EMERIO
ENGINEERING ▪ SURVEYING ▪ DESIGN

Preliminary Stormwater Management Plan
For Wilsonville TOD
Mixed-Use Apartments
Wilsonville, Oregon
(TL 703, Tax Map 31W14B)

Emerio Project Number:	0951-003
City of Wilsonville Permit Number:	TBD
Date:	11/29/2023

REGISTERED PROFESSIONAL ENGINEER
82915PE
OREGON
MAY 12 2009
STEVEN WELLS HANSEN
EXPIRES: 6/30/2025

Prepared For:
Westin Glass
Palindrome
412 NW 5th Ave., Suite 200
Portland, OR 97209
(503)-719-2742
wglass@palindromecreates.com

Prepared By:
Steve Hansen, PE
Emerio Design, LLC
6445 SW Fallbrook PL, Suite 100
Beaverton, Oregon 97008
(503) 746-8812
steveh@emeriodesign.com

City of Wilsonville
Exhibit B6 DB23-0011

List of Appendices:

Project Overview and Description:

Size and location of project site: The current site is located approximately 320 feet west of the intersection of SW Kinsman Road and SW Barber Street. The site will be developed into mixed use apartments, which will include onsite parking areas, walkways, and landscape areas, along with frontage improvements along SW Barber Street. The site address is 9749 SW Barber Street, see Appendix A(1) for a map of the site location.

Type of Development: The proposed development is primarily composed of multi-family residential apartments with the previously described amenities. The proposed onsite buildings will also include a coffee shop/tap house. Frontage improvements will include new sidewalk and landscaped areas.

Existing vs. post-construction conditions: Currently the site is made up of a primarily grassed and undeveloped lot with some trees and shrubs along the southern border of the site. In the post-developed condition, the previously described proposed apartments and associated walkways and parking areas will make up most of the site. Some of the existing trees will be preserved, while others will be removed.

Watershed Description: The site currently sheet flows toward the southwest corner of the site. In the post-developed condition, onsite impervious areas will be collected by onsite planter facilities, then routed to the existing storm system along SW Barber Road via a piped storm system. Onsite pervious areas will also route to this public storm system via overland flow or via the onsite piped storm system. Frontage improvement areas will have a similar pattern to the onsite areas, with runoff being collected by proposed planters then piped to the existing storm system along SW Barber Road.

Soil Classification:

The NRCS soil survey of Clackamas County, Oregon classifies the onsite soils as Salem Silt Loam, Willamette Silt Loam, and Woodburn Silt Loam. The associated hydrologic groups for these soils are B, B, and C respectively. Due to low onsite infiltration rates, HSG C will be used for the purposes of sizing all onsite and frontage planters. For the future analysis of the proposed storm pipe network, a curve number of 86 will be used for pervious surfaces, and a curve number of 98 will be used for impervious surfaces. See Appendix A(2) for a soil classification map and A(3) for a curve number table.

Infiltration Testing:

Onsite infiltration testing was conducted by Columbia West Engineering, Inc. on April 28 and December 7, 2023. The testing revealed that there was negligible infiltration on the site and that designing infiltration facilities is unfeasible. See Appendix A(4) for the full geotechnical report.

Onsite Treatment Methodology:

Onsite stormwater runoff from impervious areas will be addressed by filtration planters and porous pavement. All proposed roof and parking areas will route runoff to onsite filtration planters, along with some walkway areas. All walkway areas that do not route runoff to planters will be constructed of porous pavement.

The City of Wilsonville approves the use of the WES BMP Sizing Tool to size the filtration planter facilities. Per conversations with City of Wilsonville and the absence of measurable onsite infiltration rates, all planters will be sized using Hydrologic Soil Group (HSG) C. Planters in areas designated as HSG B (see Appendix A(2)) that meet infiltration setback requirements will be designed with open bottoms to allow for any extraneous infiltration. Planters facilities that meet these criteria are Planters 8, 9, 10, and 11.

Some onsite planter volumes are hydraulically connected via pipes and will operate as a single facility. The planters that will operate under this design are Planters 1 & 2 to the northwest of the proposed building and Planters 3, 4, & 5 to the west of the proposed building, see Appendix C(2).

See the following table for the basin areas routing to each facility and both the required and provided planter sizes.

Basin ID	Facility ID	Total Basin Area (SF)	Facility Area Required (SF)	Facility Area Provided (SF)	Orifice Size (in)
A	Planters 1 & 2	19,855	1,389.9	1,595.0	1.3
B	Planters 3, 4, & 5	3,766	263.6	333.0	0.6
C	Planter 6	4,697	328.8	357.0	0.7
D	Planter 7	1,124	78.7	295.0	0.3
E	Planter 8	3,668	256.8	259.0	0.6
F	Planter 9	1,305	91.4	96.0	0.3
G	Planter 10	1,096	76.7	96.0	0.3
H	Planter 11	1,928	135.0	146.0	0.4

As shown in the table above, all proposed facilities were appropriately sized to meet water quality and detention standards. See Appendix C(2) for the basin delineation map and Appendix B(1) for the BMP sizing report.

Offsite Treatment Methodology:

All new impervious areas along SW Barber Road will route to proposed filtration planters along SW Barber Road. The proposed planter facilities replace an existing treatment facility and will provide treatment to all previously treated existing impervious area along SW Barber Road.

A large portion of the existing impervious area that requires treatment is upstream of the proposed planter locations. The furthest upstream planter (ROW Planter E) will be constructed with adequate capacity to treat this existing upstream impervious area by implementing check dams into the planter design.

ROW Planters C, & D contain street trees. A 6 ft x 8 ft area around these trees will not be considered as treatment area.

See the following table for the basin areas going to each facility and both the required and provided planter sizes.

Basin ID	Facility ID	Total Basin Area (SF)	Facility Area Required (SF)	Facility Area Provided (SF)	Orifice Size (in)
I	ROW Planter A	694	48.6	96.0	0.2
J	ROW Planter B	1,366	95.6	96.0	0.4
K	ROW Planter C	2,457	172.0	174.0	0.5
L	ROW Planter D	3,272	229.0	229.0	0.5
M	ROW Planter E	7,659	536.1	539.0	0.8

As shown in the table above, all proposed facilities were appropriately sized to meet water quality and detention standards. See Appendix C(2) for the basin delineation map and Appendix B(1) for the BMP sizing report.

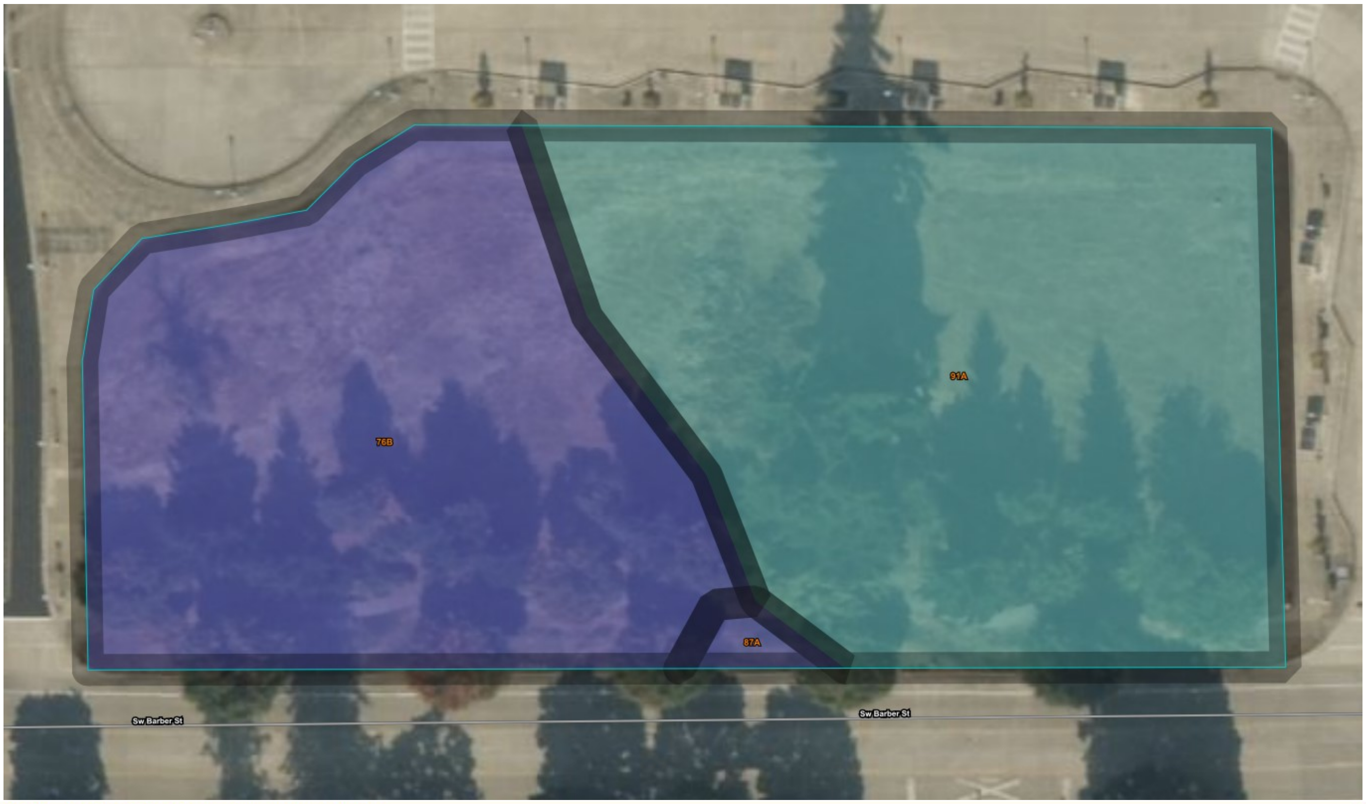
Conclusion:

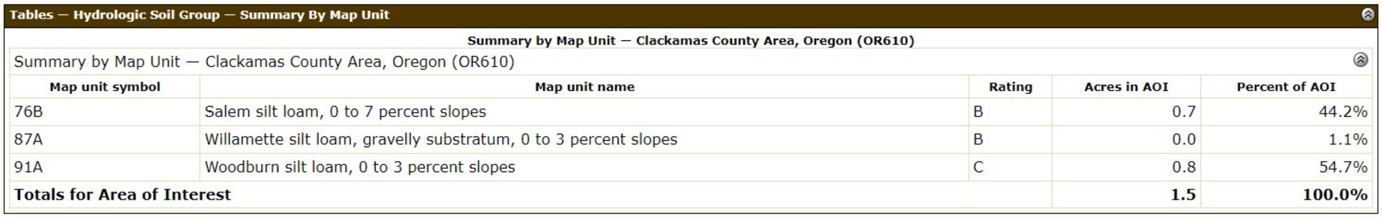
The design of the proposed site satisfies the stormwater design standards set by the City of Wilsonville.

Appendix A

Appendix A(1)
Vicinity Map

Appendix A(2)
Soil Classification Map





Tables — Hydrologic Soil Group — Summary By Map Unit

Summary by Map Unit — Clackamas County Area, Oregon (OR610)

Summary by Map Unit — Clackamas County Area, Oregon (OR610)

Map unit symbol	Map unit name	Rating	Acres in AOI	Percent of AOI
76B	Salem silt loam, 0 to 7 percent slopes	B	0.7	44.2%
87A	Willamette silt loam, gravelly substratum, 0 to 3 percent slopes	B	0.0	1.1%
91A	Woodburn silt loam, 0 to 3 percent slopes	C	0.8	54.7%
Totals for Area of Interest			**1.5**	**100.0%**

RUNOFF CURVE NUMBERS (TR55)

Table 2-2a: Runoff curve numbers for urban areas [1]

Cover description		CN for hydrologic soil group			
Cover type and hydrologic condition	Average percent impervious area[2]	A	B	C	D
Fully developed urban areas (vegetation established)					
Open space (lawns, parks, golf courses, cemeteries, etc.) [3]:					
Poor condition (grass cover <50%)		68	79	86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover >75%)		39	61	74	80
Impervious areas:					
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
Streets and roads:					
Paved; curbs and storm sewers (excluding right-of-way)				98	98
Paved; open ditches (including right-of-way)				92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Western desert urban areas:					
Natural desert landscaping (pervious areas only) [4]		63	77	85	88
Artificial desert landscaping (impervious weed barrier, desert shrub with 1- to 2-inch sand or gravel mulch and basin borders)		96	96	96	96
Urban districts:					
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential districts by average lot size:					
1/8 acre or less (town houses)	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
2 acres	12	46	65	77	82

Use CN = 86 for Onsite and Offsite Pervious Areas

Use CN = 98 for Impervious Areas

Geotechnical Site Investigation

Barber Street Housing Development

Wilsonville, Oregon

May 18, 2023

Geotechnical ■ Environmental ■ Special Inspections

Columbia West Engineering, Inc

11917 NE 95th Street
Vancouver, Washington
98682
Phone: 360-823-2900

www.columbiawestengineering.com

Geotechnical ▪ Environmental ▪ Special Inspections
Columbia West
Engineering, Inc

GEOTECHNICAL SITE INVESTIGATION
BARBER STREET HOUSING DEVELOPMENT
WILSONVILLE, OREGON

Prepared For:

Palindrome Wilsonville Limited Partnership
Attn: Jason Ellis
412 NW 5th Avenue
Portland, Oregon 97201

Site Location:

9699 SW Barber Street
Wilsonville, Oregon 97070

Prepared By:

Columbia West Engineering, Inc.
11917 NE 95th Street
Vancouver, Washington 98682
Phone: 360-823-2900
Work Order Number 23122

Date Prepared:

May 18, 2023

EXECUTIVE SUMMARY

This executive summary presents the primary geotechnical considerations associated with the proposed Barber Street Housing Development project located in Wilsonville, Oregon. 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, site soils are not susceptible to liquefaction under design levels of ground shaking.
- Foundations designed in accordance with this report should be sized based on an allowable soil bearing capacity of 2,500 psf and are expected to experience a post construction settlement of less than one inch. Differential post construction settlement between comparably-loaded footing elements is not expected to exceed 0.5 inches.
- Undocumented fill was encountered in two borings located on the northwest portion of the site to depths between approximately 3 and 6.5 feet below ground surface (BGS). Though not observed within the proposed building footprint, undocumented fill and should be completely removed if encountered under footings. There is also a risk of premature pavement distress if existing fill is left in place beneath future pavements. Additional discussions and our recommendations are provided in the report.
- Groundwater was not observed within the borings to the maximum explored depth of 31.5 feet BGS, however the driller indicated heaving soils at approximately 15 feet BGS in boring B-1. Review of information in our files and nearby well logs presented in Appendix B indicates that groundwater could range from 10 to 20 feet BGS in the vicinity of the site.
- Moisture conditioning (drying) of existing fill and native soil may be required to use the material as structural fill. Addition of moisture may also be necessary during periods of warm, dry weather. If moisture conditioning is not feasible, soils may require cement-amendment to be used as structural fill.
- Fine-grained soils will be sensitive to disturbance and softening when at a moisture content that is above optimum. Haul roads and staging areas will be necessary to minimize damage to exposed subgrade soils during construction. Subgrade protection is discussed in Section 8.2, *Construction Traffic and Staging*.
- Based on fine-textured materials and results of in situ infiltration testing, infiltration is likely not feasible for stormwater management.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF APPENDICES

GEOTECHNICAL SITE INVESTIGATION
BARBER STREET HOUSING DEVELOPMENT
WILSONVILLE, OREGON

1.0 INTRODUCTION

Columbia West Engineering, Inc. (Columbia West) was retained by Palindrome Wilsonville Limited Partnership to conduct a geotechnical site investigation for use in design and construction of the proposed Barber Street Housing Development located in Wilsonville, Oregon. This report is subject to the limitations expressed in Section 9.0, *Conclusion and Limitations*, and Appendix E.

1.1 General Site Information

As indicated on Figures 1 and 2, the subject site is located west of Interstate 5 and northeast of the intersection at SW Barber Street and SW Kinsman in Wilsonville, Oregon. The site is comprised of portions of tax lots 31W14B00702 and 31W14B00703 totaling approximately 2.28 acres. The approximate latitude and longitude are N 45° 18’ 40” and W 122° 46’ 36”. The regulatory jurisdictional agency is the City of Wilsonville.

1.2 Project Understanding

Based on client correspondence and review of the preliminary site plan shown on Figure 2A, proposed development includes construction of an approximately 114,000 square-foot, 5-story residential structure. The construction type has yet to be determined, however it is anticipated to either consist of 5 floors of conventional wood framing or 4 floors of conventional wood framing over 1 concrete podium. The foundation system is expected to be shallow spread footings.

Foundation loads were not available at the time of this report. We have assumed maximum column and wall loads for the building will be less than 250 kips and 4 kips per foot, respectively. Maximum floor slab loading is expected to be 100 psf. Proposed development also includes associated asphalt parking areas and drive aisles, subsurface utilities, stormwater management facilities, and landscaping. We have also assumed that cuts and fills will be no greater than 3 feet each.

2.0 SCOPE OF SERVICES

Columbia West’s scope of services was outlined in a proposal dated April 4, 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 in the vicinity of the site.
- Reviewed preliminary plans provided by the design team.
- Conducted subsurface exploration program at the site that included:
  - One boring drilled to depth of 30 feet BGS within the proposed building footprint
  - Three borings drilled to depths of 6.5 feet BGS within proposed future parking areas
  - Infiltration testing was conducted in two borings
- Collected disturbed soil samples from the borings for laboratory analysis.

- Classified and logged observed soil 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
    - Infiltration test results
    - Liquefaction analysis and predicted seismic settlement
    - Fill- and load-induced settlement potential
    - Geotechnical design and construction recommendations for:
        - Shallow foundations
        - Slab subgrade preparation
        - Retaining walls, including drainage, backfill, and 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, including section thicknesses for base aggregate and asphalt layers
        - Seismic design parameters in accordance with the 2022 *State of Oregon Specialty Code*

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 north-central portion of the Portland/Vancouver Basin, an open, somewhat elliptical, northwest-trending syncline approximately 60 miles wide.

According to the *Geology and Geologic Hazards of Northwest Clackamas County* (Schlicker and Finlayson, ODGMI, 1979), near-surface soils are expected to consist of Pleistocene-aged, unconsolidated, cross-bedded to graded sedimentary beds of fine sandy silt and clay deposited by glacial floods (Qws) up to 100 feet thick.

The *Web Soil Survey* (USDA, NRCS, 2023 Website) identifies surface soils as Aloha, Salem, and Woodburn silt loam. Aloha, Salem, and Woodburn silt loam series soils are generally fine-textured clays and silts with very low permeability, moderate to high water capacity, and low shear strength. Aloha, Salem, and Woodburn soils are generally moisture sensitive, somewhat compressible, and described as having moderate shrink-swell potential. The erosion hazard is slight primarily based upon slope grade.

4.0 REGIONAL SEISMOLOGY

4.1 Regional Seismic Sources

The CSZ is the region where the Juan de Fuca Plate is being subducted beneath the North American Plate. This subduction is occurring in the coastal region between Vancouver Island and northern California. Evidence has accumulated suggesting 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). The fault trace is mapped approximately 50 to 120 km off the Oregon and Washington Coast. Two types of subduction zone earthquakes are possible:

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 reportedly capable of generating earthquakes with a moment magnitude of between 8.5 and 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 7.5.

4.2 Local Seismic Sources

A significant earthquake could occur on a local fault near the site within the design life of the building. Such an event would cause ground shaking at the site that could be more intense than the CSZ events, although the duration would be shorter. The three closest mapped to the site are: Canby-Mollala Fault, Damascus-Tickle Creek Fault Zone, Beaverton Fault Zone.

Canby-Molalla Fault

The mapped trace of the north-northwest-striking Canby-Molalla fault is based on a linear series of northeast-trending discontinuous aeromagnetic anomalies that probably represent significant offset of Eocene basement and volcanic rocks of the Miocene Columbia River Basalt beneath Neogene sediments that fill the northern Willamette River basin. The fault has little geomorphic expression across the gently sloping floor of the Willamette Valley, but a small, laterally restricted berm associated with the fault may suggest young deformation. Deformation of probable Missoula flood deposits in a high-resolution seismic reflection survey conducted across the aeromagnetic anomaly east of Canby suggests possible Holocene deformation. Sense of displacement of the Canby-Molalla fault is poorly known, but the fault shows apparent right-lateral separation of several transverse magnetic anomalies, and down-west vertical displacement is also apparent in water well logs.

Damascus-Tickle Creek Fault Zone

The Damascus-Tickle Creek fault zone consists of numerous short northeast- and northwest-trending faults that form a broad, northeast-trending fault zone; these faults fold and offset rocks of the Pliocene Troutdale Formation, Plio-Pleistocene Springwater Formation, and Pleistocene Boring Lava. The area is on the southern margin of the Portland basin, and is the location of numerous eruptive vents of the Boring Lava, some of which may have been localized along faults in the zone. Most faults in the zone are buried by latest Pleistocene Missoula flood deposits, but at least one fault strand may have deformed these deposits. Most of these faults are thought to be near-vertical reverse faults with a significant component of right-lateral strike-slip.

Beaverton Fault Zone

The east-west-striking Beaverton fault zone forms the southern margin of the main part of the Tualatin basin, an isolated extension of the Willamette lowland forearc basin in northwestern Oregon. The Beaverton fault zone is not shown on most published geologic maps of the area, but is marked by a linear aeromagnetic anomaly and has been mapped in the subsurface where it offsets Miocene Columbia River Basalt Group rocks and overlying Pliocene to Pleistocene sediments. The late Neogene Tualatin basin may be a pull-apart basin, with subsidence driven by dextral shear on the nearby Gales Creek fault zone. The fault trace is buried by a thick sequence of sediment deposited by the 12.7–13.3 ka Missoula floods, but offsets middle Pleistocene and possibly younger sediments in the subsurface.

5.0 GEOTECHNICAL AND GEOLOGIC FIELD INVESTIGATION

A geotechnical field investigation consisting of visual reconnaissance, four drilled borings (B-1 through B-4), and two infiltration tests was conducted at the site on April 28, 2023.

