EXHIBIT B

Drainage Report

Milwaukie Bluffs Rezone

5515 SE Milwaukie Ave, Portland OR 97202

Prepared for

CCG Management LLC

8555 SW Apple Way, Suite 110, Portland OR 97225

09/01/2021

EXHIBIT B

Prepared for	CCG Management LLC
Project Name	Milwaukie Bluffs Rezone
Address	5515 SE Milwaukie Ave, Portland OR 97202
Date	09/01/2021

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DECEMBER 31, 2001
JEFFREY R. SHOEMAKER
EXPIRES: 06/30/2022

Name	Title	Date	Revision	Reviewer
Korey Derrick	Project Manager	09/01/2021	0	Jeff Shoemaker

Executive Summary

The Milwaukie Bluffs Zoning Study site is located between SE Ellis St and SE Insley St on the west and east side of SE Milwaukie Avenue in Portland, Oregon (Figure 1-1).

The change in zoning requires the applicant to demonstrate that the proposed stormwater disposal systems are or will be made acceptable to the Bureau of Environmental Services. In this case, the applicant is not proposing specific development and therefore, this report addresses the feasibility of providing an acceptable stormwater system for all reasonable development permitted in the proposed zones. Final stormwater plans/layout and report will be re-analyzed prior to the construction phase of the project to confirm the layout and viability.

The purpose of this report is to describe the stormwater management techniques that could be used for stormwater feasibility for future projects associated with Milwaukie Bluffs. The stormwater concept management designs follow the standards and regulations developed by the City of Portland. These regulations are identified in the City of Portland's Stormwater Management Manual, Bureau of Environmental Services, revised December 2020.

Stormwater Management

The City of Portland has developed a stormwater discharge hierarchy that includes four stormwater disposal categories. The highest technically feasible category must be used prior to moving to a lower category. Hierarchy Categories 1 and 2 require full onsite infiltration using surface vegetated facilities and/or drywells.

The project would be designed under Hierarchy Category 2. Due to site area and topographic constraints future development on the site is very unlikely to include an above grade parking lot with over 50 stalls generating more than 1,000 trips per day. However, this report includes a conservative assumption that 25% of the site area would be developed as surface parking area which requires stormwater planters for treatment prior to discharging to a drywell. All non-parking areas (roof and flatwork) can discharge directly to drywells without pollution reduction from a vegetated facility.

Stormwater from a proposed development would be routed to drywells along the eastern property line for full infiltration. Final drywell placement would be heavily coordinated with the geotechnical engineer and structural engineer to ensure slope stability along the bluff and no adverse impacts to proposed building footings/foundations. Stormwater from sidewalks would be captured in sumped, trapped catch basins for pretreatment before discharging to the drywell. Surface parking areas will be treated for pollution reduction using stormwater planter facilities prior to draining to a drywell. The drywells are designed using XPSWMM and was sized such that the system can fully infiltrate the 10-yr storm without surcharge, and draw-down within 30 hours. A geotechnical analysis of the soils indicate drywells would need to be at least 8' down to ensure no infiltration in fill material and no deeper than 30' to maintain 5' of separation between seasonally high groundwater.

I hereby certify that this Stormwater Management Report for the Milwaukie Bluffs Zoning Study has been prepared by me or under my supervision and meets minimum standards of the City of Portland and normal standards of engineering practice. I hereby acknowledge and agree that the jurisdiction does not and will not assume liability for the sufficiency, suitability, or performance of drainage facilities designed by me.

