18%			

GRAIN SIZE DISTRIBUTION



DATE TESTED	TESTED BY
09/07/23	MRS

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ATTERBERG LIMITS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Webberley Development Camas, Washington	HSR Development 500 E Broadway, Suite 120 Vancouver, Washington 98660	HSR-3-01-1	S23-1128
		REPORT DATE: 09/12/23	FIELD ID: TP8.2
		DATE SAMPLED: 08/23/23	SAMPLED BY: SSC

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
brown Clayey SAND with Gravel	Test Pit TP-08 depth = 6 feet	SC, Clayey Sand with Gravel

LABORATORY TEST DATA

LABORATORY EQUIPMENT	TEST PROCEDURE
Liquid Limit Machine, Hand Rolled	ASTM D4318

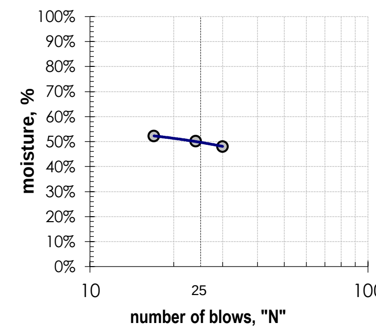
ATTERBERG LIMITS

liquid limit = 50
plastic limit = 28
plasticity index = 22

LIQUID LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	33.98	31.68	32.77	
dry soil + pan weight, g =	29.59	27.92	28.72	
pan weight, g =	20.44	20.43	20.96	
N (blows) =	30	24	17	
moisture, % =	48.0 %	50.2 %	52.2 %	

LIQUID LIMIT



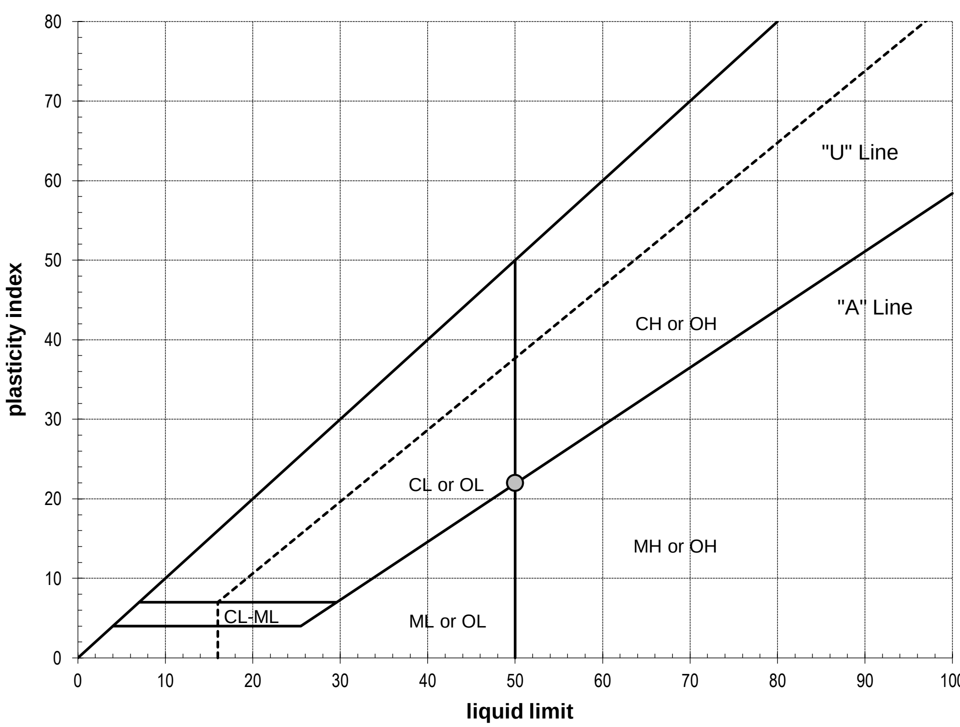
SHRINKAGE

shrinkage limit = n/a
shrinkage ratio = n/a

PLASTIC LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	27.61	27.64		
dry soil + pan weight, g =	26.15	26.03		
pan weight, g =	20.93	20.42		
moisture, % =	28.0 %	28.7 %		