Samples were collected from the borings using 1½-inch diameter split-barrel (SPT) samples in general accordance with ASTM D1586. The samplers were driven into the soil with a 140-poind hammer free falling 30 inches. The sampler was driven a total distance of 18 inches. The number of blows required to drive the sampler the final 12 inches is recorded on the exploration log, unless otherwise noted. The hammer was lifted using an automatic hammer with a reported efficiency of 77.7 percent. Sampling methods and intervals are shown on the exploration logs. Subsurface soil profiles were logged in accordance with Unified Soil Classification System (USCS) specifications. Disturbed soil samples were collected at representative depth intervals.

Analytical laboratory test results are presented in Appendix A. Exploration locations are shown on Figure 2. Boring logs are presented in Appendix B. Soil descriptions and classification information are provided in Appendix C. A photo log is presented in Appendix D.

5.1 Surface Investigation and Site Description

As indicated on Figures 2 and 2A, the subject site consists of portions of tax lots 31W14B00703 and 31W14B00702. It is bound to the south by SW Barber Street, to the west by an open grassy field, to the east by Wilsonville WES station and associated train tracks, and to the north by Oldcastle buildings and associated infrastructure. The northern half of the development area is generally characterized by asphalt parking areas and drive aisles and sparse landscape tree coverage associated with the existing park-in-ride. The asphalt parking area appears to be raised between 2 to 5 feet compared to surrounding terrain.

The southern half of the site adjacent to SW Barber Street (future building location) consists primarily of open grassy areas with isolated areas of manicured landscape to the south. There is an existing stockpile of organic material in the center of the southern portion of the site as depicted on Figure 2A. Field reconnaissance and review of site topographic mapping indicates that that the site is relatively flat and characterized by grades of 0 to 5 percent.

5.2 Subsurface Conditions

Borings were drilled to a maximum depth of 31.5 feet BGS. Exploration locations were selected to observe subsurface soil characteristics in proximity to proposed development areas and are shown on Figure 2. Field logs and observed stratigraphy for encountered materials are presented in Appendix B, *Subsurface Exploration Program.*

5.2.1 Soil Type Description

The geologic units described below were observed during our subsurface exploration: existing pavement section, root zone, undocumented fill, gravel mixtures.

Existing Paved Areas

Pavement sections in existing parking areas and drive aisles were observed to consist of 4 to 6 inches of asphalt underlain by 7 to 12 inches of crushed aggregate.

Root Zone

The grassy area in the southern portion of the site consists of 2 inches of grass and roots. A full topsoil section was not observed and was likely stripped during prior construction activities.

Undocumented Fill

Undocumented fill was observed underlying the pavement section in borings B-2 and B-3. Observed fill consisted of brown, gray, orange, dense sand with silt and gravel and extended to depths of 3 to 6.5 feet BGS. Additional recommendations pertaining to undocumented fill are presented in Section 8.1.2, *Undocumented Fill.*

Gravel Mixtures

Underlying the above materials, native dense to very dense clayey and silty gravels and medium stiff to hard silt and clays with varying proportions of sands and gravels were observed to the maximum explored depth of 31.5 feet BGS. The native deposits had moisture contents ranging from 17 to 30 percent and exhibited low-plasticity behavior.

5.2.2 Groundwater

Groundwater was not observed within the borings to the maximum explored depth of 31.5 feet BGS, however the driller indicated heaving soils at approximately 15 feet BGS in boring B-1. Review of information in our files and nearby well logs presented in Appendix B indicates that groundwater could range from 10 to 20 feet BGS in the vicinity of the site.

Note that groundwater levels are subject to seasonal variance and may rise during extended periods of increased precipitation. Perched groundwater may also be present in localized areas, as indicated. Seeps and springs may become evident during site grading, primarily along slopes or in areas cut below existing grade. Structures, pavements, and drainage design should be planned accordingly.

5.2.3 Infiltration Testing

Infiltration potential of site soils was evaluated through in situ infiltration testing within borings B-1 and B-4. Single-ring, falling head infiltration testing was performed by embedding a drill auger into undisturbed native soil, filling the apparatus with water, and measuring time relative to changes in hydraulic head. Representative soil samples were collected from select test locations and submitted for laboratory analysis. Results of in situ infiltration testing are presented in Table 1.

Table 1. Infiltration Test Results

Test Number	Location (See Figure 2)	Test Depth (feet bgs)	USCS Soil Type (*Indicates Visual Classification)	Passing No. 200 Sieve (%)	Approximate Depth to Groundwater on 04-28-23 (feet bgs)	Measured Infiltration Rate
IT-1.1	SB-1	4.0	GC. Clayey GRAVEL with Sand*	–	Not Encountered to 31.5	Negligible
IT-1.2		7.5	GC. Clayey GRAVEL with Sand	24		Negligible
IT-4.1	SB-4	4.5	SM, Silty SAND with Gravel	20	Not Encountered to 6	Negligible

Based on the presence of fine-textured, low permeability site soils, infiltration is not a feasible option for stormwater management.

6.0 SEISMIC HAZARDS

6.1 Liquefaction

Liquefaction is caused by a rapid increase in pore water pressure that reduces the effective stress between soil particles to near zero. Granular soil, which relies on interparticle friction for strength, is susceptible to liquefaction until the excess pore pressures can dissipate. In general, loose, saturated sand with low silt and clay content is the most susceptible to liquefaction. Silty soil with low plasticity is moderately susceptible to liquefaction under relatively higher levels of ground shaking. Our subsurface exploration program did not encounter soils that are susceptible to liquefaction under design levels of ground shaking.

6.2 Lateral Spreading

Lateral spreading is a liquefaction-related seismic hazard that occurs on gently sloping or flat sites underlain by liquefiable sediment adjacent to an open face, such as a riverbank. Liquefied soil adjacent to an open face can flow toward the open face, resulting in lateral ground displacement.

Since the site soils are not susceptible to liquefaction, lateral spreading is not considered a hazard.

7.0 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. The primary geotechnical considerations for the project were summarized previously in the *Executive Summary*. Specific design and construction recommendations are presented in the following sections.

7.1 Areal Settlement Considerations

A grading plan was not available at the time of this report. We have assumed cuts and fills at the site will be less than 3 feet each. Our experience indicates that fills not exceeding 3 feet above existing grade combined with anticipated footing and floor slab loads are unlikely to exceed the static settlement tolerances of the buildings.

7.2 Shallow Foundation Support

We anticipate maximum column and wall loads for the buildings will be less than 250 kips and 4 kips per foot, respectively. Provided maximum floor slab loading is less than 100 psf, the proposed buildings can be supported by conventional spread footings bearing on firm native soil or engineered structural fill. Provided fills are generally less than 3 feet, foundation construction may occur immediately after fill placement.

Foundations should not be supported by topsoil or undocumented fill material. If encountered, these materials should be improved or removed and replaced with structural fill. If footings are constructed during wet-weather conditions or when 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 8.1.1, *Structural Fill*. Columbia West should observe exposed subgrade conditions prior to placement of crushed aggregate to verify adequate subgrade support.

7.2.1 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 slab.

Footings bearing on subgrade prepared as recommended above should be sized based on an allowable bearing pressure of 2,500 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.

7.2.2 Settlement

Foundations designed in accordance with this report are expected to experience a post construction settlement of less than one inch. Differential post construction settlement between comparably-loaded footing elements is not expected to exceed 0.5 inches.

7.2.3 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 350 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 0.4.

7.2.4 Subgrade Observation

Footing and floor subgrade soils should be evaluated by Columbia West prior to placing forms or reinforcing bar to verify subgrade support conditions are as anticipated in this report. Subgrade observation should confirm that all disturbed material, organic debris, unsuitable fill, remnant topsoil zones, and softened subgrades (if present) have been removed. Over-excavation of footing subgrade soils may be required to remove deleterious material, particularly if footings are constructed during wet-weather conditions.

7.2.5 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. Floor slab settlement and seismic risks were discussed previously in Section 7.1, *Areal Settlement Considerations* and Section 6.0, *Seismic Hazards*.

To provide a capillary break, slabs should be underlain by at least 6 inches of compacted crushed aggregate that has less than 5 percent by dry 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 8.6, *Materials*.

Some flooring manufacturers will only warranty their product if a vapor barrier is installed. Selection of an appropriate vapor barrier should be selected by consulting with the design team.

Slab thickness and reinforcement should be designed by an experienced structural engineer assuming a modulus of subgrade reaction, k, of 125 pci.

7.3 Seismic Design Considerations

Seismic design for proposed structures is prescribed by the 2022 *Oregon Structural Special Code (OSSC)* which refers to *ASCE 7-16.* Based on results of subsurface exploration, site soils meet the criteria for Site Class D. Seismic design parameters for Site Class D are presented in Table 3.9.

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

	Short Period (T_s = 0.2 s)	1 Second Period (T_1 = 1.0 s)
MCE Spectral Acceleration	0.818	0.383
Site Class	D[2]	
Site Coefficient	Fa = 1.173	Fv = 1.92
Adjusted Spectral Response Acceleration	S_{MS} = 0.96	S_{M1} = 0.74
Design Spectral Response Acceleration	S_{DS} = 0.64	S_{D1} = 0.49

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

For Site Class D sites with mapped maximum considered earthquake spectral response acceleration parameter S_1 greater than 0.2, a ground motion hazard analysis may be required according to *ASCE 7-16, Section 11.4.8* unless the seismic response coefficient, C_s, is calculated in accordance with *ASCE 7-16 Section 11.4.8, Exception 2.* However, if an alternative method is utilized to determine the seismic response coefficient, the structure is seismically isolated, or structural damping systems are proposed, *ASCE 7-16* requires a ground motion hazard analysis be conducted. Columbia West recommends that the project structural engineer evaluate these requirements and exceptions to determine if a site-specific ground motion hazard evaluation will be required for proposed structures.

7.4 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 7.2, *Shallow Foundation Support*.

Permanent retaining walls that are not restrained from rotation should be designed for active earth pressures using an equivalent fluid pressure of 35 pcf. Walls that are restrained from rotation should be designed for an at-rest, equivalent fluid pressure of 55 pcf. The recommended earth pressures assume a maximum wall height of 10 feet with well-drained, 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 $7H^2$ 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.

7.4.1 Wall Drainage and Backfill

A minimum 4-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 8.6, *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 8.5.1, *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.

7.5 Pavement Design

7.5.1 Design Parameters and Traffic

Pavement should be installed on firm, competent native subgrade soil or engineered structural fill prepared as described in this report. Our pavement recommendations are based on the following design parameters and assumptions:

- 12 inches of subgrade soil directly below the pavement sections are compacted to at least 95 percent of maximum dry density, as determined by *AASHTO T-99*.
- Resilient moduli for subgrade soil and aggregate base materials were assumed to be 4,500 psi and 20,000 psi, respectively.
- Pavement design life of 20 years with no expected traffic growth.
- Initial and terminal serviceability indices of 4.2 and 2.5, respectively.
- Reliability of 85 percent and standard deviation of 0.4.
- Pavement may be exposed to a fire apparatus load of 75,000 pounds on an infrequent basis.

The specific type and frequency of traffic was not available at the time we prepared this report. Based on experience, we assume that heavy truck traffic will consist of approximately 40 percent FHWA Class Group 6 type trucks (4-axle, single unit) and 60 percent FHWA Class Group 8 type trucks (tractor/trailer 2- to 3-axle). Lightly-loaded drive aisles and parking stalls are expected to service typical passenger vehicle traffic.

7.5.2 Asphaltic Concrete (AC) Pavement Design Sections

Pavement design recommendations for a range of traffic conditions and loading scenarios are presented in Table 4. Material properties and compaction recommendations for asphalt surfacing and crushed aggregate base layers are presented in Section 8.5, *Materials*.

Table 4. Recommended AC Pavement Sections Constructed over Native Soil or Engineered Fill

Traffic	Trucks Per Day	Equivalent Single-Axle Loads (ESALs)	AC Thickness (in)	Base Aggregate Thickness (in)
Passenger Vehicle Parking	0	10,000	2.5	8
Passenger Vehicle Drive Aisles	0	20,000	3	9
Heavy Truck Areas	10	92,000	4	10.5
	25	229,000	4.5	12.5
	50	458,000	5	14
	100	916,000	5.5	16.5

Pavement sections may be reduced in areas where subgrade soils are cement-amended to a minimum depth of 12 inches with a minimum of 6 percent cement by weight. Provided the cement-amended subgrade soil achieves a seven-day unconfined compressive strength of 100 psi, AC pavement sections may be constructed as presented in Table 5.

Table 5. Recommended AC Pavement Sections Constructed over Cement-Amended Subgrade Soil

Traffic	Trucks Per Day	Equivalent Single-Axle Loads (ESALs)	AC Thickness (in)	Base Aggregate Thickness (in)	Cement-Amendment Thickness (in)
Passenger Vehicle Parking	0	10,000	2.5	4	12
Passenger Vehicle Drive Aisles	0	20,000	3	4	
Heavy Truck Areas	10	92,000	4	4	
	25	229,000	4.5	4	
	50	458,000	5	4	
	100	916,000	5.5	6	

7.5.3 General Pavement Recommendations

Recommended pavement section thicknesses are intended to be minimum acceptable values and do not include construction traffic loading. The recommendations assume that pavement construction will be completed during an extended period of warm, dry weather. Wet weather construction may require an increased thickness of base aggregate as discussed later in Section 8.2, *Construction Traffic and Staging*.

Cement-amended soil should be allowed to cure for at least four days prior to aggregate base placement or exposure to construction traffic. Prior to construction traffic access, the cement-amended subgrade should be protected by a minimum 4-inch-thick layer of compacted crushed aggregate. Construction traffic should be limited to dedicated haul roads or non-structural, unpaved portions of the site. Construction traffic should not be permitted on new pavement, unless accounted for in the pavement design section. Base aggregate and cement-amended soils supporting pavement are also not intended for construction traffic. Haul roads and staging areas supporting construction traffic are discussed later in Section 8.2, *Construction Traffic and Staging*.

Asphalt paving is generally not recommended during cold weather conditions where ambient air temperatures are less than 40 degrees Fahrenheit. Compacting asphalt in low-temperature conditions can result in low relative density of the asphalt layer and premature pavement distress.

Asphalt mix designs have a recommended compaction temperature range that is specific to the AC binder used. In low-temperature conditions, maintaining the temperature of the AC mix is difficult as heat can be lost during transport, placement, and compaction. The ambient air temperature during paving should be at least 40 degrees Fahrenheit for a lift thickness greater than 2.5 inches and at least 50 degrees Fahrenheit for a lift thickness between 2 and 2.5 inches. 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.

7.6 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 Wilsonville 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.

Site improvements construction may occur in areas where springs or seepage is present. If encountered during construction, footing drains or subdrains beneath slabs-on-grades can be installed. Figure 4 shows a typical foundation drain detail. Figure 5 shows a typical under slab drainage detail. Figure 6 shows a typical trench detail. A typical drainage mate is shown on Figure 7. Columbia West should determine drainage mat location, extent, and thickness when subsurface conditions are exposed.

8.0 CONSTRUCTION RECOMMENDATIONS

8.1 Site Preparation and Grading

A root zone of 2 inches was observed in the southern grassy area of the site. Root zones approaching 12 inches may be present in other areas of thick vegetation, trees, and shrubs. Approximately 4 to 6 inches of asphalt underlain by 7 to 12 inches of crushed aggregate was observed in existing paved areas of the site. Vegetation, organic material, unsuitable fill, and deleterious material that may be encountered 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.

The required stripping depth may increase in areas of existing fill or previously-existing structures. Actual stripping depths should be determined based upon visual observations made during construction when soil conditions are exposed.

Previously disturbed soil, debris, or undocumented fill encountered during grading or construction activities should be removed completely and thoroughly from structural areas. This includes old remnant foundations, basement walls, utilities, associated soft soils, and debris. Excavation areas should be backfilled with engineered structural fill.

Site grading activities should be performed in accordance with requirements specified in the *2018 International Building Code* (IBC), Chapter 18 and Appendix J, with exceptions noted in the text herein. Site preparation, soil stripping, and grading activities should be observed and documented by Columbia West.

8.1.1 Undocumented Fill

Undocumented fill was observed underlying the existing pavement section at the locations of borings B-2 and B-3. The fill is reported to be between 1.5 and 5 feet thick and generally consisted of sand with varying amounts of silt and gravel.

Existing fill and other previously disturbed soils or debris are not suitable for supporting structures in their current state and should be removed completely removed from the influence zone of foundations. Areas of the site where additional fill is planned, existing fill should be removed until firm native soils are encountered prior to the placement of additional fill.

To minimize long-term risk of adverse impacts to pavement structures, existing fill should also be thoroughly removed from proposed pavement areas. If existing fill is left in place, pavement structures may experience a reduction in long-term serviceability due to premature pavement distress which could include asphalt cracking, localized grade depressions, and inadequate drainage. The decision to construct pavements over existing fill and acceptance of the associated risk should be made by the owner and project stakeholders.

Partial mitigation of premature pavement distress risk may be accomplished by over-excavation and backfill with granular structural fill or application of cement amended materials. Identification of specific engineered mitigation plans is beyond the scope of this report. If this option is selected, Columbia West should be contacted for additional analysis and study, but would likely consist of improving the upper 18-inches of undocumented fill. This can be accomplished by scarifying and compacting it in place, cement emending it, or removing it and replacing it with structural fill.

Based upon Columbia West's investigation, existing fill soils as described appear to be acceptable for reuse as structural fill, provided materials are observed to exhibit index properties similar to those observed during this investigation and that construction adheres to the specifications presented in this report Note that the limited scope of exploration conducted for this investigation cannot wholly eliminate uncertainty regarding the presence of unsuitable soils in areas not explored.

8.1.2 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.

8.2 Construction Traffic and Staging

Near-surface silt and 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 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 silt migration into the base aggregate layer.

As an alternative to thickened aggregate sections, haul roads and staging areas may be constructed using a combination of cement-amended subgrade and crushed aggregate surfacing. If cement-amendment is used, the base aggregate thickness for staging areas and haul roads can typically be reduced to between 6 and 9 inches, respectively. This recommendation is based on a minimum seven-day unconfined compressive strength of 100 psi for the cement-amended soil with a treatment depth of 12 to 16 inches. Based on experience, 6 to 7 percent cement by weight is typically required to achieve the indicated compressive strength.

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.

8.3 Cut and Fill Slopes

Fill slopes should consist of structural fill material as discussed in Section 8.5.1, *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 8. 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 9.

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.

8.4 Excavation

The site was explored to a maximum depth of 31.5 feet BGS with a drill rig. Conventional earthmoving equipment in proper working condition should be capable of making necessary site excavations.

Groundwater was not observed in the borings. Review of information in our files and nearby well logs presented in Appendix B indicates that groundwater could range from 10 to 20 feet BGS in the vicinity of the site.

Temporary excavation sidewalls should maintain a vertical cut to a depth of approximately 4 feet in the near-surface silt and 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 solider piles and timber lagging, sheet pile walls, tiebacks and shotcrete, or pre-fabricated 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.

8.5 Materials

8.5.1 Structural Fill

Areas proposed for fill placement should be appropriately prepared as described in Section 8.1, *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.

8.5.1.1 Onsite Soil

Most onsite native 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 site soil 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 silt and 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.

Onsite soil will likely expand during excavation and transport and consolidate during compaction. Development of site-specific expansion and consolidation factors is beyond the scope of this investigation. We can provide site-specific factors upon request.

8.5.1.2 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.

8.5.1.3 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.

8.5.1.4 Trench Backfill

Trench backfill placed beneath, adjacent to, and for at least 12 inches above utility lines (i.e., the pipe zone) should consist of durable, well-graded granular material with a maximum particle size of 1½ inches, should have less than 7 percent fines by dry weight, and should have at least two mechanically fractured faces. The pipe zone backfill should be compacted to at least 90 percent of the maximum dry density, as determined by *ASTM D1557*, or as required by the pipe manufacturer or local building department.

Within roadway alignments, the remainder of the trench backfill up to the subgrade elevation should consist of durable, well-graded granular material with a maximum particle size of 2½ inches, should have less than 7 percent fines by dry weight, and should have at least two mechanically fractured faces. This material should be compacted to at least 92 percent of the maximum dry density, as determined by *ASTM D1557*, or as required by the pipe manufacturer or local jurisdiction. The upper 3 feet of the trench backfill should be compacted to at least 95 percent of the maximum dry density, as determined by *ASTM D1557.*

Outside of structural improvement areas (e.g., roadway alignments or building pads), trench backfill placed above the pipe zone may consist of general fill material that is free of organic material and material over 6 inches in diameter. This general trench backfill should be compacted to at least 90 percent of the maximum dry density, as determined by *ASTM D1557*, or as required by the pipe manufacturer or local building department.

8.5.1.5 Floor Slab Base Aggregate

Imported granular material used as base rock for building floor slabs should consist of ¾- or 1½-inch-minus material (depending on the application). In addition, the aggregate should have less than 5 percent fines by dry weight and at least two mechanically fractured faces. The aggregate base should be compacted to not less than 95 percent of the maximum dry density, as determined by *ASTM D1557*.

8.5.1.6 Pavement Base Aggregate

Imported granular material used as base rock for pavement should consist of ¾- or 1½-inch-minus material (depending on the application). In addition, the aggregate should have less than 5 percent fines by dry weight and at least two mechanically fractured faces. The aggregate base should be compacted to not less than 95 percent of the maximum dry density, as determined by *ASTM D1557*.

8.5.1.7 Retaining Wall Backfill

Backfill material placed behind retaining walls and extending a horizontal distance of ½H, where H is the height of the retaining wall, should consist of imported granular material as described above and should have less than 7 percent fines by dry weight. We recommend the wall backfill be separated from general fill, native soil, and/or topsoil using a geotextile fabric that meets the specifications provided below for drainage geotextiles.

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

8.5.1.8 Retaining Wall Leveling Pad

Imported granular material placed at the base of retaining wall footings should consist of select granular material. The granular material should be ¾- to 1-inch-minus aggregate size and should have at least two mechanically fractured faces. The leveling pad material should be placed in a 6- to 12-inch-thick lift and compacted to not less than 95 percent of the maximum dry density, as determined by *ASTM D1557*.

8.5.1.9 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.

8.5.1.10 Existing Concrete and Crushed Rock

Concrete and crushed rock from the existing pavement areas and improvements can be used in general structural fill, provided particles greater than 3 inches are not present, it is thoroughly mixed and well graded so that there are no voids between the fragments, and the resulting mix is moisture conditioned for compaction. This material can be used as trench backfill if it meets the requirements for imported granular material, which would require a smaller maximum particle size. The material should be placed in lifts with a maximum uncompacted thickness of 12 inches and compacted to not less than 95 percent of the maximum dry density, as determined by *ASTM D1557*.