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1 Project Overview

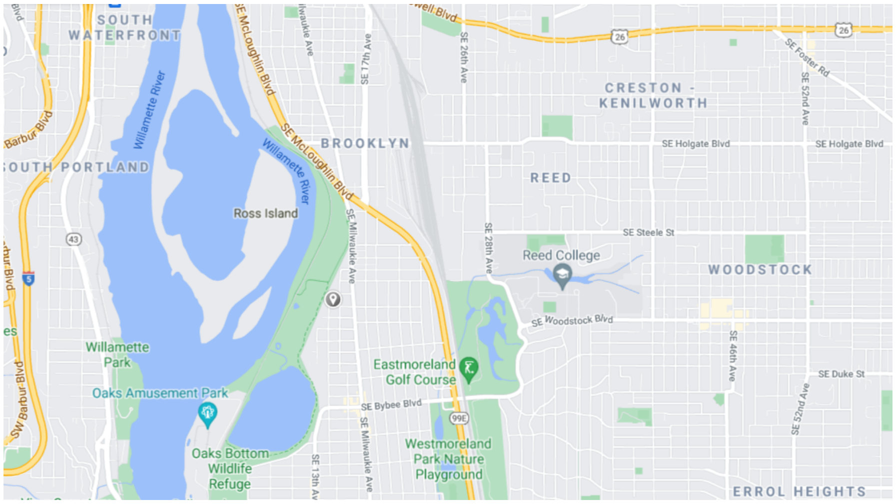
1.1 Project Overview

The proposed Milwaukie Bluffs Rezoning study will show stormwater feasibility for the proposed project. The total on-site area west of SE Milwaukie Ave is approximately 1.72 acres while the onsite area east of SE Milwaukie Ave is approximately 0.23 acres. The area west of SE Milwaukie Ave has approximately 0.92 acres of developable land due a river environmental overlay located on the property.

1.2 Location

The project site is located between SE Ellis St and SE Insley St west of SE Milwaukie Avenue in Portland, Oregon (Figure 1-1).

Figure 1-1 - Vicinity Map



1.3 Stormwater Hierarchy

The disposal hierarchy found in the City of Portland *Stormwater Management Manual* was used to evaluate stormwater management options at the site. Per Section 1.3.1 – Infiltration and Discharge Hierarchy:

> *"Stormwater must be infiltrated onsite to the maximum extent feasible, before any flows are discharged offsite... The appropriate use of infiltration depends on a number of factors, including soil type, soil conditions, slopes, and depth to groundwater."*

Category 1: Requires total onsite infiltration with vegetated infiltration facilities.

Category 2: Requires total onsite infiltration with vegetated facilities that overflow to a subsurface infiltration facility.

The project will be designed under Hierarchy Category 2. For purposes of determining the feasibility of a stormwater water system that satisfies BES requirements under the proposed zoning, potential future development is conservatively assumed to have 25% parking area with over 50 stalls and receive more than 1,000 trips per day. Under that scenario vegetated facilities would be proposed for pollution reduction prior to the drywells. All roof and sidewalk areas would be routed to drywells without pollution reduction upstream.

Category 3: Requires onsite detention with vegetated facilities that overflow to a drainageway, river, or storm pipe.

Category 4: Requires onsite detention with vegetated facilities that overflow to the combined sewer system.

2 Existing Conditions

2.1 Topography

The project site is developed with existing commercial buildings and apartments. The site is relatively flat and slightly slopes southward. Elevations on site range from 85 to 90 ft.

2.2 Climate

The site is located in Portland, Oregon approximately 50 miles inland from the Pacific Ocean. There is a gradual change in seasons with defined seasonal characteristics. Average daily temperatures range from 44°F to 82°F. Record temperatures recorded for this region of the state are -18°F and 108°F. Average annual rainfall recorded in this area is 41 inches.

2.3 Site Geology

The underlying soil type on the existing site as classified by the United States Department of Agriculture Soil Survey of Multnomah County, Oregon as Urban Land, with 3 to 8 percent slopes and Haploxerolls, with steep slopes. (See Appendix A: USGS Soils Map - Multnomah County) A hydrologic soil group is not assigned to this soil type.

The geotechnical engineer reported an unfactored, field measured infiltration rate of 8.2 in/hr at approximately 10' below ground surface (See Technical Appendix: Geotechnical Report).

2.4 Coefficients

The major factors for determining the CN values are hydrologic soil group, cover type, treatment, hydrologic condition and antecedent runoff condition. The curve number represents runoff potential from the soil. A pervious curve number of 95 was used, which represents commercial and business in urban areas – with D soils (See Technical Appendix: Table A.2. Curve Numbers for Urban Areas).

2.5 Time of Concentration

The time of concentration (TC) as described in NEH-4 Chapter 15 is defined in two ways; the time for runoff to travel from the furthermost point of the watershed to the point in question, and the time from the end of excess rainfall to the point of inflection on the trailing limb of the unit hydrograph. Time of concentration can be estimated from several formulas. A time of concentration value of 5 minutes was used for existing conditions.