PLASTICITY CHART



ADDITIONAL DATA

% gravel = 32.8%
% sand = 48.8%
% silt and clay = 18.4%
% silt = n/a
% clay = n/a
moisture content = 19%

DATE TESTED	TESTED BY
09/06/23	KMS

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PARTICLE-SIZE ANALYSIS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Webberley Development Camas, Washington	HSR Development 500 E Broadway, Suite 120 Vancouver, Washington 98660	HSR-3-01-1	S23-1130
		REPORT DATE: 09/12/23	FIELD ID: TP10.1
		DATE SAMPLED: 08/23/23	SAMPLED BY: SSC

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
brown Silty GRAVEL with Sand and Cobbles	Test Pit TP-10 depth = 2 feet	GM, Silty Gravel with Sand and cobbles

SPECIFICATIONS	AASHTO CLASSIFICATION
none	A-7-6(3)

LABORATORY TEST DATA

LABORATORY EQUIPMENT	TEST PROCEDURE
Rainhart "Mary Ann" Sifter, air-dried prep, hand washed, composite sieve - #4 split	ASTM D6913, Method A

ADDITIONAL DATA

initial dry mass (g) =	44455.8		
as-received moisture content =	11%	coefficient of curvature, C_C =	n/a
liquid limit =	47	coefficient of uniformity, C_U =	n/a
plastic limit =	28	effective size, $D_{(10)}$ =	n/a
plasticity index =	19	$D_{(30)}$ =	n/a
fineness modulus =	n/a	$D_{(60)}$ =	4.135 mm

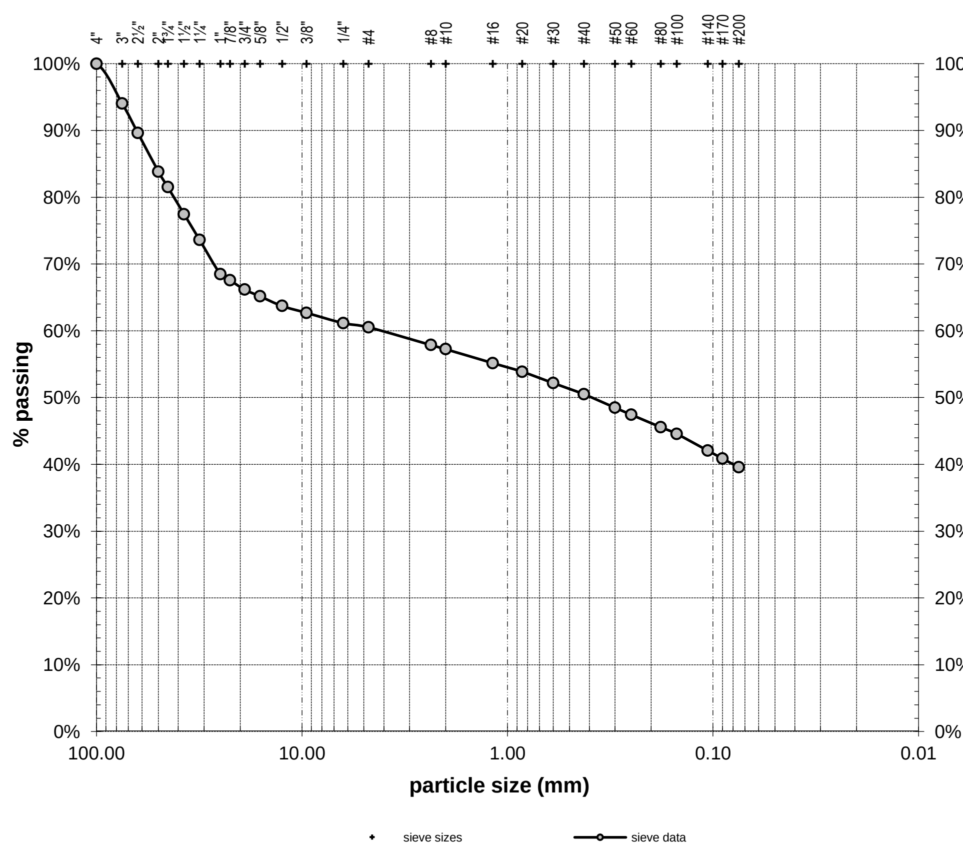
NOTES: Entire sample used for analysis; did not meet minimum size required.