8.5.2 Geotextile Fabric

8.5.2.1 Subgrade Geotextile

Subgrade geotextile should conform to OSSC Table 02320-4 and OSSC 00350 (Geosynthetic Installation). A minimum initial aggregate base lift of 6 inches is required over geotextiles. All drainage aggregate and stabilization material should be underlain by a subgrade geotextile.

8.5.2.2 Drainage Geotextile

Drainage geotextile should conform to Type 2 material of OSSC Table 02320-1 and OSSC 00350 (Geosynthetic Installation). A minimum initial aggregate base lift of 6 inches is required over geotextiles.

8.5.3 Soil Amendment with Cement

The on-site soil can be amended with Portland cement to obtain suitable properties for use as wet-weather structural fill or subbase for pavement. The effectiveness of soil amendment is highly dependent on proper mixing techniques, soil moisture conditioning, and the quantity of cement. The quantity of cement applied during amendment should be based on an assumed dry unit weight of 100 pcf for site soil.

8.5.3.1 Subbase Stabilization

Specific recommendations for soil amendment should be based on exposed site conditions at the time of construction. For preliminary design purposes, we recommend cement-amended subgrade for building pads and pavement subbase (below the base aggregate layer) achieve a target strength of 100 psi. The quantity of cement required to achieve the target strength will vary with moisture content and soil type. Laboratory testing of cement-amended soil should be used to confirm design expectations.

Based on our experience, near-surface silt and clay will require approximately 6 to 7 percent cement by weight to achieve the target strength of 100 psi. This cement percentage assumes that the soil moisture content does not exceed 20 percent at the time of amendment. If the soil moisture content is in the range of 25 to 35 percent, 7 to 8 percent cement by weight may be required to achieve the target strength. The amount of cement added to the soil at the time of construction should be based on observed field conditions and subgrade performance. During extended periods of dry weather, water may need to be applied during the amendment and tilling process to achieve the optimum moisture content required for compaction.

Cement-amendment of the agricultural till zone will likely require higher quantities of cement due to the organic content and high-plasticity characteristics of the material. A minimum cement percentage of 7 to 8 percent by weight should be assumed for till zone soil. In addition, increased mixing effort and tilling passes will likely be required to adequately blend the cement into the high plasticity material.

Cement-amendment equipment should have balloon tires to minimize softening, rutting, and disturbance of fine-grained site soil. A sheepsfoot or segmented pad roller with a minimum static weight of 40,000 pounds should be used for initial compaction. Rollers with vibratory action should not be used to compact fine-grained, cement-amended soil. Final compaction should be conducted with a smooth-drum roller with a minimum applied linear force of 700 pounds per inch. The amended soil should be compacted to at least 95 percent of the maximum dry density as determined by *ASTM D558*.

Following cement amendment, a minimum curing time of four days is required prior to exposure to construction traffic. Construction traffic should not be allowed on unprotected, cement-amended subgrade. To protect cement-amended areas from damage, the finished surface should be covered with 4 to 6 inches of imported granular material. The protective layer of crushed rock often becomes contaminated with soil during construction, particularly in staging and haul road areas. Contaminated aggregate, where present, should be removed and replaced with clean crushed aggregate prior to construction of pavement or other permanent site improvements supported by base aggregate.

Cement amendment should not be attempted during moderate to heavy precipitation or when the ambient air temperature is below 40 degrees Fahrenheit. Cement should not be placed in areas of standing water or where saturated subgrade conditions exist.

8.5.3.2 Cement-Amended Structural Fill

If adequate compaction is not achievable with onsite silt and clay due to moisture or weather conditions, the soil may be cement-amended and placed as general structural fill. Prior to placement of cement-amended fill, subgrade soils should be prepared as described in Section 8.1, *Site Preparation and Grading*. Where multiple lifts of cement-amended fill are necessary to meet finished grade, consecutive lifts may be placed immediately following amendment and compaction of the underlying lift. However, where the final lift of cement-amended fill will serve as building pad or pavement subbase material, the four-day cure period as discussed above is recommended.

8.5.3.3 Verification Testing

Cement-amendment of site soils should be observed and tested by Columbia West to document conformance with design recommendations. Cement spread rate should be verified with a pan sample test conducted at one random location per lift per 20,000 square-feet of cement-amended fill. Treatment depth should be verified through excavation of a small test pit and measurement at one random location per lift of cement-amended fill. Adequate compaction and moisture content should be verified by conducting nuclear gauge density testing at a frequency of approximately one test per 5,000 square feet of cement-amended fill in accordance with ASTM D6938. At least one representative sample should be collected per day of cement-amendment, cured for 7 days, and tested for unconfined compressive strength in accordance with ASTM D1633. The tested samples should have a minimum 7-day, unconfined compressive strength of 100 psi.

8.5.3.4 Drainage Considerations

Cement-amended soil will be poorly-drained and will not be suitable for planting areas. The material may also be difficult to excavate with light-duty landscaping equipment. Proposed landscape areas should not be cement-amended unless accommodations are made for drainage and planting.

Cement-amendment within building pad areas should consider the potential for trapped water below the floor slab. Columbia West should be consulted to provide appropriate recommendations if cement-amendment is proposed within building pad areas.

8.5.4 Pavement

8.5.4.1 Asphaltic Concrete

Asphaltic concrete should be Level 2, ½-inch, dense ACP according to OSSC 00744 (Asphalt Concrete Pavement) and compacted to 91 percent of the theoretical maximum density of the mix, as determined by AASHTO T 209. The minimum and maximum lift thicknesses are 2 and 3 inches, respectively, for ½-inch ACP. Asphalt binder should be performance graded and conform to PG 64-22 or better. The binder grade should be adjusted depending on the aggregate gradation and amount of recycled asphalt pavement and/or recycled asphalt shingles in the contractor's mix design submittal.

8.6 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.

9.0 CONCLUSION 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.

This report was prepared solely for the client and is not to be reproduced without prior authorization from Columbia West. Final engineering plans and specifications for the project should be reviewed and approved by Columbia West as they relate to geotechnical and grading issues prior to final design approval. Columbia West is not responsible for independent conclusions or recommendations made by other parties based upon information presented in this report. Unless a particular service was expressly included in the scope, it was not performed and there should be no assumptions based upon services not provided. Additional report limitations and important information about this document are presented in Appendix E. This information should be carefully read and understood by the client and other parties reviewing this document.

Sincerely,

COLUMBIA WEST ENGINEERING, Inc.

Jason F. Merritt, P.E.
Senior Project Engineer

Brett A. Shipton, PE, GE
Principal

REGISTERED PROFESSIONAL ENGINEER
56207
OREGON
July 11, 2000
BRETT A. SHIPTON
EXPIRES: 6/30/24

REFERENCES

Annual Book of ASTM Standards, Soil and Rock (I), v04.08, American Society for Testing and Materials, 1999.

ASCE 7-16, Minimum Design Loads for Buildings and Other Structures, American Society of Civil Engineers, 2016.

Geomatrix Consultants, *Seismic Design Mapping, State of Oregon*, January 1995.

International Building Code: *2018 International Building Code, 2018 edition*, International Code Council, 2018.

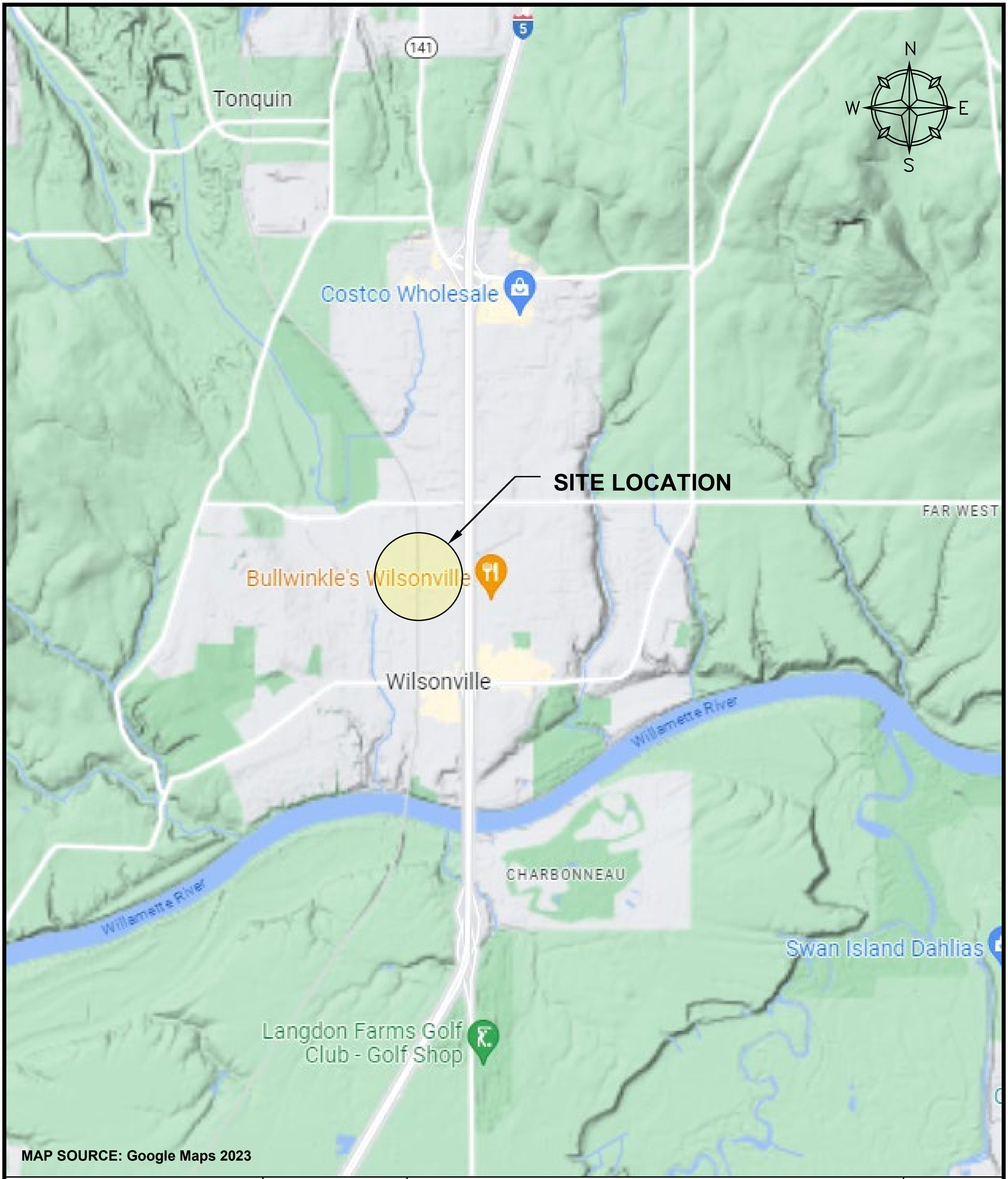
Safety and Health Regulations for Construction, 29 CFR Part 1926, Occupational Safety and Health Administration (OSHA), revised July 1, 2001.

Schlicker, H.G., Finlayson, C.T. *Geology and Geologic Hazards of Northwestern Clackamas County, Oregon;* State of Oregon, Department of Geology and Mineral Industries, 1979

Web Soil Survey, Natural Resources Conservation Service, United States Department of Agriculture, website (http://websoilsurvey.nrcs.usda.gov/app/HomePage.htm).

Wong, Ivan, et al, *Earthquake Scenario and Probabilistic Earthquake Ground Shaking Maps for the Portland, Oregon, Metropolitan Area*, IMS-16, Oregon Department of Geology and Mineral Industries, 2000.

FIGURES



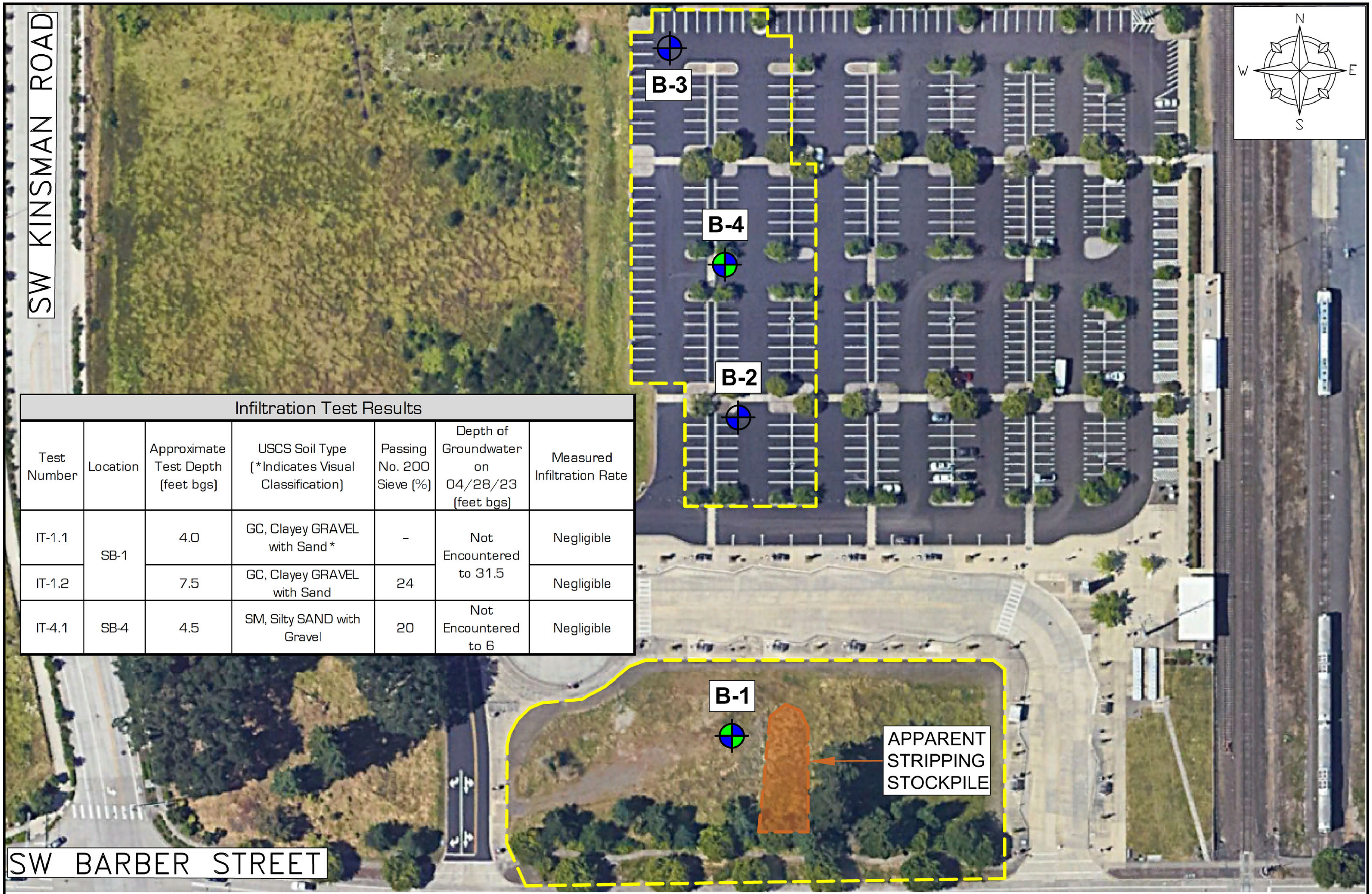
MAP SOURCE: Google Maps 2023

Geotechnical ▪ Environmental ▪ Special Inspections
Columbia West
Engineering, Inc

Job No: 23122
Date: 05/01/23
Drawn: EMU
Checked: JFM

SITE LOCATION MAP
BARBER STREET HOUSING DEVELOPMENT
WILSONVILLE, OREGON

FIGURE
1



Infiltration Test Results

Test Number	Location	Approximate Test Depth (feet bgs)	USCS Soil Type (*Indicates Visual Classification)	Passing No. 200 Sieve (%)	Depth of Groundwater on 04/28/23 (feet bgs)	Measured Infiltration Rate
IT-1.1	SB-1	4.0	GC, Clayey GRAVEL with Sand*	-	Not Encountered to 31.5	Negligible
IT-1.2		7.5	GC, Clayey GRAVEL with Sand	24		Negligible
IT-4.1	SB-4	4.5	SM, Silty SAND with Gravel	20	Not Encountered to 6	Negligible

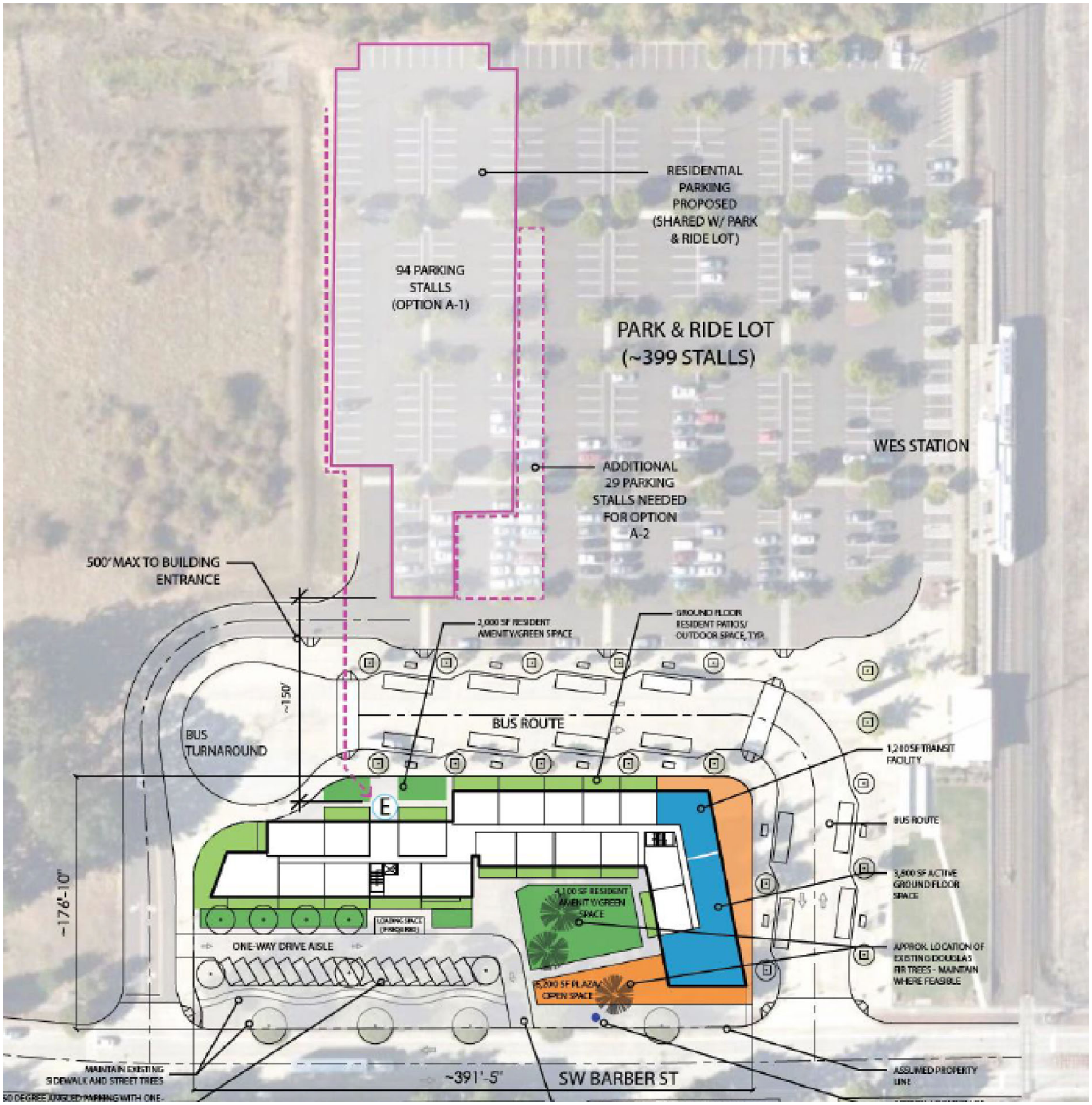
Geotechnical ▪ Environmental ▪ Special Inspections
Columbia West
Engineering, Inc

Job No: 23122
Date: 05/18/23
Drawn: EMU
Checked: JFM

EXPLORATION LOCATION MAP
BARBER STREET HOUSING DEVELOPMENT

NOTES:
1. SITE LOCATION: 9699 SW BARBER STREET IN WILSONVILLE, OREGON
2. SITE CONSISTS OF PORTIONS OF TAX PARCELS 31W14B00703 AND 31W14B00702, TOTALING APPROXIMATELY 2.28 ACRES.
3. AERIAL PHOTO SOURCED FROM GOOGLE EARTH.
4. EXPLORATION LOCATIONS ARE APPROXIMATE AND NOT SURVEYED.
5. BORINGS BACKFILLED WITH BENTONITE ON APRIL 28, 2023.
6. BORINGS B-2, B-3, AND B-4 WHERE CAPPED WITH COLD PATCH ASPHALT ON APRIL 28, 2023.