2.6 Hydrology

Runoff from the existing site generally infiltrates through the gravel area, or sheet flows into the adjacent landscaping. Pollution reduction and flow control are not present on the existing site.

2.7 Basin Area

Table 2-1 lists the basin areas in existing conditions. The existing basins were assumed to be 95% impervious (See Technical Appendix: Figure 1 – Existing Conditions). The steep bluff portion or anything west of the river environmental overall was not included for ease in calculations.

Table 2-1 Existing Basin Areas

Basin	Impervious Area (ac)	Pervious Area (ac)	Total Area (ac)
Existing Basin 1	0.874	0.046	0.92
Existing Basin 2	0.219	0.011	0.23

3 Proposed Conditions

3.1 Coefficients

A pervious curve number of 95 was used, which represents Commercial and Business in Urban Districts – with D soils (See Technical Appendix: Table A.2. Curve Numbers for Urban Areas).

3.2 Time of Concentration

The time of concentration value used for proposed conditions is 5 minutes.

3.3 Hydrology

As discussed above, for purposes of this report it is assumed that 25% of the developable site area will be parking area that received over 1,000 trips and more than 50 parking stalls. Under that scenario stormwater from the parking areas would be routed to stormwater planter facilities for pollution reduction prior to discharging into a drywell to ensure flexibility for future design. Roof areas would be directly routed into the drywells for full infiltration and sidewalks and plaza area would be pre-treated in sumped catchbasins prior to discharging into drywells.

3.4 Basin Area

Table 3-1 lists the basin areas under proposed conditions. The proposed basins are assumed to be 100% impervious for conservative purposes (See Technical Appendix: Figure 2 – Proposed Conditions).

Table 3-1 Proposed Basin Areas

Basin	Impervious Area (ac)	Pervious Area (ac)	Total Area (ac)
Proposed Basin 1	0.92	0.00	0.92
Proposed Basin 2	0.23	0.00	0.23

4 Hydrologic and Hydraulic Analysis

4.1 Design Guidelines

The site is located within the City of Portland. The analysis and design criteria used for stormwater management will follow the City of Portland *Stormwater Management Manual* - August 2016.

4.2 Hydrologic Method

The Santa Barbara Urban Hydrograph (SBUH) method was used for this analysis. The SBUH method is based on the curve number (CN) approach and uses the Natural Resource Conservation Service's (NRCS) equations for computing soil absorption and precipitation excess. The SBUH method converts the incremental runoff depths into instantaneous hydrographs, which are then routed through an imaginary reservoir with a time delay equal to the basin time of concentration.

The XPSWMM software version 18.1 was used for the hydrology and hydraulics analysis. The runoff function of XPSWMM generates surface and subsurface runoff based on design or measured rainfall conditions, land use and topography. The XPSWMM software is based on the public EPA SWMM program and is an approved method of analysis by City of Portland.

4.3 Design Storm

The rainfall distribution used within the City of Portland's jurisdiction is the design storm of 24-hour duration based on the standard NRCS Type 1A rainfall distribution. Table 4-1 shows total precipitation depths for different storm events which were used for the type 1A 24-hour rainfall distribution in XPSWMM. A typical NRCS Type 1A 24-hour rainfall distribution is shown in Figure 4-1.

Table 4-1 Precipitation Depth

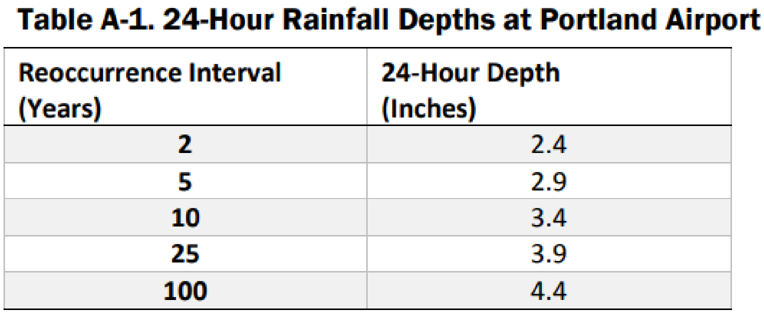
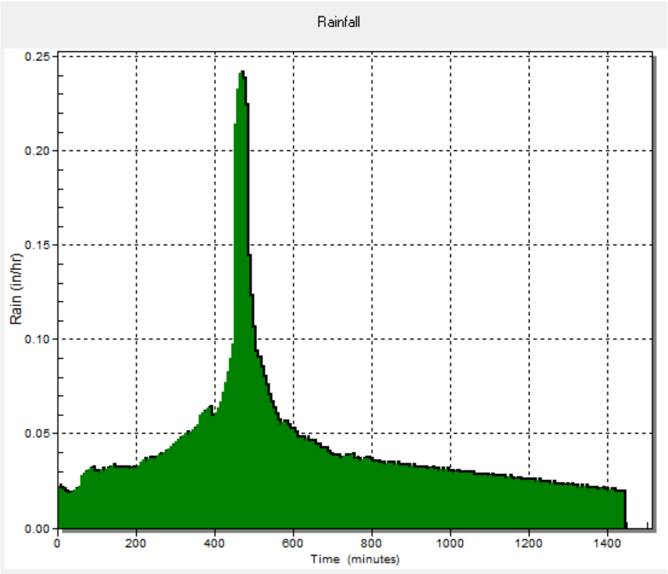