SIEVE DATA

% gravel = 39.5%
% sand = 21.0%
% silt and clay = 39.6%

	SIEVE SIZE US	mm	PERCENT PASSING SIEVE act.	interp.	SPECS max	min
GRAVEL	6.00"	150.0		100%		
	4.00"	100.0	100%			
	3.00"	75.0	94%			
	2.50"	63.0		90%		
	2.00"	50.0	84%			
	1.75"	45.0		81%		
	1.50"	37.5		77%		
	1.25"	31.5		74%		
	1.00"	25.0	68%			
	7/8"	22.4		68%		
	3/4"	19.0	66%			
	5/8"	16.0		65%		
	1/2"	12.5	64%			
	3/8"	9.50		63%		
	1/4"	6.30	61%			
	#4	4.75	61%			
SAND	#8	2.36		58%		
	#10	2.00	57%			
	#16	1.18		55%		
	#20	0.850	54%			
	#30	0.600		52%		
	#40	0.425	50%			
	#50	0.300		48%		
	#60	0.250	47%			
	#80	0.180		46%		
	#100	0.150	45%			
	#140	0.106		42%		
	#170	0.090		41%		
	#200	0.075	40%			

GRAIN SIZE DISTRIBUTION



DATE TESTED	TESTED BY
09/08/23	MRS

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ATTERBERG LIMITS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Webberley Development Camas, Washington	HSR Development 500 E Broadway, Suite 120 Vancouver, Washington 98660	HSR-3-01-1	S23-1130
		REPORT DATE: 09/12/23	FIELD ID: TP10.1
		DATE SAMPLED: 08/23/23	SAMPLED BY: SSC

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
brown Silty GRAVEL with Sand and Cobbles	Test Pit TP-10 depth = 2 feet	GM, Silty Gravel with Sand and cobbles

LABORATORY TEST DATA

LABORATORY EQUIPMENT	TEST PROCEDURE
Liquid Limit Machine, Hand Rolled	ASTM D4318

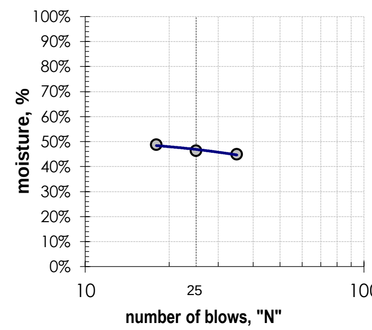
ATTERBERG LIMITS

liquid limit = 47
plastic limit = 28
plasticity index = 19

LIQUID LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	31.40	32.97	33.85	
dry soil + pan weight, g =	28.04	29.08	29.53	
pan weight, g =	20.56	20.67	20.67	
N (blows) =	35	25	18	
moisture, % =	44.9 %	46.3 %	48.8 %	