FIGURE
2



Geotechnical ■ Environmental ■ Special Inspections
Columbia West
Engineering, Inc

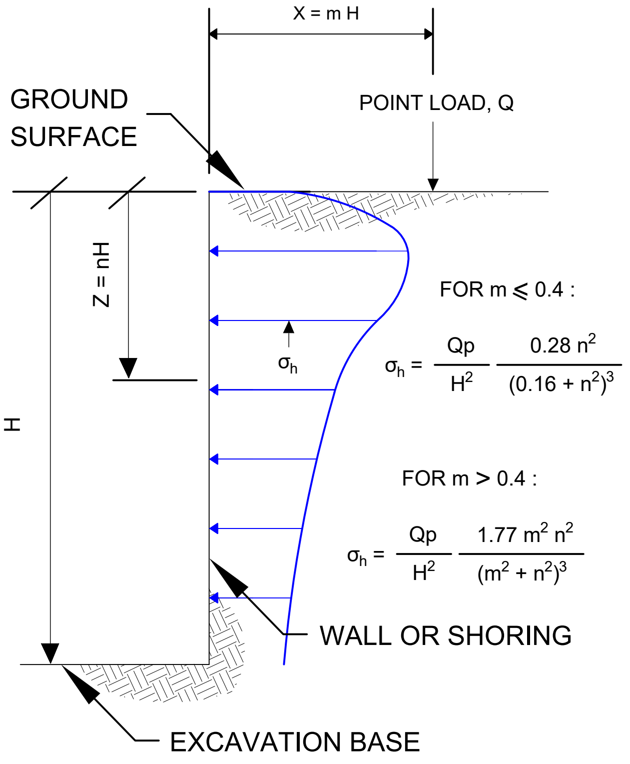
Job No: 23122
Date: 05/15/23
Drawn: EMU
Checked: JFM

PRELIMINARY SITE PLAN
BARBER STREET HOUSING DEVELOPMENT

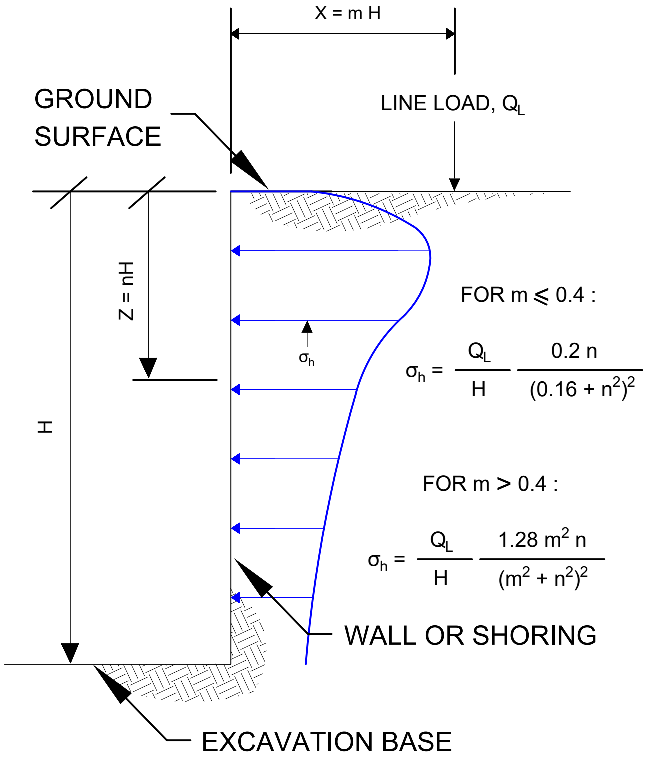
NOTES:
1. PRELIMINARY SITE PLAN PROVIDED BY CLIENT.
2. SITE LOCATION: 9699 SW BARBER STREET IN WILSONVILLE, OREGON
3. SITE CONSISTS OF PORTIONS OF TAX PARCELS 31W14B00703 AND 31W14B00702, TOTALING APPROXIMATELY 2.28 ACRES.

FIGURE
2A

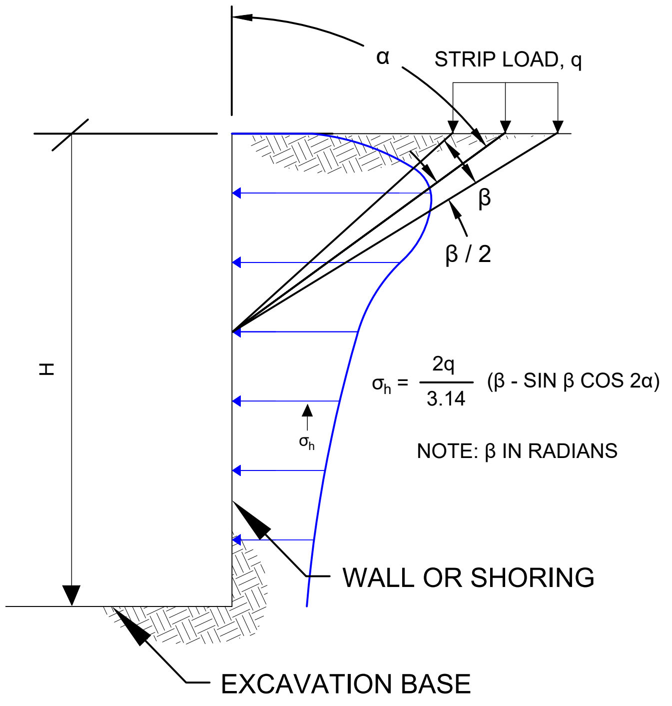
VERTICAL POINT LOAD



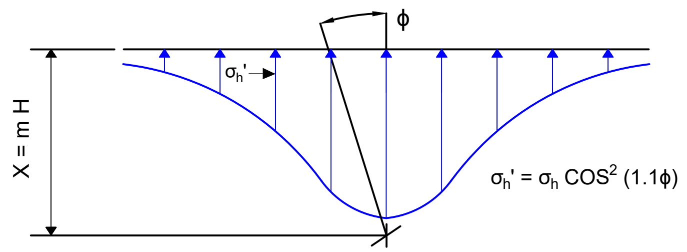
LINE LOAD PARALLEL TO WALL



STRIP LOAD PARALLEL TO WALL



VERTICAL POINT LOAD
HORIZONTAL PRESSURE DISTRIBUTION



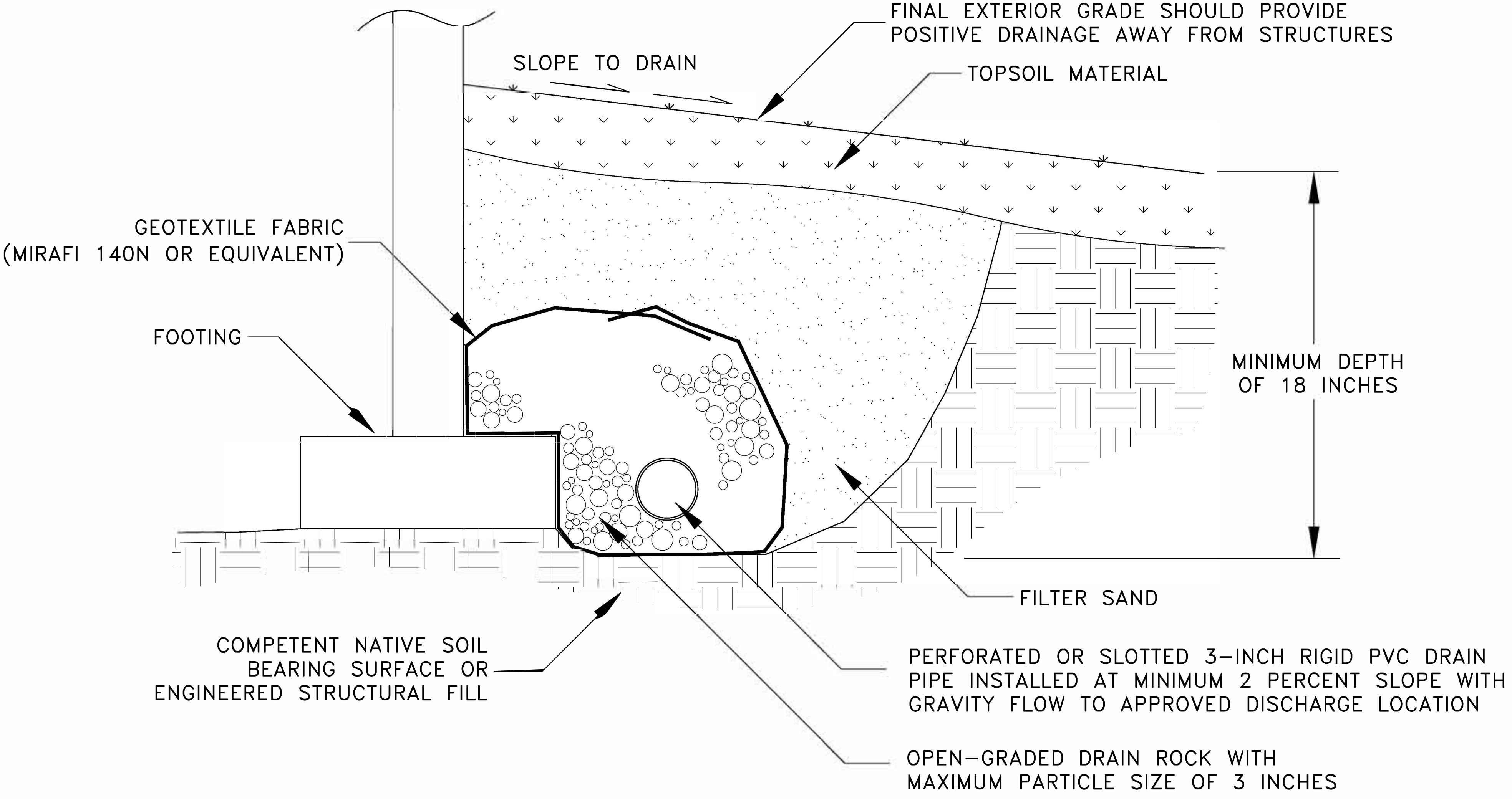
NOTES:

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.

Geotechnical ▪ Environmental ▪ Special Inspections
Columbia West
Engineering, Inc

SURCHARGE–INDUCED LATERAL EARTH PRESSURES

FIGURE
3



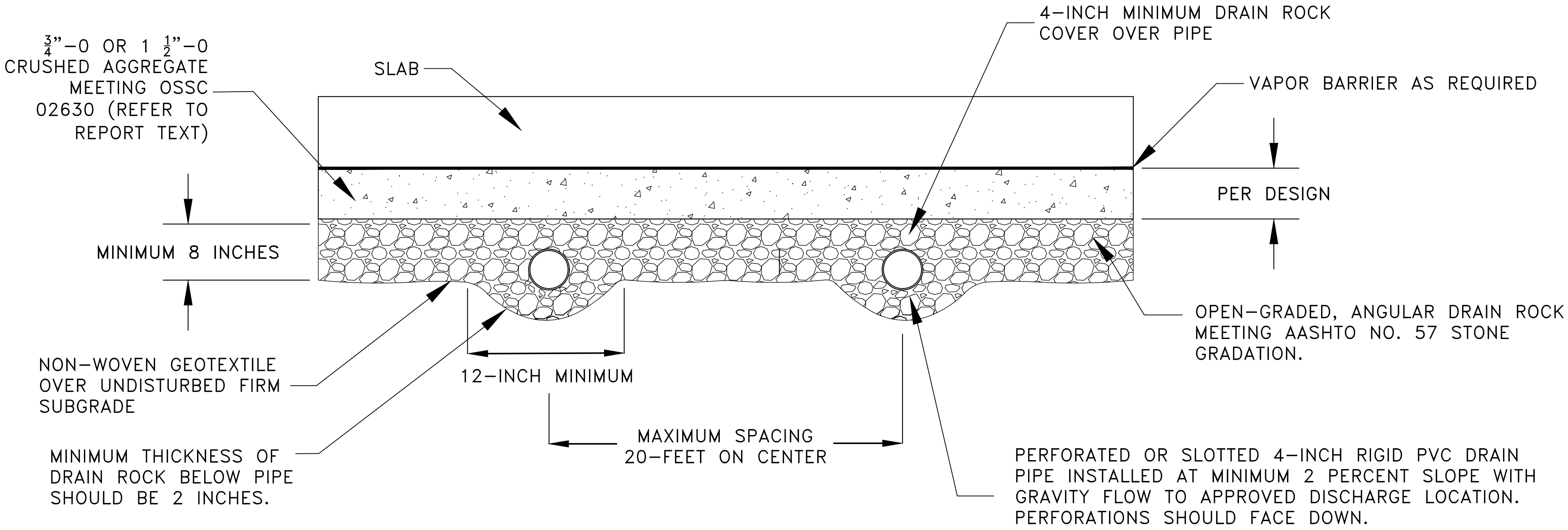
Geotechnical • Environmental • Special Inspections
Columbia West
Engineering, Inc

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

TYPICAL UNDER SLAB DRAINAGE DETAIL

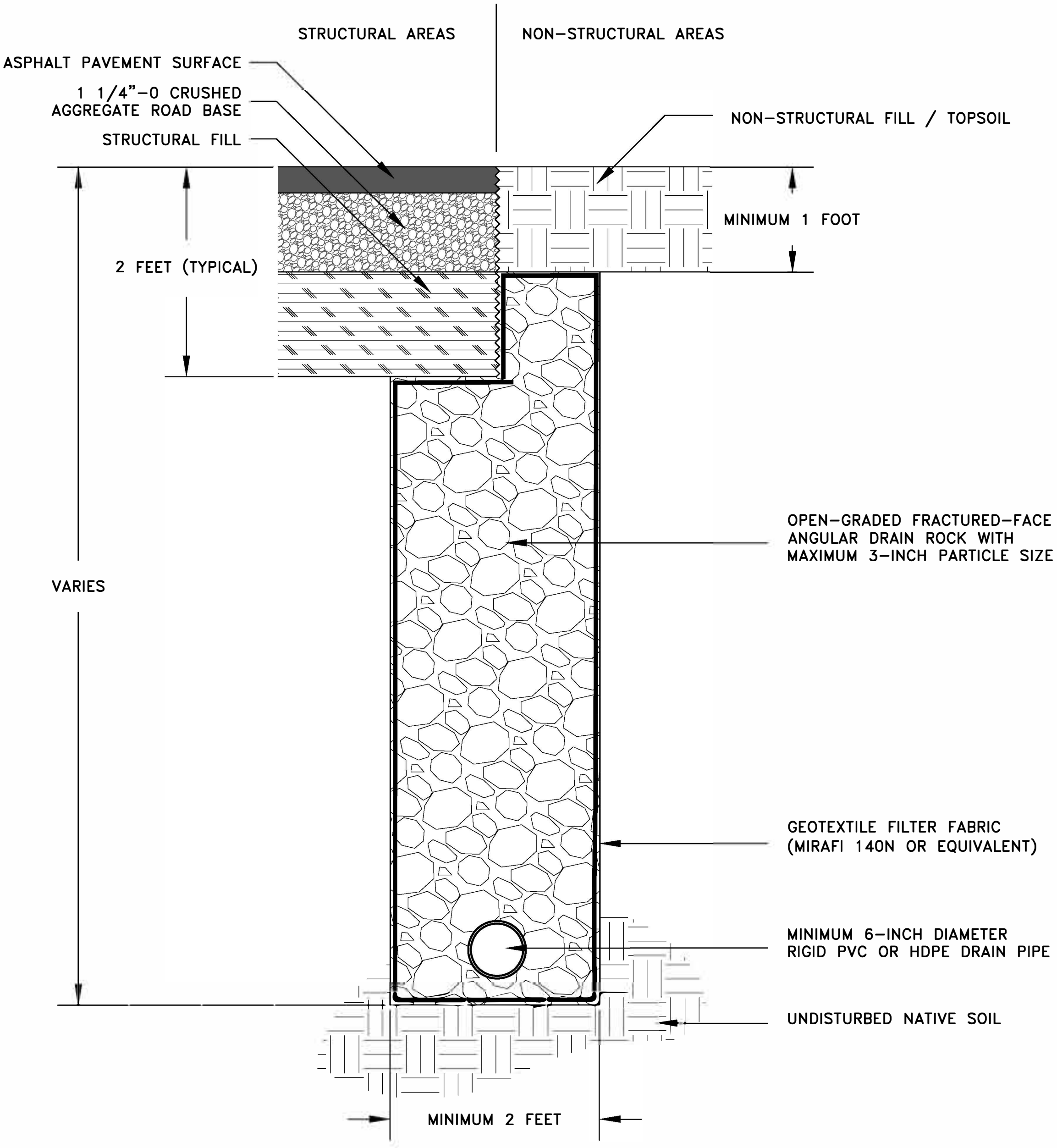


Geotechnical ■ Environmental ■ Special Inspections
Columbia West
Engineering, Inc

TYPICAL UNDER SLAB DRAINAGE DETAIL

NOTES:
1. DRAWING IS NOT TO SCALE.
2. DRAWING REPRESENTS TYPICAL SLAB UNDERDRAIN DETAIL AND MAY NOT BE SITE–SPECIFIC.

FIGURE
5



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.

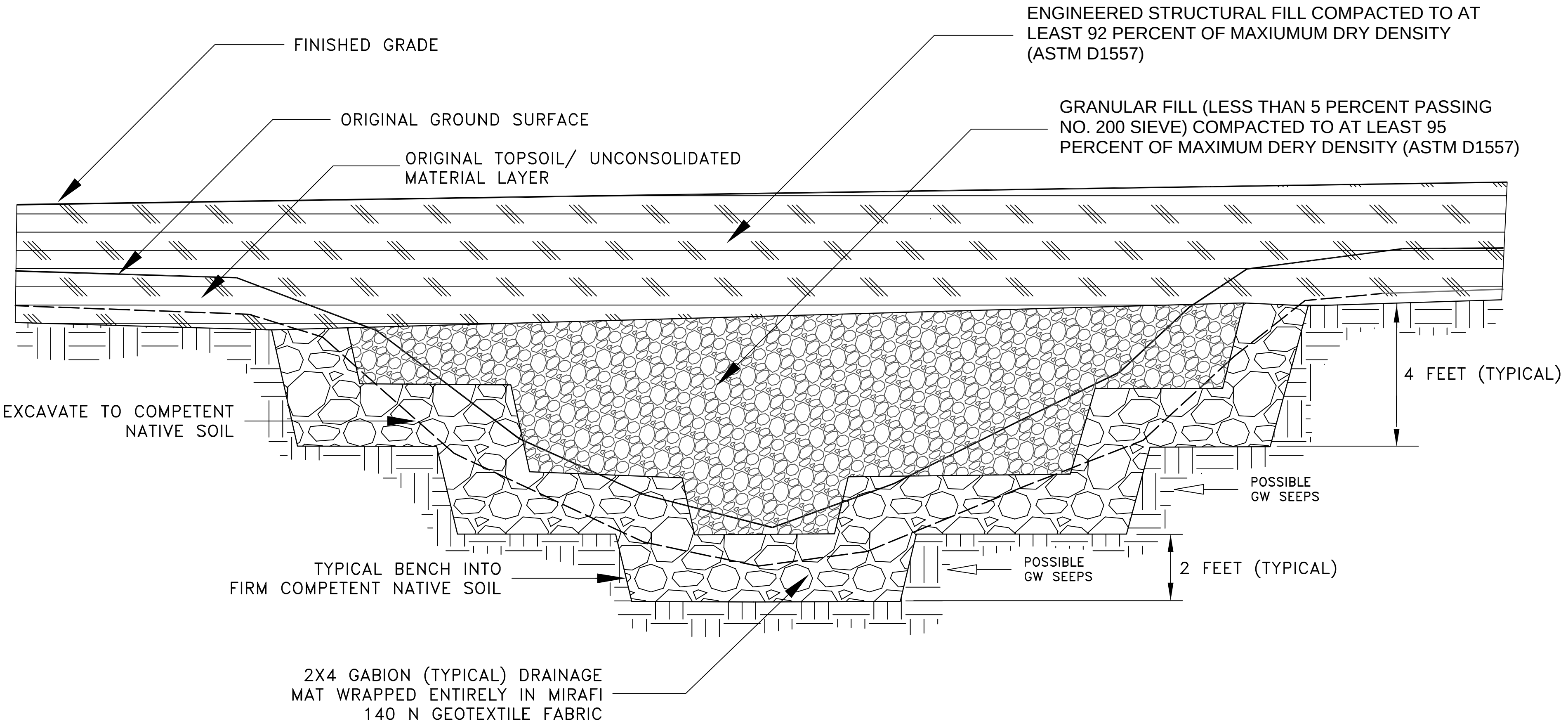
Geotechnical ▪ Environmental ▪ Special Inspections
Columbia West
Engineering, Inc

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
6

TYPICAL DRAINAGE MAT CROSS-SECTION

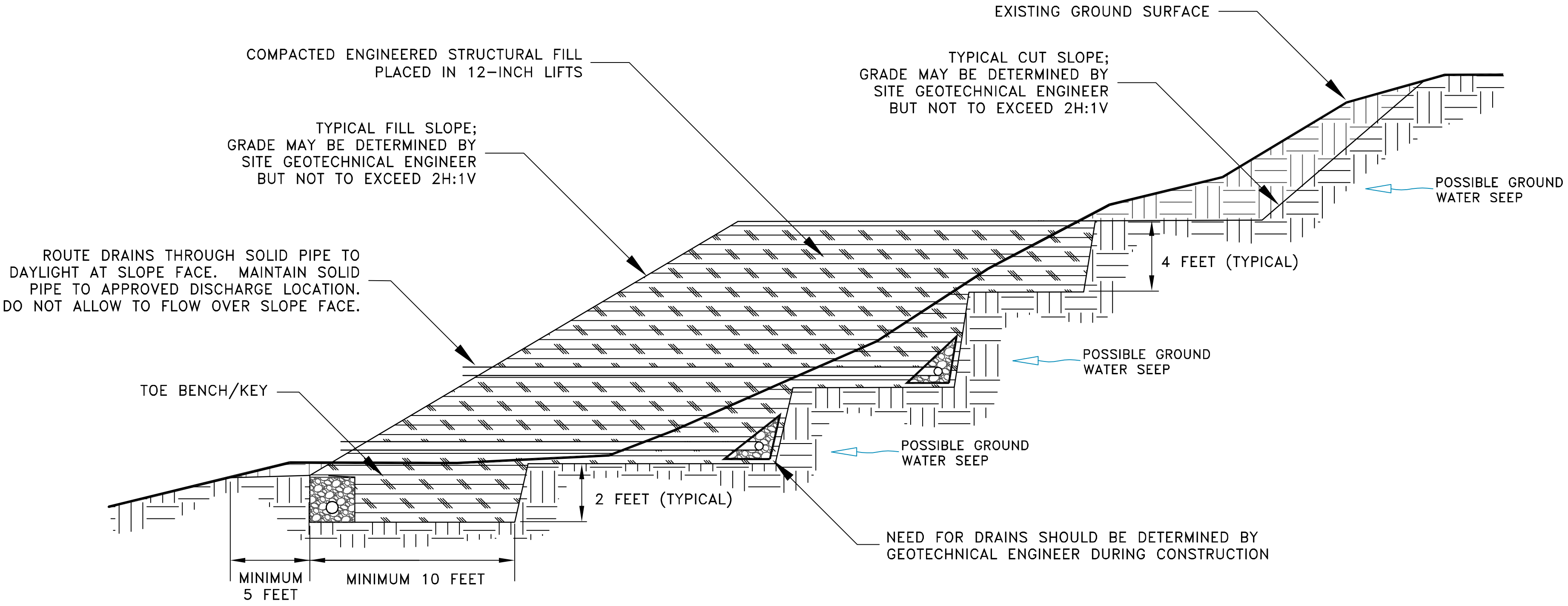


Geotechnical ■ Environmental ■ Special Inspections
Columbia West
Engineering, Inc

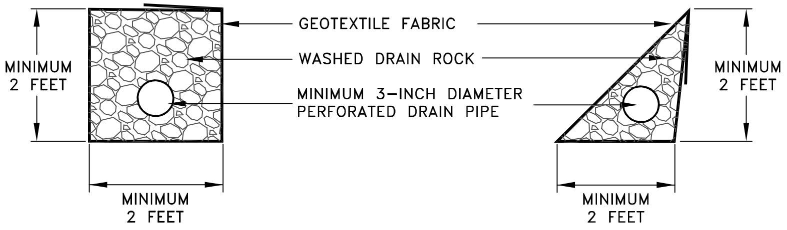
TYPICAL DRAINAGE MAT CROSS SECTION

NOTES:
1. DRAWING IS NOT TO SCALE.
2. DRAWING REPRESENTS TYPICAL DRAINAGE MAT SECTION AND MAY NOT BE SITE-SPECIFIC.