Table A-1. 24-Hour Rainfall Depths at Portland Airport

Reoccurrence Interval (Years)	24-Hour Depth (Inches)
2	2.4
5	2.9
10	3.4
25	3.9
100	4.4

Figure 4-1 - Type 1A Rainfall Distribution



5 Conveyance Analysis

5.1 Design Guidelines

The analysis and design criteria used for stormwater management described in this section follows the City of Portland *Sewer and Drainage Facilities Design Manual*, revised in March 2020. The manual requires storm drainage facilities be designed to pass the 10-year storm event without surcharging and a means to pass the 25-year storm event without damage to property.

Final site design for the site would be to ensure an emergency overflow pathway.

6 Water Quality

6.1 Design Guidelines

The City of Portland's *Stormwater Management Manual* was used for the onsite stormwater quality design. The City of Portland requires 70 percent removal of total suspended solids for 90 percent of the average annual runoff.

6.2 Pollution Reduction

Proposed density of the site could contribute to over 1,000 trips per day. Due to site area and topographic constraints future development on the site is very unlikely to include an above grade parking lot with over 50 stalls generating more than 1,000 trips per day. However, in order to account for future development that could be developed on the site under the proposed zones, we have assumed 25% of the site area is a parking lot which requires stormwater planters for treatment prior to discharging to a drywell. Pollution reduction would be achieved through vegetated planters.

6.3 Vegetated Planter Facilities

Vegetated planters are landscaped depressions used to collect and hold stormwater runoff, allowing pollutants to settle and filter out as water passes through the soil media. Planter design incorporates the following criteria:

- Freeboard: 2 inches
- Storage Depth: 6 inches minimum
- Growing Medium Depth: 18 inches
- Rock Storage Depth: 12 inches
- Impermeable liner due to proximity to steep slopes.
- Basins with 3:1 side slopes

The online BES PAC calculator was used to size the facilities. The facilities were sized for pollution reduction only. Table 6-1 summarizes the facility drainage area and planter area size used for the pollution reduction storm (See Technical Appendix: PAC Report). For locations of each planter facility, see Technical Appendix: Figure 2 – Proposed Conditions.

Table 6-1 Filtration Planter Summary

Catchment Name	Impervious Area (sf)	Planter Bottom Area (sf)	Facility Name
Basin 1 Parking	10,020	130	B1
Basin 2 Parking	2,505	25	B2

6.4 Drywells

Stormwater from the sidewalks, parking after pollution reduction, and roof areas would be routed to drywells along the east property line for infiltration. Basin 1 would utilize five drywells with a depth of 30’ (20’ of perforations) while Basin 2 would utilize two drywells with a depth of 20’ (10’ of perforations). Stormwater from parking areas would be captured and treated in vegetated facilities while sidewalk areas would be captured in sumped, trapped catch basins for pretreatment before discharging to the drywell.