LIQUID LIMIT



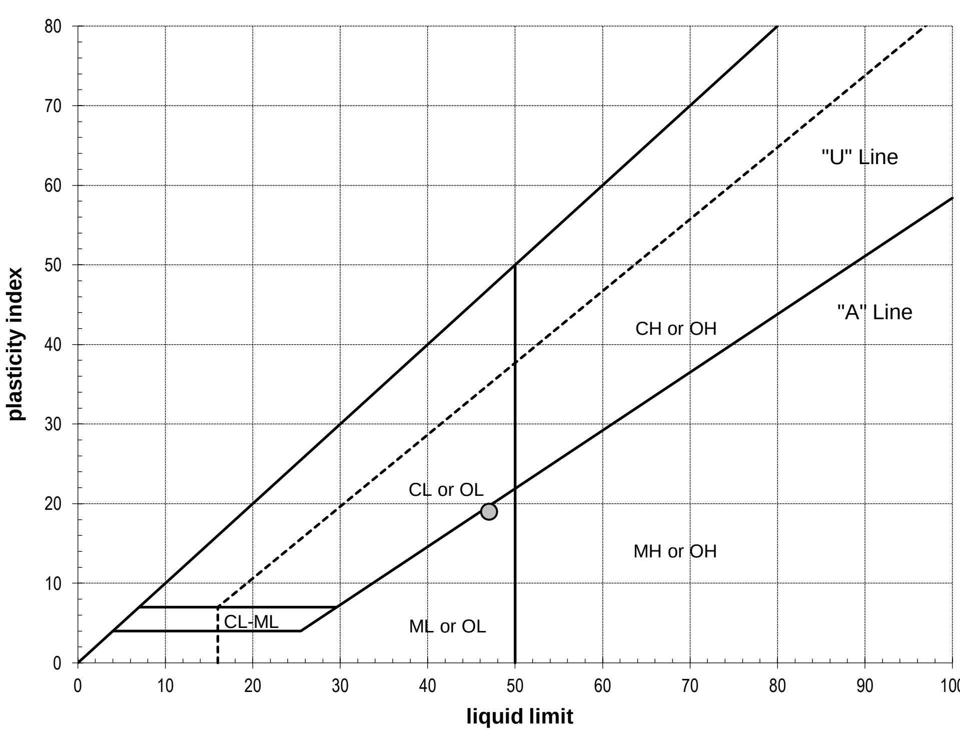
SHRINKAGE

shrinkage limit = n/a
shrinkage ratio = n/a

PLASTIC LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	27.76	28.65		
dry soil + pan weight, g =	26.23	26.96		
pan weight, g =	20.77	20.94		
moisture, % =	28.0 %	28.1 %		

PLASTICITY CHART



ADDITIONAL DATA

% gravel = 39.5%
% sand = 21.0%
% silt and clay = 39.6%
% silt = n/a
% clay = n/a
moisture content = 11%

DATE TESTED	TESTED BY
09/06/23	KMS

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APPENDIX C
REPORT LIMITATIONS AND IMPORTANT INFORMATION

Geotechnical and Environmental Report Limitations and Important Information

Report Purpose, Use, and Standard of Care

This report has been prepared in accordance with standard fundamental principles and practices of geotechnical engineering and/or environmental consulting, and in a manner consistent with the level of care and skill typical of currently practicing local engineers and consultants. This report has been prepared to meet the specific needs of specific individuals for the indicated site. It may not be adequate for use by other consultants, contractors, or engineers, or if change in project ownership has occurred. It should not be used for any other reason than its stated purpose without prior consultation with Columbia West Engineering, Inc. (Columbia West). It is a unique report and not applicable for any other site or project. If site conditions are altered, or if modifications to the project description or proposed plans are made after the date of this report, it may not be valid. Columbia West cannot accept responsibility for use of this report by other individuals for unauthorized purposes, or if problems occur resulting from changes in site conditions for which Columbia West was not aware or informed.

Report Conclusions and Preliminary Nature

This geotechnical or environmental report should be considered preliminary and summary in nature. The recommendations contained herein have been established by engineering interpretations of subsurface soils based upon conditions observed during site exploration. The exploration and associated laboratory analysis of collected representative samples identifies soil conditions at specific discreet locations. It is assumed that these conditions are indicative of actual conditions throughout the subject property. However, soil conditions may differ between tested locations at different seasonal times of the year, either by natural causes or human activity. Distinction between soil types may be more abrupt or gradual than indicated on the soil logs. This report is not intended to stand alone without understanding of concomitant instructions, correspondence, communication, or potential supplemental reports that may have been provided to the client.