FIGURE 7



TYPICAL DRAIN SECTION DETAIL



DRAIN SPECIFICATIONS

GEOTEXTILE FABRIC SHALL CONSIST OF MIRAFI 140N OR APPROVED EQUIVALENT WITH AOS BETWEEN No. 70 AND No. 100 SIEVE.

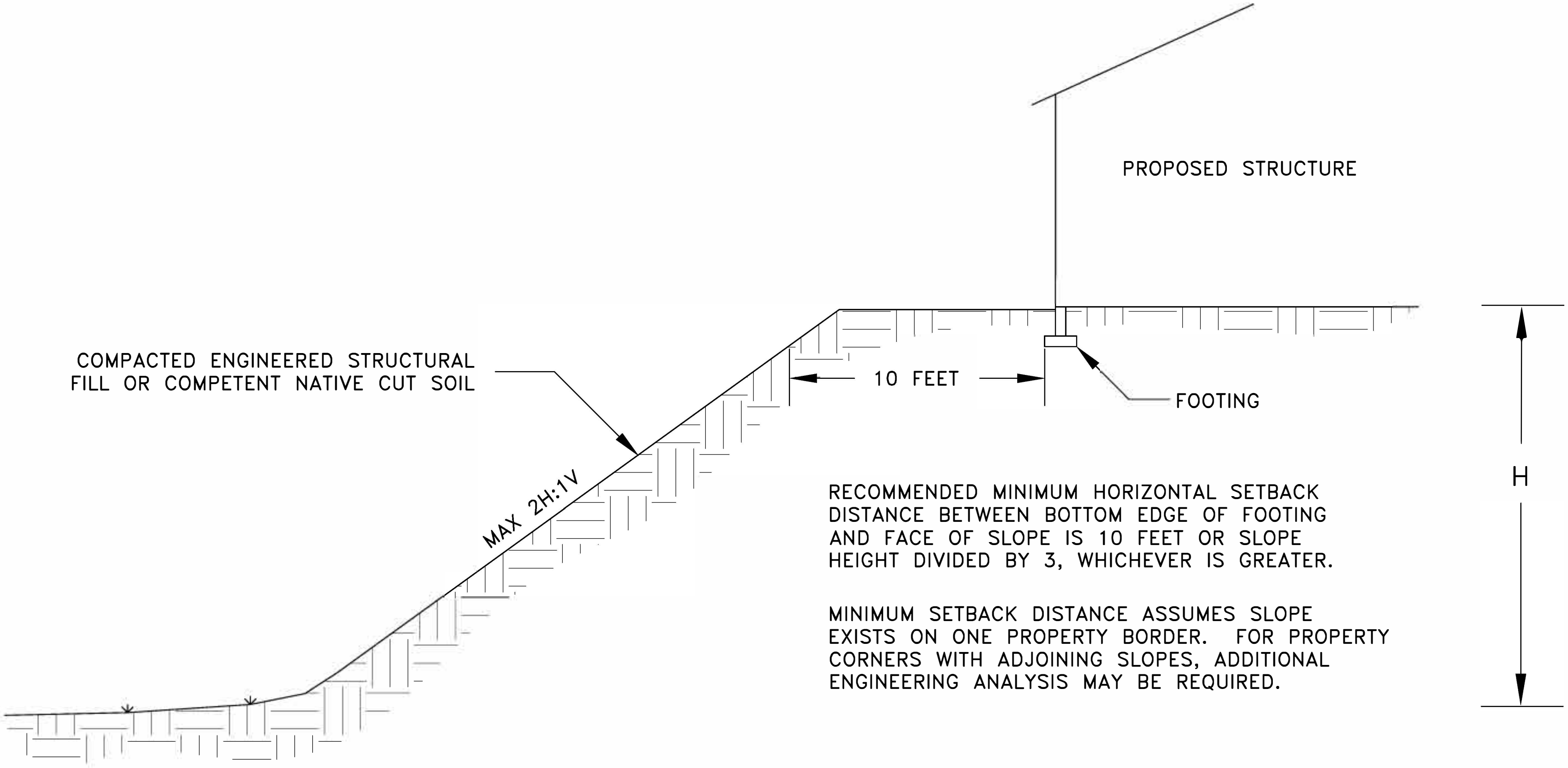
WASHED DRAIN ROCK SHALL BE OPEN-GRADED ANGULAR DRAIN ROCK WITH LESS THAN 2 PERCENT PASSING THE No. 200 SIEVE AND A MAXIMUM PARTICLE SIZE OF 3 INCHES.

Geotechnical ■ Environmental ■ Special Inspections
Columbia West
Engineering, Inc

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
8



RECOMMENDED MINIMUM HORIZONTAL SETBACK DISTANCE BETWEEN BOTTOM EDGE OF FOOTING AND FACE OF SLOPE IS 10 FEET OR SLOPE HEIGHT DIVIDED BY 3, WHICHEVER IS GREATER.

MINIMUM SETBACK DISTANCE ASSUMES SLOPE EXISTS ON ONE PROPERTY BORDER. FOR PROPERTY CORNERS WITH ADJOINING SLOPES, ADDITIONAL ENGINEERING ANALYSIS MAY BE REQUIRED.

Geotechnical ■ Environmental ■ Special Inspections
Columbia West
Engineering, Inc

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
9

APPENDIX A

LABORATORY TESTING RESULTS

CLASSIFICATION

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

MOISTURE CONTENT

We 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 analyses 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. The test results are presented in this appendix.

ATTERBERG LIMITS

We determined the Atterberg Limits on selected samples in general accordance with ASTM D4318. Atterberg limits include the liquid limit, plastic limit, and the plasticity index of soils. These index properties are used to classify soils and for correlation with other engineering properties of soils. The test results are presented in this appendix.

11917 NE 95th Street, Vancouver, Washington 98682
Phone: 360-823-2900
www.columbiawestengineering.com

Geotechnical ■ Environmental ■ Special Inspections
Columbia West
Engineering, Inc

MOISTURE CONTENT, PERCENT PASSING NO. 200 SIEVE BY WASHING

PROJECT
Barber Street Housing Development
Wilsonville, Oregon

CLIENT
Palindrome Communities, LLC
412 NW 5th Avenue
Portland, Oregon 97209

PROJECT NO. 23122

REPORT DATE 05/12/23

DATE SAMPLED 04/28/23

SAMPLED BY EMU

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
S23-0533	215.29	889.64	793.23	638.12	brown-gray Clayey SAND with Gravel	B1.1	2.5 feet	17%	27%
S23-0534	302.14	992.44	890.34	sieved sample	brown Clayey GRAVEL with Sand	B1.3	7.5 feet	17%	24%
S23-0535	341.17	1,063.41	960.29	884.39	brown Silty GRAVEL with Sand	B1.5	15 feet	17%	12%
S23-0536	208.54	513.08	443.02	231.97	blue-gray-brown Lean CLAY	B1.8	30 feet	30%	90%
S23-0537	231.30	1,077.77	1,014.25	n/a	gray-brown SAND with Silt and Gravel	B2.1	1 foot	8%	n/a
S23-0538	243.68	808.64	767.14	n/a	gray-brown SAND with Silt and Gravel	B3.1	1.5 feet	8%	n/a
S23-0539	204.28	957.94	828.11	705.40	brown-gray Silty SAND with Gravel	B4.3	4.5 feet	21%	20%

NOTES:
Sample weights received for Lab ID: S23-0533, 0534, 0537 and 0538 did not meet the minimum size requirements; entire sample used for analysis.

DATE TESTED 05/10/23

TESTED BY KMS

COLUMBIA WEST ENGINEERING, INC. authorized signature

11917 NE 95th Street, Vancouver, Washington 98682
Phone: 360-823-2900
www.columbiawestengineering.com

Geotechnical ▪ Environmental ▪ Special Inspections
Columbia West
Engineering, Inc

PARTICLE-SIZE ANALYSIS REPORT

PROJECT
Barber Street Housing Development
Wilsonville, Oregon

CLIENT
Palindrome Communities, LLC
412 NW 5th Avenue
Portland, Oregon 97209

PROJECT NO. 23122	LAB ID S23-0534
REPORT DATE 05/12/23	FIELD ID B1.3
DATE SAMPLED 04/28/23	SAMPLED BY EMU

MATERIAL DATA

MATERIAL SAMPLED
brown Clayey GRAVEL with Sand

MATERIAL SOURCE
Boring B-01
depth = 7.5 feet

USCS SOIL TYPE
GC, Clayey Gravel with Sand

SPECIFICATIONS
none

AASHTO CLASSIFICATION
A-2-4(0)

LABORATORY TEST DATA

LABORATORY EQUIPMENT
Rainhart "Mary Ann" Sifter, air-dried prep, hand washed, composite sieve - #4 split

TEST PROCEDURE
ASTM D6913, Method A

ADDITIONAL DATA

initial dry mass (g) = 588.15
as-received moisture content = 17%
liquid limit = 31
plastic limit = 21
plasticity index = 10
fineness modulus = n/a

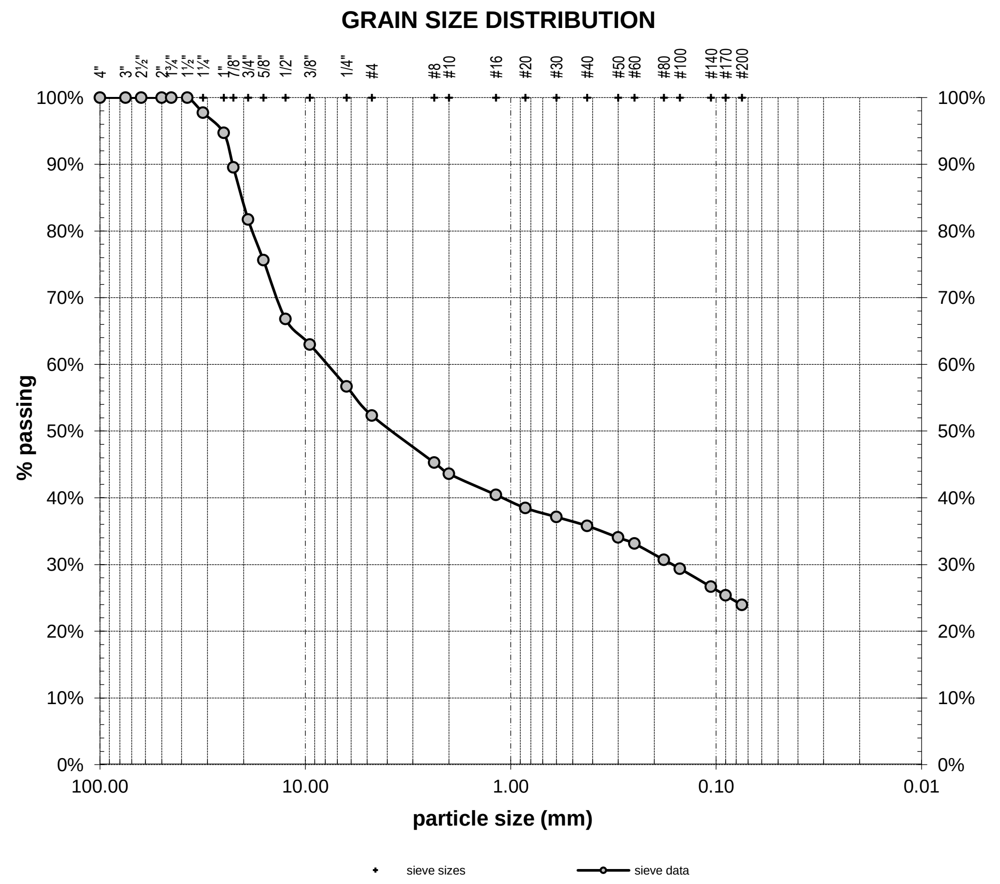
coefficient of curvature, C_C = n/a
coefficient of uniformity, C_U = n/a
effective size, $D_{(10)}$ = n/a
$D_{(30)}$ = 0.163 mm
$D_{(60)}$ = 7.828 mm

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

SIEVE DATA

% gravel = 47.7%
% sand = 28.4%
% silt and clay = 23.9%

	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		98%		
	1.00"	25.0	95%			
	7/8"	22.4		90%		
	3/4"	19.0	82%			
	5/8"	16.0		76%		
	1/2"	12.5	67%			
	3/8"	9.50	63%			
	1/4"	6.30		57%		
	#4	4.75	52%			
SAND	#8	2.36		45%		
	#10	2.00	44%			
	#16	1.18		40%		
	#20	0.850	38%			
	#30	0.600		37%		
	#40	0.425	36%			
	#50	0.300		34%		
	#60	0.250	33%			
	#80	0.180		31%		
	#100	0.150	29%			
	#140	0.106		27%		
	#170	0.090		25%		
	#200	0.075	24%			



DATE TESTED 05/10/23	TESTED BY KMS

COLUMBIA WEST ENGINEERING, INC. authorized signature

11917 NE 95th Street, Vancouver, Washington 98682
Phone: 360-823-2900
www.columbiawestengineering.com

Geotechnical ■ Environmental ■ Special Inspections
Columbia West
Engineering, Inc

ATTERBERG LIMITS REPORT

PROJECT
Barber Street Housing Development
Wilsonville, Oregon

CLIENT
Palindrome Communities, LLC
412 NW 5th Avenue
Portland, Oregon 97209

PROJECT NO. 23122	LAB ID S23-0534
REPORT DATE 05/12/23	FIELD ID B1.3
DATE SAMPLED 04/28/23	SAMPLED BY EMU

MATERIAL DATA

MATERIAL SAMPLED
brown Clayey GRAVEL with Sand

MATERIAL SOURCE
Boring B-01
depth = 7.5 feet

USCS SOIL TYPE
GC, Clayey Gravel with Sand

LABORATORY TEST DATA

LABORATORY EQUIPMENT
Liquid Limit Machine, Hand Rolled

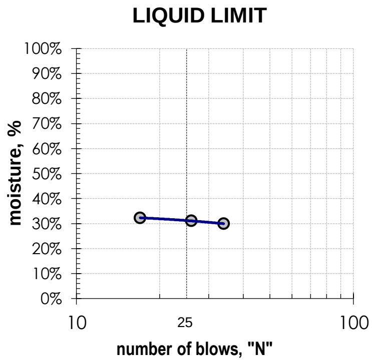
TEST PROCEDURE
ASTM D4318

ATTERBERG LIMITS

liquid limit = 31
plastic limit = 21
plasticity index = 10

LIQUID LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	32.18	33.02	32.90	
dry soil + pan weight, g =	29.60	30.16	30.00	
pan weight, g =	20.98	20.97	21.02	
N (blows) =	34	26	17	
moisture, % =	29.9 %	31.1 %	32.3 %	

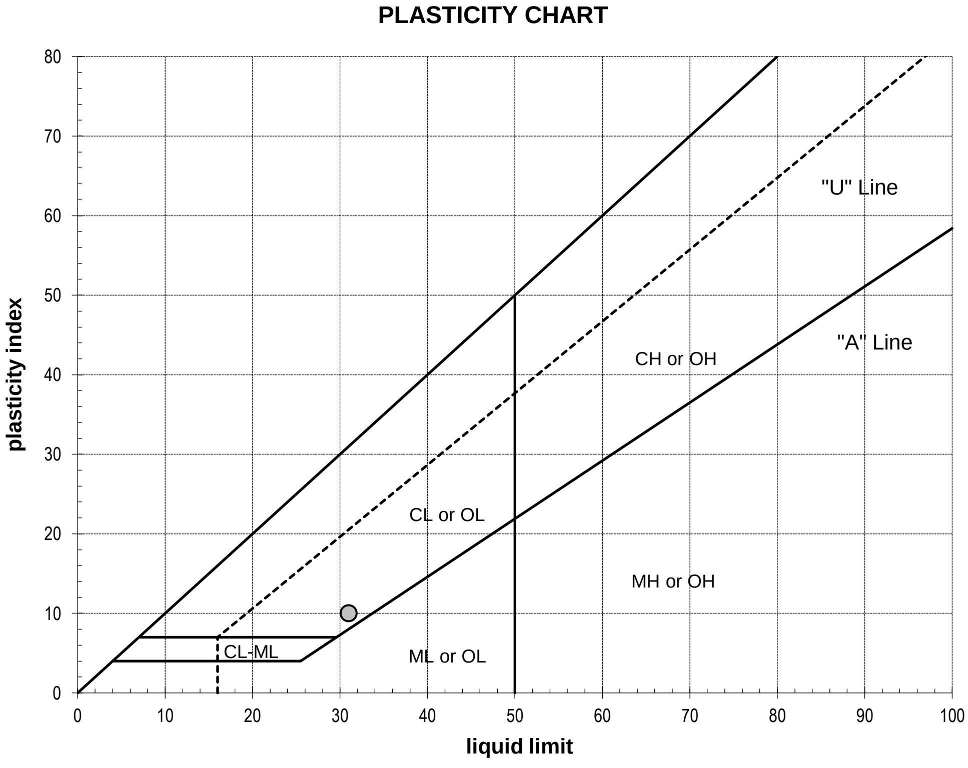


SHRINKAGE

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

PLASTIC LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	27.90	27.48		
dry soil + pan weight, g =	26.66	26.24		
pan weight, g =	20.87	20.42		
moisture, % =	21.4 %	21.3 %		



ADDITIONAL DATA

% gravel = 47.7%
% sand = 28.4%
% silt and clay = 23.9%
% silt = n/a
% clay = n/a
moisture content = 17%

DATE TESTED 05/10/23	TESTED BY MRS/KMS

COLUMBIA WEST ENGINEERING, INC. authorized signature

APPENDIX B

SUBSURFACE EXPLORATION PROGRAM

GENERAL

We explored subsurface conditions at the site by drilling four borings using a truck-mounted drill rig. The borings were drilled by Western States Soil Conservation, Inc. on April 28, 2023, to a maximum depth of 31.5 feet BGS. The boring logs are presented in this appendix.

SOIL SAMPLING

Disturbed samples were collected from the boring at representative depth intervals using 1½-inch diameter split-barrel (SPT) samples in general accordance with ASTM D1586. The sampler was driven into the soil with a 140-poind hammer free falling 30 inches. The sampler was driven a total distance of 18 inches. The number of blows required to drive the sampler the final 12 inches is recorded on the exploration log, unless otherwise noted. The hammer was lifted using an automatic hammer with a reported efficiency of 77.7 percent. A copy of the hammer calibration report is on file at our office. Sampling methods and intervals are shown on the exploration log.

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 soil or their characteristics change, although the change actually could be gradual. If the change occurred between sample locations, the depth was interpreted. Soil classifications are shown on the exploration logs.

NEARBY WELL LOGS

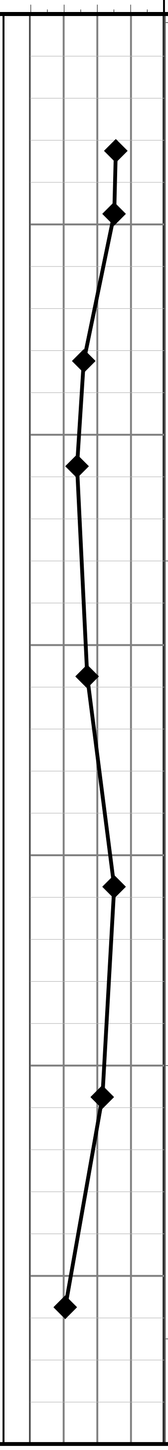
Relevant well logs in the vicinity of the site are presented following the boing logs. Well logs were obtained from the Oregon Water Resource Department.