6.5 Drywell Facilities

The project will consist of two basin areas. Basin area 1 assumes 40,000 SF of impervious area, while Basin area 2 assumes 10,000 SF of impervious area. Basin 1 would utilize five stormwater UIC drywell facilities while basin 2 would utilize two stormwater UIC drywell facilities, which would treat and infiltrate the stormwater runoff per the *Stormwater Management Manual* requirements. Below is a summary of the UIC facilities:

Basin 1

> 5 - 30’ deep drywells with perforations in the bottom 20’
> 48” diameter drywell with 12” of stone surrounding
> Measured infiltration rate = 8.2 in/hr
> Design infiltration rate = 4.1 in/hr

Basin 2

- 2 - 20’ deep drywells with perforations in the bottom 10’
- 48” diameter drywell with 12” of stone surrounding
- Measured infiltration rate = 8.2 in/hr
- Design infiltration rate = 4.1 in/hr

7 Water Quantity

7.1 Design Guidelines

Water quantity facilities were designed in accordance with the City of Portland *Stormwater Management Manual* - August 2016. Per Section 1.3.3 – Infiltration Requirements (Categories 1 and 2):

> *“Onsite infiltration is required. Facilities must infiltrate Portland’s 10-year design storm. When full, the facility drawdown time must not exceed 30 hours.”*

7.2 Drywells

Per the City of Portland *Stormwater Management Manual*, the performance approach was used to size the private drywell.

The drywells have been modeled using XPSWMM and sized per the requirements listed under Hierarchy 2. The five drywells on the west basin are assumed to be 30’ deep with perforated sections in the bottom 20’. Two drywells on the east basin are assumed to be 20’ deep with perforated sections in the bottom 10’. The infiltration rate used for the drywell analysis was 4.2 in/hr. This was found by taking the observed infiltration rate at the site of 8.2 in/hr and applying a safety factor of 2. Groundwater was not encountered during the infiltration testing. USGS mapping indicates groundwater is at 36-40 feet bgs (See Technical Appendix: Geotechnical Report). This depth meets the 5’ separation from the ordinary high groundwater mark requirement for drywells. Final drywell placement on Basin 1 will be heavily coordinated with the geotechnical engineer and structural engineer to ensure slope stability along the bluff and no adverse impacts to proposed building footings/foundations.

Basin 1 and Basin 2 systems are able to fully infiltrate the 10-yr design storm, and the facility drawdown time does not exceed 30 hours. In the event the drywell overtops during a larger storm, an overland flow path out to the street would be designed for in the final site design for the project.

Table 7-1 below summarizes the drywell facility and performance during the 10-yr, 25-yr, and 100-yr storm events. Sizing was completed in XPSWMM using the SBUH method. A rating curve was created for the flow leaving the drywell based on the depth of water within the system (See Technical Appendix: Drywell Rating Curve). The draw-down time for the drywell is approximately 18 hours once full, and less than two hours after the storm ends (See Technical Appendix: XPSWMM Results).

Table 7-1 Drywell Sizing

Facility ID	System Size	Rim Elevation (ft)*	Invert Elevation (ft)*	Max WSE during 10yr Event (ft)*
Basin 1 – 5 Drywells	48" MH with 12" rock surrounding	120	90	118.40
Basin 2 – 2 Drywells	48” MH with 12” rock surrounding	120	100	117.54

*Elevations are assumed at 120 as no survey or grading design has been completed for a site design layout.

8 Operation & Maintenance

An Operation and Maintenance (O&M) Plan would be provided during the site design phase of this project.

9 Summary

Stormwater pollution reduction and flow control for the future project would be provided using trapped catch basins and drywells.

Stormwater in parking areas that receive over a 1,000 trips per day and 50 parking stalls or more would be captured by vegetated facilities prior to discharge into a drywell. Any sidewalk or plaza area would be captured in sumped, trapped catch basins for pretreatment before discharging to the drywells. Additionally, roof runoff can be discharged directly into drywells without a vegetated facility for pollution reduction.

The drywells are modeled to fully infiltrate the 10-yr storm event and draw-down within 30 hours.

The proposed stormwater management system meets the pollution reduction and flow control requirements of the City of Portland *Stormwater Management Manual* – August 2016.