Because this report is based upon observations obtained at the time of exploration, its adequacy may be compromised with time. This is particularly relevant in the case of natural disasters, earthquakes, floods, or other significant events. Report conclusions or interpretations may also be subject to revision if significant development or other manmade impacts occur within or in proximity to the subject property. Groundwater conditions, if presented in this report, reflect observed conditions at the time of investigation. These conditions may change annually, seasonally or as a result of adjacent development.

Additional Investigation and Construction QA/QC

Columbia West should be consulted prior to construction to assess whether additional investigation above and beyond that presented in this report is necessary. Even slight variations in soil or site conditions may produce impacts to the performance of structural facilities if not adequately addressed. This underscores the importance of diligent QA/QC construction observation and testing to verify soil conditions do not differ materially or significantly from the interpreted conditions utilized for preparation of this report.

Therefore, this report contains several recommendations for field observation and testing by Columbia West personnel during construction activities. Actual subsurface conditions are more readily observed and discerned during the earthwork phase of construction when soils are exposed. Columbia West cannot accept responsibility for deviations from recommendations described in this report or future performance of structural facilities if another consultant is retained during the construction phase or Columbia West is not engaged to provide construction observation to the full extent recommended.

Collected Samples

Uncontaminated samples of soil or rock collected in connection with this report will be retained for thirty days. Retention of such samples beyond thirty days will occur only at client's request and in return for payment of storage charges incurred. All contaminated or environmentally impacted materials or samples are the sole property of the client. Client maintains responsibility for proper disposal.

Report Contents

This geotechnical or environmental report should not be copied or duplicated unless in full, and even then only under prior written consent by Columbia West, as indicated in further detail in the following text section entitled *Report Ownership*. The recommendations, interpretations, and suggestions presented in this report are only understandable in context of reference to the whole report. Under no circumstances should the soil boring or test pit excavation logs, monitor well logs, or laboratory analytical reports be separated from the remainder of the report. The logs or reports should not be redrawn or summarized by other entities for inclusion in architectural or civil drawings, or other relevant applications.

Report Limitations for Contractors

Geotechnical or environmental reports, unless otherwise specifically noted, are not prepared for the purpose of developing cost estimates or bids by contractors. The extent of exploration or investigation conducted as part of this report is usually less than that necessary for contractor's needs. Contractors should be advised of these report limitations, particularly as they relate to development of cost estimates. Contractors may gain valuable information from this report but should rely upon their own interpretations as to how subsurface conditions may affect cost, feasibility, accessibility and other components of the project work. If believed necessary or relevant, contractors should conduct additional exploratory investigation to obtain satisfactory data for the purposes of developing adequate cost estimates. Clients or developers cannot insulate themselves from attendant liability by disclaiming accuracy for subsurface ground conditions without advising contractors appropriately and providing the best information possible to limit potential for cost overruns, construction problems, or misunderstandings.

Report Ownership

Columbia West retains the ownership and copyright property rights to this entire report and its contents, which may include, but may not be limited to, figures, text, logs, electronic media, drawings, laboratory reports, and appendices. This report was prepared solely for the client, and other relevant approved users or parties, and its distribution must be contingent upon prior express written consent by Columbia West. Furthermore, client or approved users may not use, lend, sell, copy, or distribute this document without express written consent by Columbia West. Client does not own nor have rights to electronic media files that constitute this report, and under no circumstances should said electronic files be distributed or copied. Electronic media is susceptible to unauthorized manipulation or modification and may not be reliable.

Consultant Responsibility

Geotechnical and environmental engineering and consulting is much less exact than other scientific or engineering disciplines, and relies heavily upon experience, judgment, interpretation, and opinion often based upon media (soils) that are variable, anisotropic, and non-homogenous. This often results in unrealistic expectations, unwarranted claims, and uninformed disputes against a geotechnical or environmental consultant. To reduce potential for these problems and assist relevant parties in better understanding of risk, liability, and responsibility, geotechnical and environmental reports often provide definitive statements or clauses defining and outlining consultant responsibility. The client is encouraged to read these statements carefully and request additional information from Columbia West if necessary.