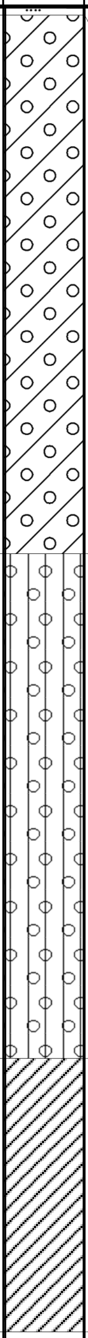
11917 NE 95TH Street, Vancouver, Washington 98682
Phone: 360-823-2900
www.columbiawestengineering.com

Geotechnical ■ Environmental ■ Special Inspections
Columbia West Engineering, Inc

SOIL BORING LOG

PROJECT NAME Baber Street Housing Development	CLIENT Palindrome Wilsonville Limited Partnership		PROJECT NO. 23122	BORING NO. SB-1
PROJECT LOCATION Wilsonville, Oregon	DRILLING CONTRACTOR Western States	DRILL RIG CME75 Truck 9	TECHNICIAN EMU	PAGE NO. 1 of 1
BORING LOCATION See Figure 2	DRILLING METHOD HSA	SAMPLING METHOD SPT	START DATE 04/28/23	START TIME 0820
REMARKS None	GROUNDWATER DEPTH Not encountered		FINISH DATE 04/28/23	FINISH TIME 1130





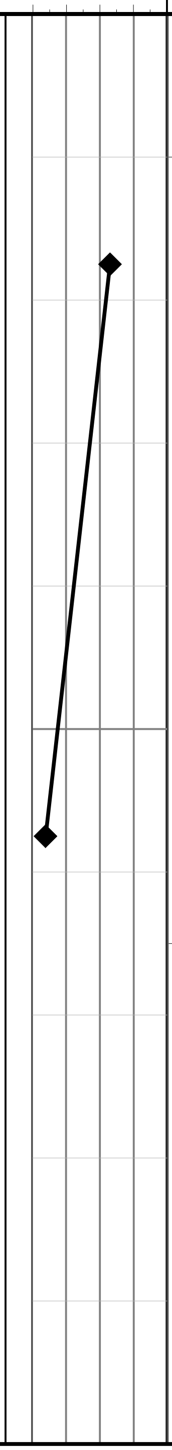
Field ID + Sample Type	SPT N-value (uncorrected)	USCS Soil Type	AASHTO Soil Type	LITHOLOGIC DESCRIPTION AND REMARKS	Infiltration (in/hr)	Moisture Content (%)	Passing No. 200 Sieve (%)	Liquid Limit	Plasticity Index
				2-inch root zone.					
		GC		Clayey GRAVEL with sand, brown and gray, damp, very dense, clay is nonplastic to low plasticity, fine- to medium-textued sand, fractured gravels.					
SPT SB1.1	51					17	27		
				Infiltration test run prior to SPT at 4 feet.					
SPT SB1.2	50				Neg.				
				Infiltration test run prior to SPT at 7.5 feet. Becomes dense at 7.5 feet.					
SPT SB1.3	32				Neg.	17	24	31	10
SPT SB1.4	28								
				Drill started to grind on gravel at 13 feet.					
		GM		Silty GRAVEL with sand, brown, very moist, dense, silt is nonplastic to low plasticity, fine- to coarse-textured sand, fine- to coarse-textued gravel. Driller indicated heaving at 15 feet.					
SPT SB1.5	34					17	12		
				Driller indicated auger was spinning on cobble or boulder at 19 feet. Becomes very dense at 20 feet.					
SPT SB1.6	50								
		CL		Lean CLAY, blue and brown, moist, hard, low to medium plasticity, fine-textured sand.					
SPT SB1.7	43								
				Becomes blue-gray and brown and very stiff at 30 feet.					
SPT SB1.8	21					30	90		
				Boring completed at 31.5 feet bgs. Groundwater not observed on 4/28/23.					

11917 NE 95TH Street, Vancouver, Washington 98682
Phone: 360-823-2900
www.columbiawestengineering.com

Geotechnical ■ Environmental ■ Special Inspections
Columbia West Engineering, Inc

SOIL BORING LOG

PROJECT NAME Baber Street Housing Development	CLIENT Palindrome Wilsonville Limited Partnership		PROJECT NO. 23122	BORING NO. SB-2
PROJECT LOCATION Wilsonville, Oregon	DRILLING CONTRACTOR Western States	DRILL RIG CME75 Truck 9	TECHNICIAN EMU	PAGE NO. 1 of 1
BORING LOCATION See Figure 2	DRILLING METHOD HSA	SAMPLING METHOD SPT	START DATE 04/28/23	START TIME 1150
REMARKS None	GROUNDWATER DEPTH Not encountered		FINISH DATE 04/28/23	FINISH TIME 1218



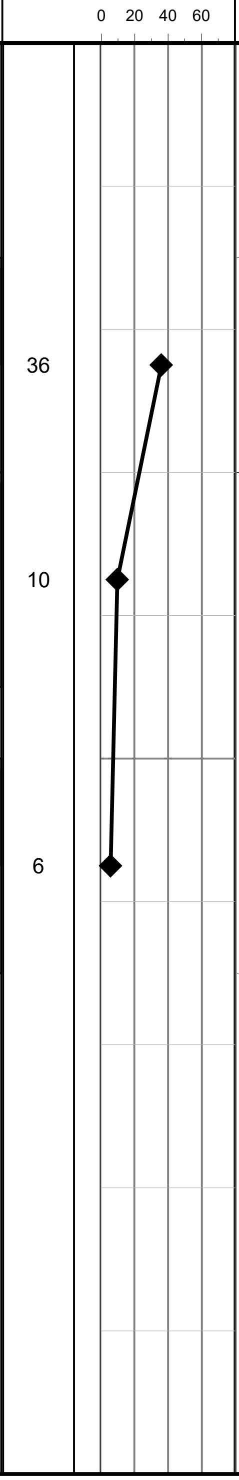
Depth (ft)	Field ID + Sample Type	SPT N-value (uncorrected)	USCS Soil Type	AASHTO Soil Type	Graphic Log	LITHOLOGIC DESCRIPTION AND REMARKS	Infiltration (in/hr)	Moisture Content (%)	Passing No. 200 Sieve (%)	Liquid Limit	Plasticity Index
0					▨	Approximately 4-inches of asphalt underlain by 8-inches of crushed aggregate.					
1	SPT SB2.1	46			▨	FILL. SAND with Silt and Gravel, gray and brown, damp, dense, silt is nonplastic, fine- to coarse-textured sand, fractured gravel.		8			
2					▨						
4					▨						
5	SPT SB2.2	8			▨	Becomes black and brown, moist, and loose at 5 feet.					
6					▨						
6.5						Boring completed at 6.5 feet bgs. Groundwater not observed on 4/28/23.					
8											
10											

11917 NE 95TH Street, Vancouver, Washington 98682
Phone: 360-823-2900
www.columbiawestengineering.com

Geotechnical ▪ Environmental ▪ Special Inspections
Columbia West
Engineering, Inc

SOIL BORING LOG

PROJECT NAME Baber Street Housing Development	CLIENT Palindrome Wilsonville Limited Partnership		PROJECT NO. 23122	BORING NO. SB-3
PROJECT LOCATION Wilsonville, Oregon	DRILLING CONTRACTOR Western States	DRILL RIG CME75 Truck 9	TECHNICIAN EMU	PAGE NO. 1 of 1
BORING LOCATION See Figure 2	DRILLING METHOD HSA	SAMPLING METHOD SPT	START DATE 04/28/23	START TIME 1219
REMARKS None	GROUNDWATER DEPTH Not encountered		FINISH DATE 04/28/23	FINISH TIME 1258



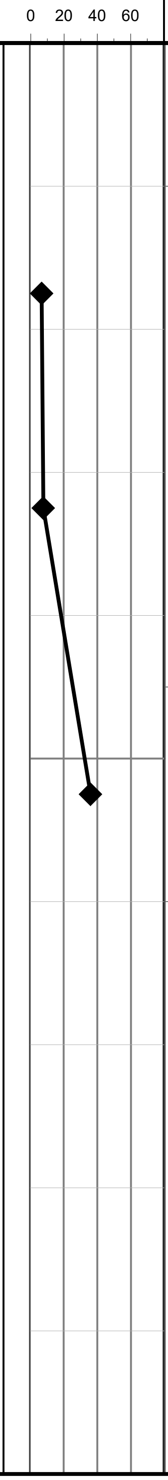
Depth (ft)	Field ID + Sample Type	SPT N-value (uncorrected)	USCS Soil Type	AASHTO Soil Type	Graphic Log	LITHOLOGIC DESCRIPTION AND REMARKS	Infiltration (in/hr)	Moisture Content (%)	Passing No. 200 Sieve (%)	Liquid Limit	Plasticity Index
0					▩	Approximately 6-inches of asphalt underlain by 12-inches of crushed aggregate.					
2	SPT SB3.1	36			▩	FILL. SAND with Silt and Gravel, gray, orange, and brown, damp, dense, silt is nonplastic, fine- to coarse-textured sand, fractured gravel.		8			
4	SPT SB3.2	10	ML		▥	Sandy SILT with gravel, black and brown, moist, stiff, low plasticity, fine-textued sand, fine-textured gravel.					
6	SPT SB3.3	6			▥	Becomes Sandy SILT and medium stiff at 5 feet.					
8 10						Boring completed at 6.5 feet bgs. Groundwater not observed on 4/28/23.					

11917 NE 95TH Street, Vancouver, Washington 98682
Phone: 360-823-2900
www.columbiawestengineering.com

Geotechnical ■ Environmental ■ Special Inspections
Columbia West
Engineering, Inc

SOIL BORING LOG

PROJECT NAME Baber Street Housing Development	CLIENT Palindrome Wilsonville Limited Partnership		PROJECT NO. 23122	BORING NO. SB-4
PROJECT LOCATION Wilsonville, Oregon	DRILLING CONTRACTOR Western States	DRILL RIG CME75 Truck 9	TECHNICIAN EMU	PAGE NO. 1 of 1
BORING LOCATION See Figure 2	DRILLING METHOD HSA	SAMPLING METHOD SPT	START DATE 04/28/23	START TIME 1252
REMARKS None	GROUNDWATER DEPTH Not encountered		FINISH DATE 04/28/23	FINISH TIME 1400



Depth (ft)	Field ID + Sample Type	SPT N-value (uncorrected)	USCS Soil Type	AASHTO Soil Type	Graphic Log	LITHOLOGIC DESCRIPTION AND REMARKS	Infiltration (in/hr)	Moisture Content (%)	Passing No. 200 Sieve (%)	Liquid Limit	Plasticity Index
0						Approximately 5-inches of asphalt underlain by 7-inches of crushed aggregate.					
	SPT		ML			SILT with Sand, brown and gray, moist, medium stiff, low plasticity, fine-textured sand.					
		7									
2											
	SB4.1 SPT					Becomes brown and medium stiff to stiff at 2.5 feet.					
		8									
4	SB4.2										
						Infiltration test peformed before SPT at 4.5 feet.					
	SPT		SM			Silty SAND with gravel, brown and gray, moist, dense, silt is nonplastic to low plasticity, fine- to coarse-textured sand, fine-textured gravels.					
		36					Neg.	21	20		
6	SB4.3					Boring completed at 6 feet bgs. Groundwater not observed on 4/28/23.					
8											
10											

APPENDIX C
SOIL AND ROCK CLASSIFICATION INFORMATION

SOIL DESCRIPTION AND CLASSIFICATION GUIDELINES

Particle-Size Classification

COMPONENT	**ASTM/USCS**		**AASHTO**	
	size range	sieve size range	size range	sieve size range
Cobbles	> 75 mm	greater than 3 inches	> 75 mm	greater than 3 inches
Gravel	75 mm – 4.75 mm	3 inches to No. 4 sieve	75 mm – 2.00 mm	3 inches to No. 10 sieve
Coarse	75 mm – 19.0 mm	3 inches to 3/4-inch sieve	-	-
Fine	19.0 mm – 4.75 mm	3/4-inch to No. 4 sieve	-	-
Sand	4.75 mm – 0.075 mm	No. 4 to No. 200 sieve	2.00 mm – 0.075 mm	No. 10 to No. 200 sieve
Coarse	4.75 mm – 2.00 mm	No. 4 to No. 10 sieve	2.00 mm – 0.425 mm	No. 10 to No. 40 sieve
Medium	2.00 mm – 0.425 mm	No. 10 to No. 40 sieve	-	-
Fine	0.425 mm – 0.075 mm	No. 40 to No. 200 sieve	0.425 mm – 0.075 mm	No. 40 to No. 200 sieve
Fines (Silt and Clay)	< 0.075 mm	Passing No. 200 sieve	< 0.075 mm	Passing No. 200 sieve

Consistency for Cohesive Soil

CONSISTENCY	**SPT N-VALUE (BLOWS PER FOOT)**	**D&M N-VALUE (BLOWS PER FOOT)**	**POCKET PENETROMETER (UNCONFINED COMPRESSIVE STRENGTH, tsf)**
Very Soft	Less than 2	Less than 3	less than 0.25
Soft	2 to 4	3 to 6	0.25 to 0.50
Medium Stiff	4 to 8	6 to 12	0.50 to 1.0
Stiff	8 to 15	12 to 25	1.0 to 2.0
Very Stiff	15 to 30	25 to 65	2.0 to 4.0
Hard	30 to 60	65 to 145	greater than 4.0
Very Hard	greater than 60	greater than 145	-

RELATIVE DENSITY	**SPT N-VALUE (BLOWS PER FOOT)**	**D&M N-VALUE (BLOWS PER FOOT)**
Very Loose	0 to 4	0 to 11
Loose	4 to 10	11 to 26
Medium Dense	10 to 30	26 to 74
Dense	30 to 50	74 to 120
Very Dense	more than 50	More than 120

Relative Density for Granular Soil

Moisture Designations

TERM	**FIELD IDENTIFICATION**
Dry	No moisture. Dusty or dry.
Damp	Some moisture. Cohesive soils are usually below plastic limit and are moldable.
Moist	Grains appear darkened, but no visible water is present. Cohesive soils will clump. Sand will bulk. Soils are often at or near plastic limit.
Wet	Visible water on larger grains. Sand and silt exhibit dilatancy. Cohesive soil can be readily remolded. Soil leaves wetness on the hand when squeezed. Soil is much wetter than optimum moisture content and is above plastic limit.

Additional Constituents

Percent	Silt and Clay In:		Percent	Sand and Gravel In:	
	Fine-Grained Soil	Coarse-Grained Soil		Fine-Grained Soil	Coarse-Grained Soil
< 5	trace	trace	< 5	trace	trace
5 – 12	minor	with	5 – 15	minor	minor
> 12	some	silty/clayey	15 – 30	with	with
			> 30	sandy/gravelly	with Indicate approx. percentage

AASHTO SOIL CLASSIFICATION SYSTEM

TABLE 1. Classification of Soils and Soil-Aggregate Mixtures

	Granular Materials			Silt-Clay Materials			
General Classification	(35 Percent or Less Passing .075 mm)			(More than 35 Percent Passing 0.075)			
Group Classification	A-1	A-3	A-2	A-4	A-5	A-6	A-7
Sieve analysis, percent passing:							
2.00 mm (No. 10)	-	-	-				
0.425 mm (No. 40)	50 max	51 min	-	-	-	-	-
0.075 mm (No. 200)	25 max	10 max	35 max	36 min	36 min	36 min	36 min
Characteristics of fraction passing 0.425 mm (No. 40)							
Liquid limit				40 max	41 min	40 max	41 min
Plasticity index	6 max	N.P.		10 max	10 max	11 min	11 min
General rating as subgrade	Excellent to good			Fair to poor			

Note: The placing of A-3 before A-2 is necessary in the "left to right elimination process" and does not indicate superiority of A-3 over A-2.

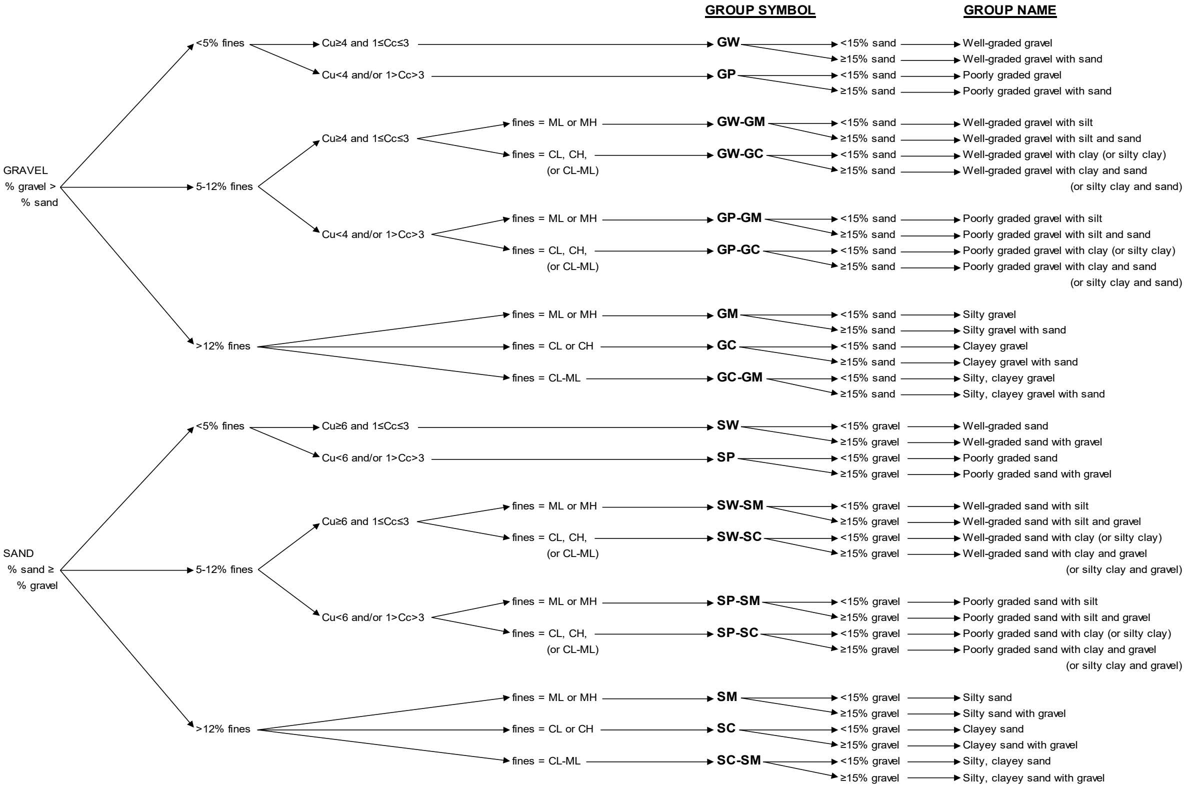
TABLE 2. Classification of Soils and Soil-Aggregate Mixtures

	Granular Materials							Silt-Clay Materials			
General Classification	(35 Percent or Less Passing 0.075 mm)							(More than 35 Percent Passing 0.075 mm)			
	A-1			A-2							A-7
Group Classification	A-1-a	A-1-b	A-3	A-2-4	A-2-5	A-2-6	A-2-7	A-4	A-5	A-6	A-7-5, A-7-6
Sieve analysis, percent passing:											
2.00 mm (No. 10)	50 max	-	-	-	-	-	-	-	-	-	-
0.425 mm (No. 40)	30 max	50 max	51 min	-	-	-	-	-	-	-	-
0.075 mm (No. 200)	15 max	25 max	10 max	35 max	35 max	35 max	35 max	36 min	36 min	36 min	36 min
Characteristics of fraction passing 0.425 mm (No. 40)											
Liquid limit				40 max	41 min	40 max	41 min	40 max	41 min	40 max	41 min
Plasticity index	6 max		N.P.	10 max	10 max	11 min	11 min	10 max	10 max	11 min	11min
Usual types of significant constituent materials	Stone fragments, gravel and sand		Fine sand	Silty or clayey gravel and sand				Silty soils		Clayey soils	
General ratings as subgrade	Excellent to Good							Fair to poor			

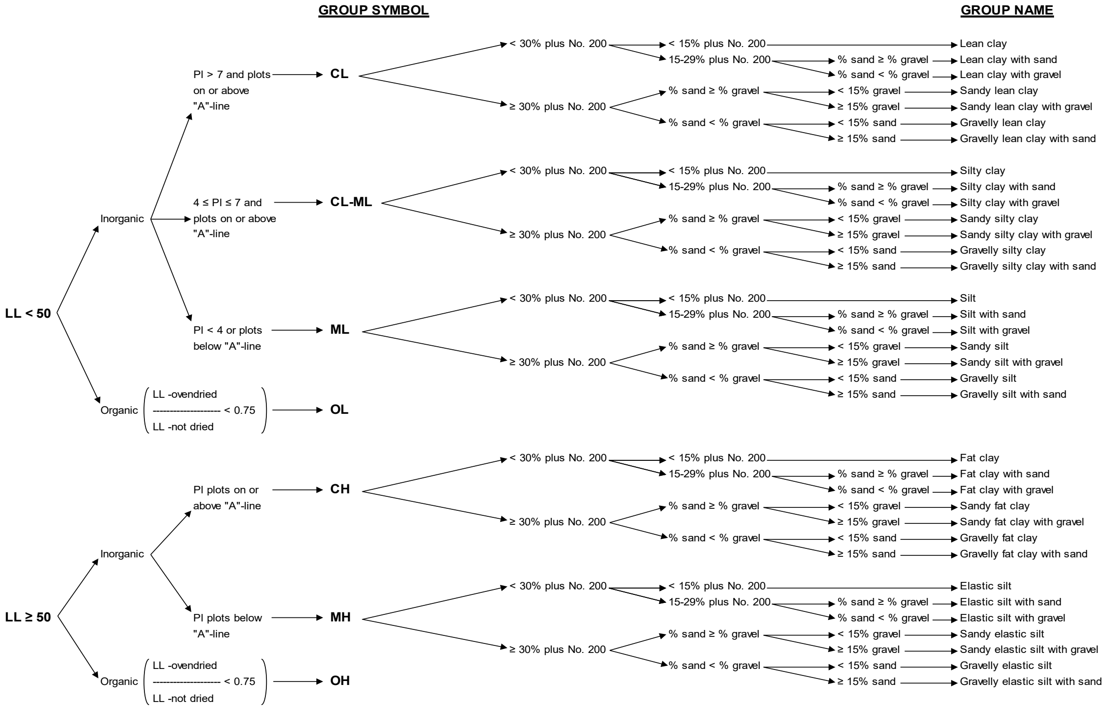
Note: Plasticity index of A-7-5 subgroup is equal to or less than LL minus 30. Plasticity index of A-7-6 subgroup is greater than LL minus 30 (see Figure 2).

AASHTO = American Association of State Highway and Transportation Officials

UNIFIED SOIL CLASSIFICATION SYSTEM



Flow Chart for Classifying Coarse-Grained Soils (More Than 50% Retained on No. 200 Sieve)



Flow Chart for Classifying Fine-Grained Soil (50% or More Passes No. 200 Sieve)

ROCK CLASSIFICATION INFORMATION

ROCK HARDNESS	DESCRIPTION	UNCONFINED COMPRESSIVE STRENGTH (PSI)
Extremely Soft (R0)	Easily indented and scratched by fingernail - soil like texture	<100
Very Soft (R1)	Scratched with fingernail, peeled by knife, indented by rock pick	100 - 200
Soft (R2)	Peeled by knife, indented by rock pick (moderate difficulty)	200 - 800
Moderately Soft (R3)	Peeled by knife, indented by rock pick (with difficulty)	800 - 1,800
Moderately Hard (R4)	Scratched by knife or rock pick, cannot be peeled	1,800 - 7,300
Hard (R5)	Scratched by knife or rock pick (with difficulty)	7,300 - 14,500
Very Hard (R6)	Cannot be scratched with knife or rock pick	14,500 - 36,300
Extremely Hard (R7)	Can only be chipped, not broken by repeated blows with rock pick	> 36,300

ROCK WEATHERING	DESCRIPTION
Decomposed	Completely decomposed - mass structure is disintegrated to a soil
Completely Weathered	Completely decomposed - mass structure is largely intact
Highly Weathered	> 50% of rock is decomposed, fresh or discolored rock is present
Moderately Weathered	< 50% of rock is decomposed, fresh or discolored rock is present
Slightly Weathered	Discoloration indicates weathering and discontinuity surfaces
Fresh	No visible weathering, slight discoloration on discontinuity surfaces

ROCK QUALITY	RQD (%)
Very poor (Completely weathered rock)	<25%
Poor (Weathered rocks)	25 to 50%
Fair (Moderately weathered rocks)	51 to 75%
Good (Hard Rock)	76 to 90%
Very Good (Fresh rocks)	91 to 100%

Rock Quality Designation (RQD) is a measure of quality of rock core taken from a borehole. The length of core pieces is measured along center line of the pieces. All pieces of intact rock core equal to or greater than 100 mm (4 in.) long are summed and divided by the total length of the core run to obtain RQD value

ROCK JOINT SPACING	DESCRIPTION
Very Close	< 0.2 foot
Close	0.2 foot - 1 foot
Moderately Close	1 foot - 3 feet
Wide	3 feet - 10 feet
Very Wide	> 10 feet

ROCK FRACTURING	DESCRIPTION
Very Intensely Fractured	Chips, fragments, with scattered short core lengths
Intensely Fractured	0.1 foot - 0.3 foot with scattered fragments
Moderately Fractured	0.3 foot - 1 foot
Slightly Fractured	1 foot - 3 feet
Very Slightly Fractured	> 3 feet
Unfractured	No fractures observed

ROCK HEALING	DESCRIPTION
Not Healed	Discontinued surface, fractured zone, sheared material, filling is not cemented
Partly Healed	Fractured/sheared material - bonded is < 50%
Moderately Healed	Fractured/sheared material - bonded is > 50%
Totally Healed	All fragments are bonded

APPENDIX D
PHOTO LOG

Barber Street Housing Project

April, 2023

Wilsonville, Oregon

North Site Area, Facing North

Barber Street Housing Project

April, 2023

Wilsonville, Oregon

Southwestern Site Area, Facing East

Barber Street Housing Project

April, 2023

Wilsonville, Oregon

Southeastern Site Area, Facing West

Barber Street Housing Project

April, 2023

Wilsonville, Oregon

Split Spoon Sample, SB1.3 Depth 7.5 feet

Barber Street Housing Project

April, 2023

Wilsonville, Oregon

Split Spoon Sample, SB3.3 Depth 5 feet

APPENDIX E
REPORT LIMITATIONS AND IMPORTANT INFORMATION

Geotechnical ▪ Environmental ▪ Special Inspections
Columbia West
Engineering, Inc

Date: May 18, 2023
Project: Barber Street Housing Development
Wilsonville, Oregon

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.

Geotechnical ▪ Environmental ▪ Special Inspections
Columbia West
Engineering, Inc

Vancouver, Washington • Phone: 360-823-2900
Portland, Oregon • Phone: 971-384-1666
www.columbiawestengineering.com

December 7, 2023

Palindrome Wilsonville Limited Partnership
412 NW 5th Avenue
Portland, Oregon 97209

Attn: Jason Ellis

Re: Report of Geotechnical Engineering Services
Barber Street Housing Development
Supplemental Infiltration Testing
9699 SW Barber Street
Wilsonville, Oregon
CWE Project: Palindrome-3-01-1

INTRODUCTION

Columbia West Engineering, Inc. (Columbia West) is pleased to submit this report of geotechnical engineering services for the Barber Street Housing Development located at 9699 SW Barber Street in Wilsonville, Oregon. Columbia West previously prepared the following geotechnical documents for the project:

- Columbia West Engineering, Inc., *Geotechnical Site Investigation, Barber Street Housing Development, Wilsonville, Oregon,* May 18, 2023.
- Columbia West Engineering, Inc., *Infiltration Feasibility, Barber Street Housing Development, Wilsonville, Oregon,* June 20, 2023.

The City of Wilsonville has requested additional infiltration testing at the locations of proposed stormwater facilities to meet applicable stormwater design code requirements.

INFILTRATION TESTING

Infiltration potential of site soils was evaluated through in situ infiltration testing in boring B-1 (Columbia West, May 18, 2023) and in hand auger borings HA-1 through HA-7 conducted for this current supplemental investigation. The approximate locations of the boring and hand augers are shown on Figure 1. Exploration logs are presented in Appendix A.

Stand pipe, falling head infiltration testing was performed by embedding a hollow stem auger in boring B-1 and steel pipe in HA-1 through HA-7 in undisturbed native soil, filling the apparatus with water, and measuring time relative to changes in hydraulic head. Representative soil samples were collected from select test locations and submitted for laboratory analysis. Laboratory test reports are presented in Appendix B. Results of in situ infiltration testing are presented below in Table 1.

Table 1. Infiltration Test Results

Test Number	Location	Depth (feet BGS)	Passing No. 200	Depth to Groundwater (feet BGS)	Measured Infiltration Rate (in/hr)
IT-1.1	B-1	4.0	-	Not Encountered to 31.5	Negligible
IT-1.2	B-1	7.5	24	Not Encountered to 31.5	Negligible
HA-1.1	HA-1	2.0	64	Not Encountered to 2.0	Negligible
HA-2.1	HA-2	1.0	-	Not Encountered to 1.0	Negligible
HA-3.1	HA-3	0.75	31	Not Encountered to 0.75	Negligible
HA-4.1	HA-4	2.25	-	Not Encountered to 2.25	Negligible
HA-5.1	HA-5	1	-	Not Encountered to 1.0	Negligible
HA-6.1	HA-6	2.75	-	Not Encountered to 2.75	Negligible
HA-7.1	HA-7	2.25	-	Not Encountered to 2.25	Negligible

Based on the presence of fine-textured, very dense, low permeability site soils, infiltration is not a feasible option for stormwater management.

LIMITATIONS

We have prepared this report for use by Palindrome Wilsonville Limited Partnership and members of the design and construction team for the proposed project. The data and report can be used for design purposes, but our report, conclusions, and interpretations should not be construed as a warranty of the subsurface conditions and are not applicable to other sites.

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

If there are changes in the site grades or location, configuration, design loads, or type of construction, the conclusions and recommendations presented may not be applicable. If the design changes are made, we should be retained to review our conclusions and recommendations and to provide a written evaluation or modification.

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

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

◆◆◆

Sincerely,
Columbia West Engineering, Inc.

Jason F. Merritt, PE
Senior Project Engineer

94308
NOV 13, 2018
JASON F. MERRITT
EXPIRES: 12/31/24

Brett A. Shipton, PE, GE
Principal

JFM:BAS

Attachments

Document ID:Palindrome-3-01-1-120723-geol

Geotechnical ▪ Environmental ▪ Special Inspections
Columbia West
Engineering, Inc

PROJECT: PALINDROME-3-01-1
DATE: 12-01-23
DRAWN: EMU
CHECKED: JFM

EXPLORATION MAP
BARBER STREET HOUSING
DEVELOPMENT

NOTES:
1. SITE LOCATION: 9699 SW BARBER STREET IN WILSONVILLE, OREGON.
2. SITE CONSISTS OF TAX PARCEL 31W14B00703 TOTALING APPROXIMATELY 3.36 ACRES.
3. AERIAL PHOTO SOURCED FROM GOOGLE EARTH.
4. EXPLORATION LOCATIONS ARE APPROXIMATE AND NOT SURVEYED.
5. BORING BACKFILLED WITH BETONITE ON APRIL 28, 2023.
6. HAND AUGERS BACKFILLED WITH ONSITE SOIL ON NOVEMBER 30, 2023.

FIGURE
1

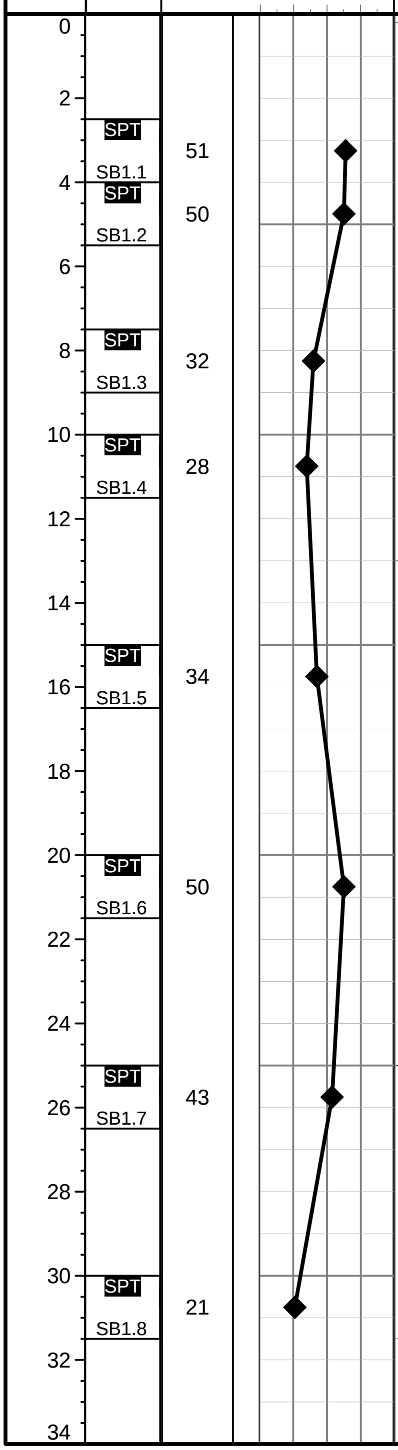
APPENDIX A
EXPLORATION LOGS

11917 NE 95TH Street, Vancouver, Washington 98682
Phone: 360-823-2900
www.columbiawestengineering.com

Geotechnical ■ Environmental ■ Special Inspections
Columbia West Engineering, Inc

SOIL BORING LOG

PROJECT NAME Barber Street Housing Development	CLIENT Palindrome Wilsonville Limited Partnership	PROJECT NO. 23122	BORING NO. SB-1
PROJECT LOCATION Wilsonville, Oregon	DRILLING CONTRACTOR Western States DRILL RIG CME75 Truck 9	TECHNICIAN EMU	PAGE NO. 1 of 1
BORING LOCATION See Figure 2	DRILLING METHOD HSA SAMPLING METHOD SPT	START DATE 04/28/23	START TIME 0820
REMARKS None	GROUNDWATER DEPTH Not encountered	FINISH DATE 04/28/23	FINISH TIME 1130



Field ID + Sample Type	SPT N-value (uncorrected)	USCS Soil Type	AASHTO Soil Type	LITHOLOGIC DESCRIPTION AND REMARKS	Infiltration (in/hr)	Moisture Content (%)	Passing No. 200 Sieve (%)	Liquid Limit	Plasticity Index
		GC		2-inch root zone.					
				Clayey GRAVEL with sand, brown and gray, damp, very dense, clay is nonplastic to low plasticity, fine- to medium-textued sand, fractured gravels.					
SPT SB1.1	51					17	27		
				Infiltration test run prior to SPT at 4 feet.					
SPT SB1.2	50				Neg.				
				Infiltration test run prior to SPT at 7.5 feet. Becomes dense at 7.5 feet.					
SPT SB1.3	32				Neg.	17	24	31	10
SPT SB1.4	28								
				Drill started to grind on gravel at 13 feet.					
		GM		Silty GRAVEL with sand, brown, very moist, dense, silt is nonplastic to low plasticity, fine- to coarse-textured sand, fine- to coarse-textued gravel. Driller indicated heaving at 15 feet.					
SPT SB1.5	34					17	12		
				Driller indicated auger was spinning on cobble or boulder at 19 feet.					
				Becomes very dense at 20 feet.					
SPT SB1.6	50								
		CL		Lean CLAY, blue and brown, moist, hard, low to medium plasticity, fine-textured sand.					
SPT SB1.7	43								
				Becomes blue-gray and brown and very stiff at 30 feet.					
SPT SB1.8	21					30	90		
				Boring completed at 31.5 feet bgs. Groundwater not observed on 4/28/23.					

Vancouver, Washington • Phone: 360-823-2900
Portland, Oregon • Phone: 971-384-1666
www.columbiawestengineering.com

Geotechnical ▪ Environmental ▪ Special Inspections
Columbia West
Engineering, Inc

HAND AUGER LOG

PROJECT NAME Barber Street Housing Development	CLIENT Palindrome Wilsonville Limited Partnership		PROJECT NO. Palindrome-3-01-1	BORING NO. HA-1
PROJECT LOCATION Willsonville, Oregon	CONTRACTOR N/A	EQUIPMENT Hand Auger	TECHNICIAN EMU	DATE 11/30/23
TEST PIT LOCATION See Figure 2	GROUNDWATER DEPTH Not Observed		START TIME 0845	FINISH TIME 1326

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 6-inches topsoil (2-inch root zone).					
				CL	▨	Brown, lean CLAY with sand, moist, stiff, low plasticity, fine sand. Rounded gravels at 2 feet. Infiltration test performed at 2 feet.					
	HA1.1					Hand auger terminated at 2 feet due to dense gravels. Groundwater not observed on 11/30/23.	35	64			
5											
10											

Vancouver, Washington • Phone: 360-823-2900
Portland, Oregon • Phone: 971-384-1666
www.columbiawestengineering.com

Geotechnical ▪ Environmental ▪ Special Inspections
Columbia West Engineering, Inc

HAND AUGER LOG

PROJECT NAME Barber Street Housing Development	CLIENT Palindrome Wilsonville Limited Partnership	PROJECT NO. Palindrome-3-01-1	BORING NO. HA-2
PROJECT LOCATION Willsonville, Oregon	CONTRACTOR N/A EQUIPMENT Hand Auger	TECHNICIAN EMU	DATE 11/30/23
TEST PIT LOCATION See Figure 2	GROUNDWATER DEPTH Not Observed	START TIME 0900	FINISH TIME 1035

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 9-inches topsoil (2-inch root zone).					
				CL	▨	Brown, lean CLAY with sand and gravel, moist, stiff, low plasticity, fine sand, fine to coarse gravel.					
						Infiltration test performed at 1 foot. Hand auger terminated at 1 feet due to dense gravels. Groundwater not observed on 11/30/23.					
5											
10											

Vancouver, Washington • Phone: 360-823-2900
Portland, Oregon • Phone: 971-384-1666
www.columbiawestengineering.com

Geotechnical ■ Environmental ■ Special Inspections
Columbia West
Engineering, Inc

HAND AUGER LOG

PROJECT NAME Barber Street Housing Development	CLIENT Palindrome Wilsonville Limited Partnership	PROJECT NO. Palindrome-3-01-1	BORING NO. HA-3
PROJECT LOCATION Willsonville, Oregon	CONTRACTOR N/A	EQUIPMENT Hand Auger	TECHNICIAN EMU	DATE 11/30/23
TEST PIT LOCATION See Figure 2	GROUNDWATER DEPTH Not Observed	START TIME 0930	FINISH TIME 1320

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	HA3.1					Approximately 8-inches topsoil (2-inch root zone). Infiltration test performed at 0.75 feet.	39	31			
						Hand auger terminated at 0.75 feet due to dense gravels. Groundwater not observed on 11/30/23.					
5											
10											

Vancouver, Washington • Phone: 360-823-2900
Portland, Oregon • Phone: 971-384-1666
www.columbiawestengineering.com

Geotechnical ■ Environmental ■ Special Inspections
Columbia West
Engineering, Inc

HAND AUGER LOG

PROJECT NAME: Barber Street Housing Development
CLIENT: Palindrome Wilsonville Limited Partnership
PROJECT NO.: Palindrome-3-01-1
BORING NO.: HA-4
PROJECT LOCATION: Willsonville, Oregon
CONTRACTOR: N/A
EQUIPMENT: Hand Auger
TECHNICIAN: EMU
DATE: 11/30/23
TEST PIT LOCATION: See Figure 2
GROUNDWATER DEPTH: Not Observed
START TIME: 0940
FINISH TIME: 1338

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 6-inches topsoil (2-inch root zone).					
				CL	▨	Brown, lean CLAY with sand and gravel, moist, stiff, low plasticity, fine sand, fine to coarse gravel. Infiltration test performed at 2.25 feet.					
						Hand auger terminated at 2.25 feet due to dense gravels. Groundwater not observed on 11/30/23.					
5											
10											

Vancouver, Washington • Phone: 360-823-2900
Portland, Oregon • Phone: 971-384-1666
www.columbiawestengineering.com

Geotechnical ■ Environmental ■ Special Inspections
Columbia West
Engineering, Inc

HAND AUGER LOG

PROJECT NAME Barber Street Housing Development	CLIENT Palindrome Wilsonville Limited Partnership		PROJECT NO. Palindrome-3-01-1	BORING NO. HA-5
PROJECT LOCATION Willsonville, Oregon	CONTRACTOR N/A	EQUIPMENT Hand Auger	TECHNICIAN EMU	DATE 11/30/23
TEST PIT LOCATION See Figure 2	GROUNDWATER DEPTH Not Observed		START TIME 1007	FINISH TIME 1245

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 12-inches topsoil (2-inch root zone).					
				CL		Brown, lean CLAY with sand and gravel, moist, stiff, low plasticity, fine sand, fine to coarse gravel.					
						Infiltration test performed at 1.25 feet. Hand auger terminated at 1.25 feet due to dense gravels. Groundwater not observed on 11/30/23.					
5											
10											

Vancouver, Washington • Phone: 360-823-2900
Portland, Oregon • Phone: 971-384-1666
www.columbiawestengineering.com

Geotechnical ▪ Environmental ▪ Special Inspections
Columbia West
Engineering, Inc

HAND AUGER LOG

PROJECT NAME: Barber Street Housing Development
CLIENT: Palindrome Wilsonville Limited Partnership
PROJECT NO.: Palindrome-3-01-1
BORING NO.: HA-6
PROJECT LOCATION: Willsonville, Oregon
CONTRACTOR: N/A
EQUIPMENT: Hand Auger
TECHNICIAN: EMU
DATE: 11/30/23
TEST PIT LOCATION: See Figure 2
GROUNDWATER DEPTH: Not Observed
START TIME: 1038
FINISH TIME: 1355

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 8-inches topsoil (2-inch root zone).					
				CL	▨	Brown, lean CLAY with sand, moist, stiff, low plasticity, fine sand. Infiltration test performed at 2.75 feet.					
						Hand auger terminated at 2.75 feet due to dense gravels. Groundwater not observed on 11/30/23.					
5											
10											

Vancouver, Washington • Phone: 360-823-2900
Portland, Oregon • Phone: 971-384-1666
www.columbiawestengineering.com

Geotechnical ■ Environmental ■ Special Inspections
Columbia West Engineering, Inc

HAND AUGER LOG

PROJECT NAME Barber Street Housing Development	CLIENT Palindrome Wilsonville Limited Partnership		PROJECT NO. Palindrome-3-01-1	BORING NO. HA-7
PROJECT LOCATION Willsonville, Oregon	CONTRACTOR N/A	EQUIPMENT Hand Auger	TECHNICIAN EMU	DATE 11/30/23
TEST PIT LOCATION See Figure 2	GROUNDWATER DEPTH Not Observed		START TIME 1105	FINISH TIME 1347

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 8-inches topsoil (2-inch root zone).					
				CL	▨	Brown, lean CLAY with sand, moist, stiff, low plasticity, fine sand. Fine to coarse gravels at 2 feet. Infiltration test performed at 2.25 feet.					
						Hand auger terminated at 2.25 feet due to dense gravels. Groundwater not observed on 11/30/23.					
5											
10											

APPENDIX B
LABORATORY TEST REPORTS

11917 NE 95th Street, Vancouver, Washington 98682
Phone: 360-823-2900
www.columbiawestengineering.com

Geotechnical ▪ Environmental ▪ Special Inspections
Columbia West
Engineering, Inc

PARTICLE-SIZE ANALYSIS REPORT

PROJECT
Barber Street Housing Development
Wilsonville, Oregon

CLIENT
Palindrome Communities, LLC
412 NW 5th Avenue
Portland, Oregon 97209

PROJECT NO. 23122	LAB ID S23-0534
REPORT DATE 05/12/23	FIELD ID B1.3
DATE SAMPLED 04/28/23	SAMPLED BY EMU

MATERIAL DATA

MATERIAL SAMPLED
brown Clayey GRAVEL with Sand

MATERIAL SOURCE
Boring B-01
depth = 7.5 feet

USCS SOIL TYPE
GC, Clayey Gravel with Sand

SPECIFICATIONS
none

AASHTO CLASSIFICATION
A-2-4(0)

LABORATORY TEST DATA

LABORATORY EQUIPMENT
Rainhart "Mary Ann" Sifter, air-dried prep, hand washed, composite sieve - #4 split

TEST PROCEDURE
ASTM D6913, Method A

ADDITIONAL DATA

initial dry mass (g) = 588.15
as-received moisture content = 17%
liquid limit = 31
plastic limit = 21
plasticity index = 10
fineness modulus = n/a

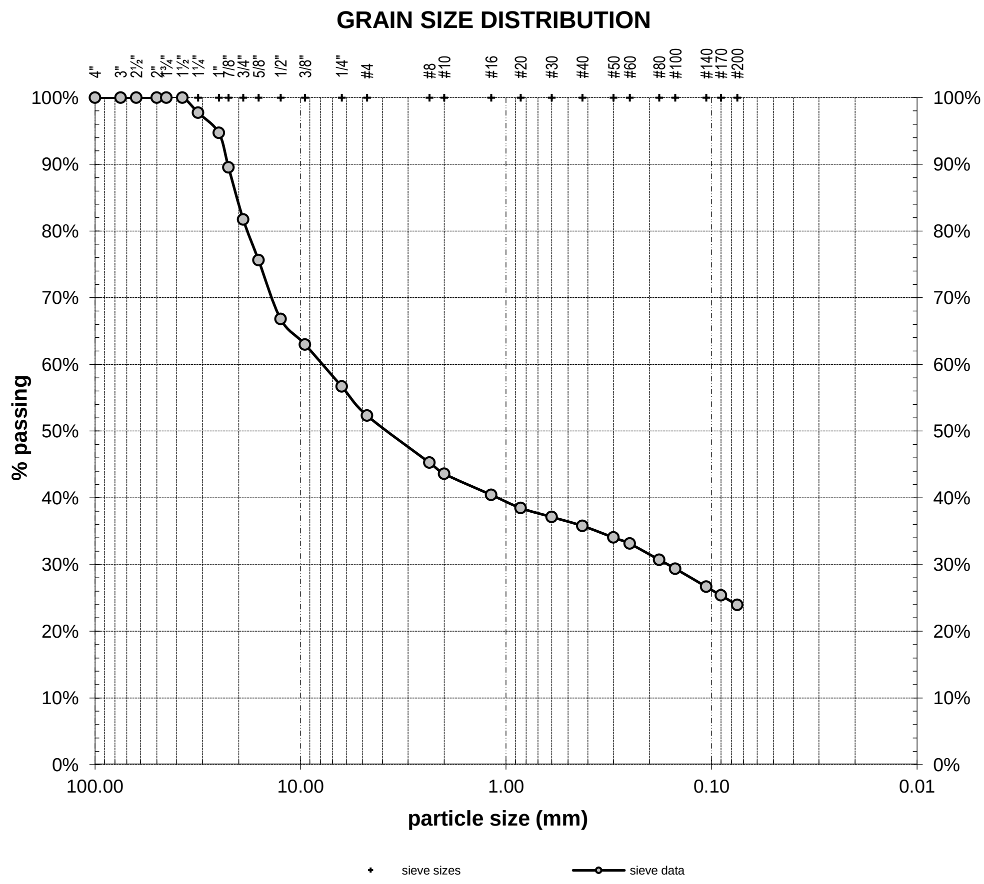
coefficient of curvature, C_C = n/a
coefficient of uniformity, C_U = n/a
effective size, $D_{(10)}$ = n/a
$D_{(30)}$ = 0.163 mm
$D_{(60)}$ = 7.828 mm

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

SIEVE DATA

% gravel = 47.7%
% sand = 28.4%
% silt and clay = 23.9%

	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		98%		
	1.00"	25.0	95%			
	7/8"	22.4		90%		
	3/4"	19.0	82%			
	5/8"	16.0		76%		
	1/2"	12.5	67%			
	3/8"	9.50	63%			
	1/4"	6.30		57%		
	#4	4.75	52%			
SAND	#8	2.36		45%		
	#10	2.00	44%			
	#16	1.18		40%		
	#20	0.850	38%			
	#30	0.600		37%		
	#40	0.425	36%			
	#50	0.300		34%		
	#60	0.250	33%			
	#80	0.180		31%		
	#100	0.150	29%			
	#140	0.106		27%		
	#170	0.090		25%		
	#200	0.075	24%			



DATE TESTED 05/10/23	TESTED BY KMS

COLUMBIA WEST ENGINEERING, INC. authorized signature

11917 NE 95th Street, Vancouver, Washington 98682
Phone: 360-823-2900
www.columbiawestengineering.com

Geotechnical ▪ Environmental ▪ Special Inspections
Columbia West
Engineering, Inc

ATTERBERG LIMITS REPORT

PROJECT	CLIENT	PROJECT NO.	LAB ID
Barber Street Housing Development Wilsonville, Oregon	Palindrome Communities, LLC 412 NW 5th Avenue Portland, Oregon 97209	23122	S23-0534
		REPORT DATE 05/12/23	FIELD ID B1.3
		DATE SAMPLED 04/28/23	SAMPLED BY EMU

MATERIAL DATA

MATERIAL SAMPLED	MATERIAL SOURCE	USCS SOIL TYPE
brown Clayey GRAVEL with Sand	Boring B-01 depth = 7.5 feet	GC, Clayey Gravel with Sand

LABORATORY TEST DATA

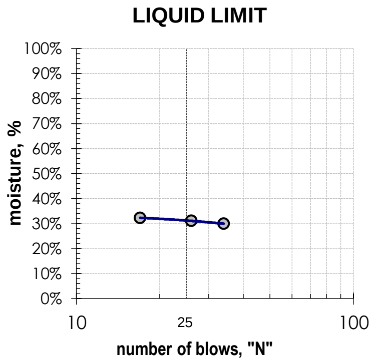
LABORATORY EQUIPMENT	TEST PROCEDURE
Liquid Limit Machine, Hand Rolled	ASTM D4318

ATTERBERG LIMITS

liquid limit = 31
plastic limit = 21
plasticity index = 10

LIQUID LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	32.18	33.02	32.90	
dry soil + pan weight, g =	29.60	30.16	30.00	
pan weight, g =	20.98	20.97	21.02	
N (blows) =	34	26	17	
moisture, % =	29.9 %	31.1 %	32.3 %	



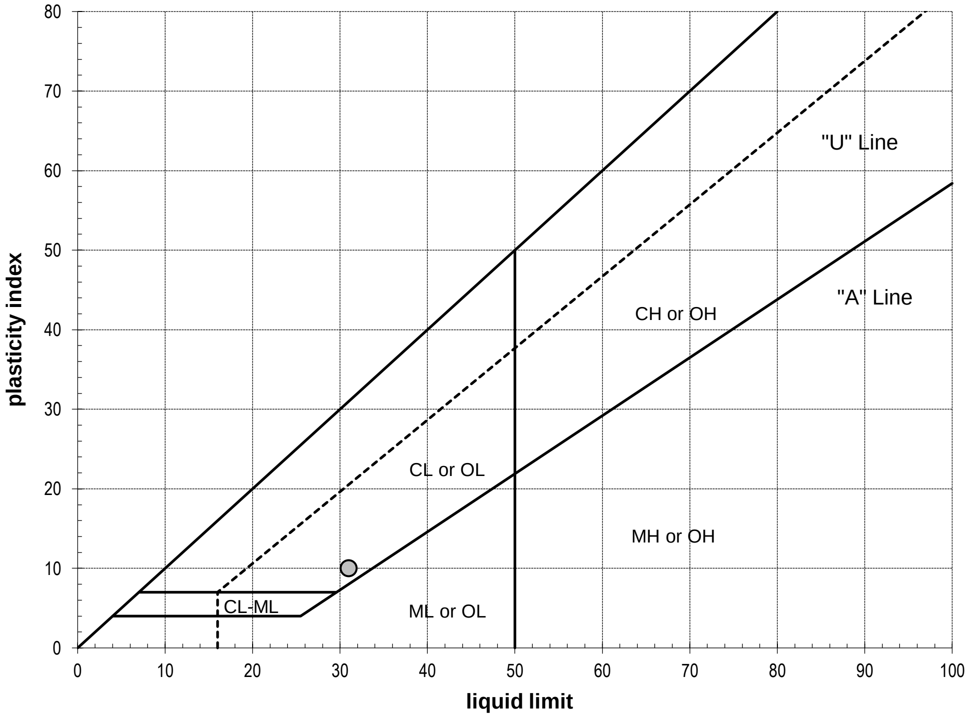
SHRINKAGE

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

PLASTIC LIMIT DETERMINATION

	❶	❷	❸	❹
wet soil + pan weight, g =	27.90	27.48		
dry soil + pan weight, g =	26.66	26.24		
pan weight, g =	20.87	20.42		
moisture, % =	21.4 %	21.3 %		

PLASTICITY CHART



ADDITIONAL DATA

% gravel = 47.7%
% sand = 28.4%
% silt and clay = 23.9%
% silt = n/a
% clay = n/a
moisture content = 17%

DATE TESTED	TESTED BY
05/10/23	MRS/KMS

COLUMBIA WEST ENGINEERING, INC. authorized signature

MOISTURE CONTENT, PERCENT PASSING NO. 200 SIEVE BY WASHING

PROJECT	CLIENT	PROJECT NO.	REPORT DATE
Barber Street Housing Development Supplemental Infiltration Testing 9699 SW Barber Street Wilsonville, Oregon 97070	Palindrome Wilsonville Limited Partnership 412 NW 5th Avenue Portland, Oregon 97209	Palindrome-3-01-1	12/05/23
		SAMPLED BY: EMU	PAGE: 1 of 1
		DATE SAMPLED: 11/30/23	

LABORATORY TEST DATA

TEST PROCEDURE

ASTM D2216 - Method A, ASTM D1140

LAB ID	CONTAINER MASS (g)	MOIST MASS + CONTAINER (g)	DRY MASS + CONTAINER (g)	AFTER WASH DRY MASS + CONTAINER (g)	FIELD ID	SAMPLE DEPTH (ft)	PERCENT MOISTURE CONTENT	PERCENT PASSING NO. 200 SIEVE
S23-1573	301.33	1,530.31	1,213.76	630.12	HA1.1	2	35%	64%
S23-1574	547.88	804.25	731.84	674.05	HA3.1	0.75	39%	31%

NOTES:

Sample weights received for Lab ID: S23-1573 and 1574 did not meet the minimum size requirements; entire sample used for analysis.

DATE TESTED	TESTED BY
12/04/23	MRS

COLUMBIA WEST ENGINEERING, INC. authorized signature

Appendix B

WES BMP Sizing Software Version 1.6.0.2, May 2018

WES BMP Sizing Report

Project Information

Project Name	Wilsonville TOD Apartments
Project Type	MultiFamily
Location	9749 SW Barber Street
Stormwater Management Area	52893
Project Applicant	Emerio Design
Jurisdiction	OutofDistrict

Drainage Management Area

Name	Area (sq-ft)	Pre-Project Cover	Post-Project Cover	DMA Soil Type	BMP
A	19,855	Grass	Roofs	C	Planters 1 & 2
B	3,766	Grass	Roofs	C	Planters 3, 4, & 5
C	4,697	Grass	Roofs	C	Planter 6
D	1,124	Grass	Roofs	C	Planter 7
E	3,668	Grass	ConventionalConcrete	C	Planter 8
F	1,305	Grass	ConventionalConcrete	C	Planter 9
G	1,096	Grass	ConventionalConcrete	C	Planter 10
H	1,928	Grass	ConventionalConcrete	C	Planter 11
I	694	Grass	ConventionalConcrete	C	ROW Planter A
J	1,366	Grass	ConventionalConcrete	C	ROW Planter B
K	2,457	Grass	ConventionalConcrete	C	ROW Planter C
L	3,272	Grass	ConventionalConcrete	C	ROW Planter D
M	7,659	Grass	ConventionalConcrete	C	ROW Planter E

LID Facility Sizing Details

LID ID	Design Criteria	BMP Type	Facility Soil Type	Minimum Area (sq-ft)	Planned Areas (sq-ft)	Orifice Diameter (in)
Planters 1 & 2	FlowControlAndTreatment	Stormwater Planter - Filtration	Lined	1,389.9	1,595.0	1.3
Planters 3, 4, & 5	FlowControlAndTreatment	Stormwater Planter - Filtration	Lined	263.6	333.0	0.6
Planter 6	FlowControlAndTreatment	Stormwater Planter - Filtration	Lined	328.8	357.0	0.7
Planter 7	FlowControlAndTreatment	Stormwater Planter - Filtration	Lined	78.7	295.0	0.3
Planter 8	FlowControlAndTreatment	Stormwater Planter - Filtration	Lined	256.8	259.0	0.6
Planter 9	FlowControlAndTreatment	Stormwater Planter - Filtration	Lined	91.4	96.0	0.3
Planter 10	FlowControlAndTreatment	Stormwater Planter - Filtration	Lined	76.7	96.0	0.3
Planter 11	FlowControlAndTreatment	Stormwater Planter - Filtration	Lined	135.0	146.0	0.4
ROW Planter A	FlowControlAndTreatment	Stormwater Planter - Filtration	Lined	48.6	96.0	0.2
ROW Planter B	FlowControlAndTreatment	Stormwater Planter - Filtration	Lined	95.6	96.0	0.4
ROW Planter C	FlowControlAndTreatment	Stormwater Planter - Filtration	Lined	172.0	174.0	0.5
ROW Planter D	FlowControlAndTreatment	Stormwater Planter - Filtration	Lined	229.0	229.0	0.5
ROW Planter E	FlowControlAndTreatment	Stormwater Planter - Filtration	Lined	536.1	539.0	0.8

Pond Sizing Details

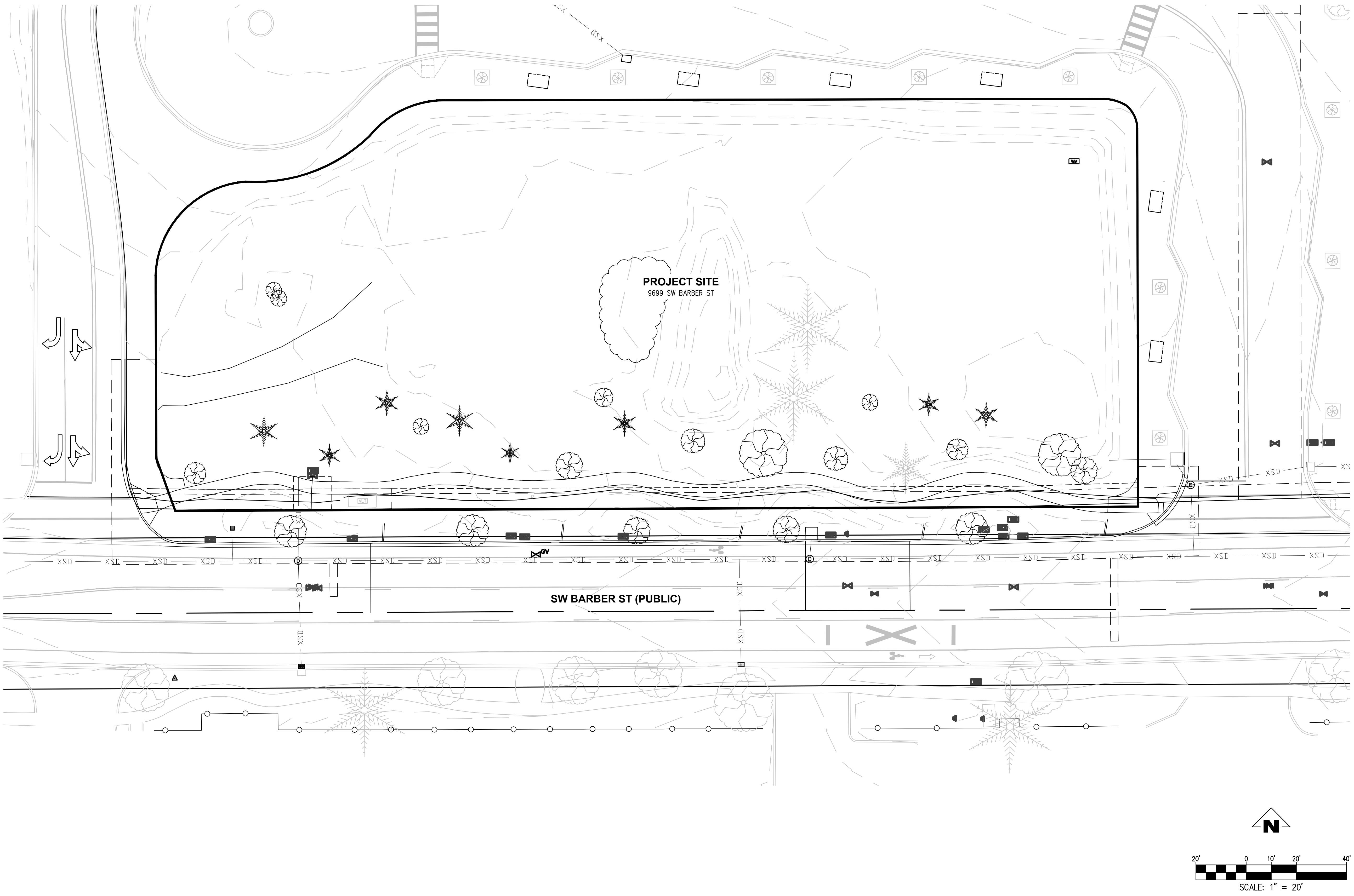
1. FCWQT = Flow control and water quality treatment, WQT = Water quality treatment only

2. Depth is measured from the bottom of the facility and includes the three feet of media (drain rock, separation layer and growing media).

3. Maximum volume of the facility. Includes the volume occupied by the media at the bottom of the facility.

4. Maximum water storage volume of the facility. Includes water storage in the three feet of soil media assuming a 40 percent porosity.

Appendix C



WILSONVILLE TOD APARTMENTS

TAX MAP 31W14B
TAX LOT 703
WILSONVILLE, OREGON

PRE-DEVELOPED SITE MAP

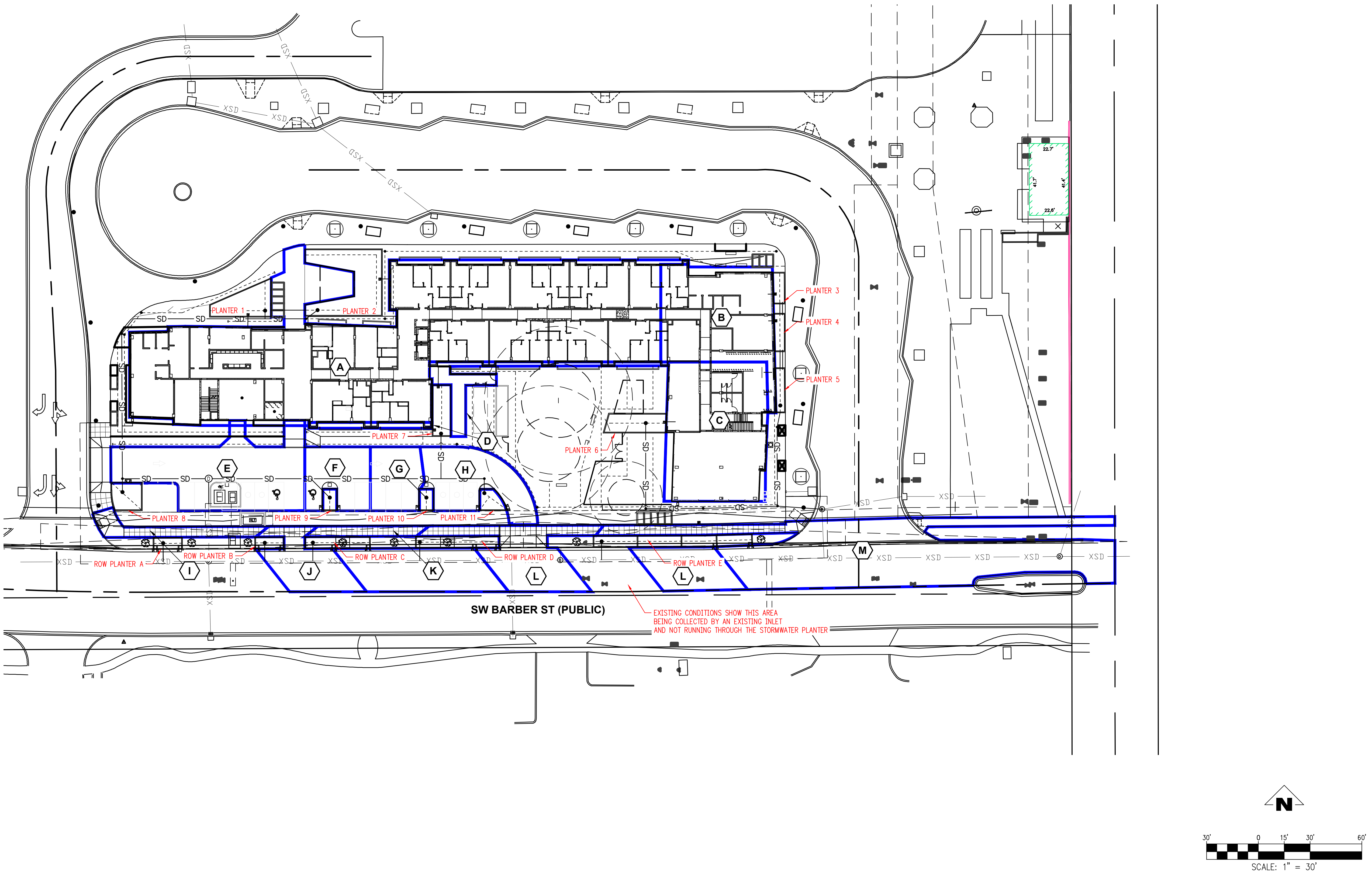
REVISIONS

NO.	DATE	DESCRIPTION

EMERIO
ENGINEERING • SURVEYING • DESIGN
6445 SW FALLBROOK PLACE, SUITE 100
BEAVERTON, OREGON 97008
TEL: (503) 746-8812
FAX: (503) 639-9592
www.emeriodesign.com

SHEET
1
OF
2

FILE:P:\0951-003 Wilsonville TOD Mixed Use Apartments\docs\civil\STORM\CAD\0951-003 Pre-Dev, Layout: EXTG & DEMO, Plot Date: 11/29/2023 9:30 AM, by: Steve Hansen



EXISTING CONDITIONS SHOW THIS AREA BEING COLLECTED BY AN EXISTING INLET AND NOT RUNNING THROUGH THE STORMWATER PLANTER

WILSONVILLE TOD APARTMENTS

TAX MAP 31W14B
TAX LOT 703
WILSONVILLE, OREGON

POST-DEVELOPED BASIN MAP

REVISIONS

NO.	DATE	DESCRIPTION

EMERIO
ENGINEERING • SURVEYING • DESIGN
6445 SW FALLBROOK PLACE, SUITE 100
BEAVERTON, OREGON 97008
TEL: (503) 746-8812
FAX: (503) 639-9592
www.emeriodesign.com

SHEET
2
OF
2

FILE:P:\0951-003 Wilsonville TOD Mixed Use Apartments\docs\civ\STORM\CAD\0951-003 Basin, Layout: Layout1, Plot Date: 11/29/2023 9:31 AM, by: Steve Hansen