Technical Appendix

- Figure 1 – Existing Conditions
- Figure 2 – Proposed Conditions
- Pac Report

- USGS Soils Map - Multnomah County
- Table A.2 – Curve Numbers for Urban Areas

- XPSWMM Results – Drywell Stage Graphs

- Geotechnical Report – GeoDesign, Inc – May 2020

References

1. *USDA Soil Survey of Multnomah County, Oregon Area*
2. City of Portland *Stormwater Management Manual* – August 2016
3. City of Portland Sewer and *Drainage Facilities Design Manual* – March 2020

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EXISTING CONDITIONS
Basin Map

Existing Conditions

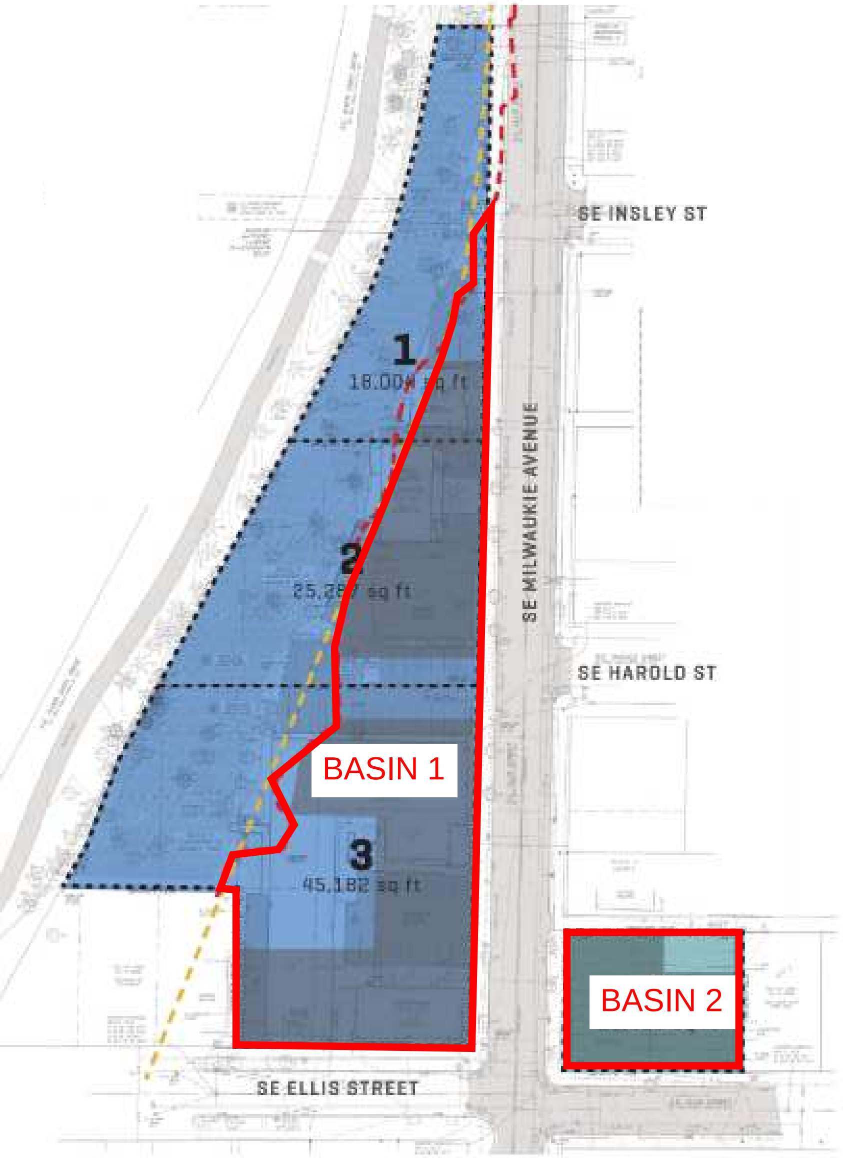
Basin 1
Total Site Area: 1.72 Acres
Area West of River Environmental line: 0.80 Acres
Area Remaining: 0.92 Acres*

Assuming 95% Impervious
Pervious Area: 0.046 Acres
Impervious Area: 0.874 Acres

Basin 2
Total Site Area: 0.23 Acres

Assuming 95% Impervious
Pervious Area: 0.011 Acres
Impervious Area: 0.219 Acres

* Area on Bluff steep slope and in environmental zone has been removed for conservative purposes.



PRELIMINARY DRAINGE REPORT EXHIBIT - EX1

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PROPOSED CONDITIONS
Basin Map

Proposed Conditions

Basin 1
Total Site Area: 1.72 Acres
Area West of River Environmental line: 0.80 Acres
Area Remaining: 0.92 Acres*

Assuming 100% Impervious**
Pervious Area: 0.00 Acres
Impervious Area: 0.92 Acres

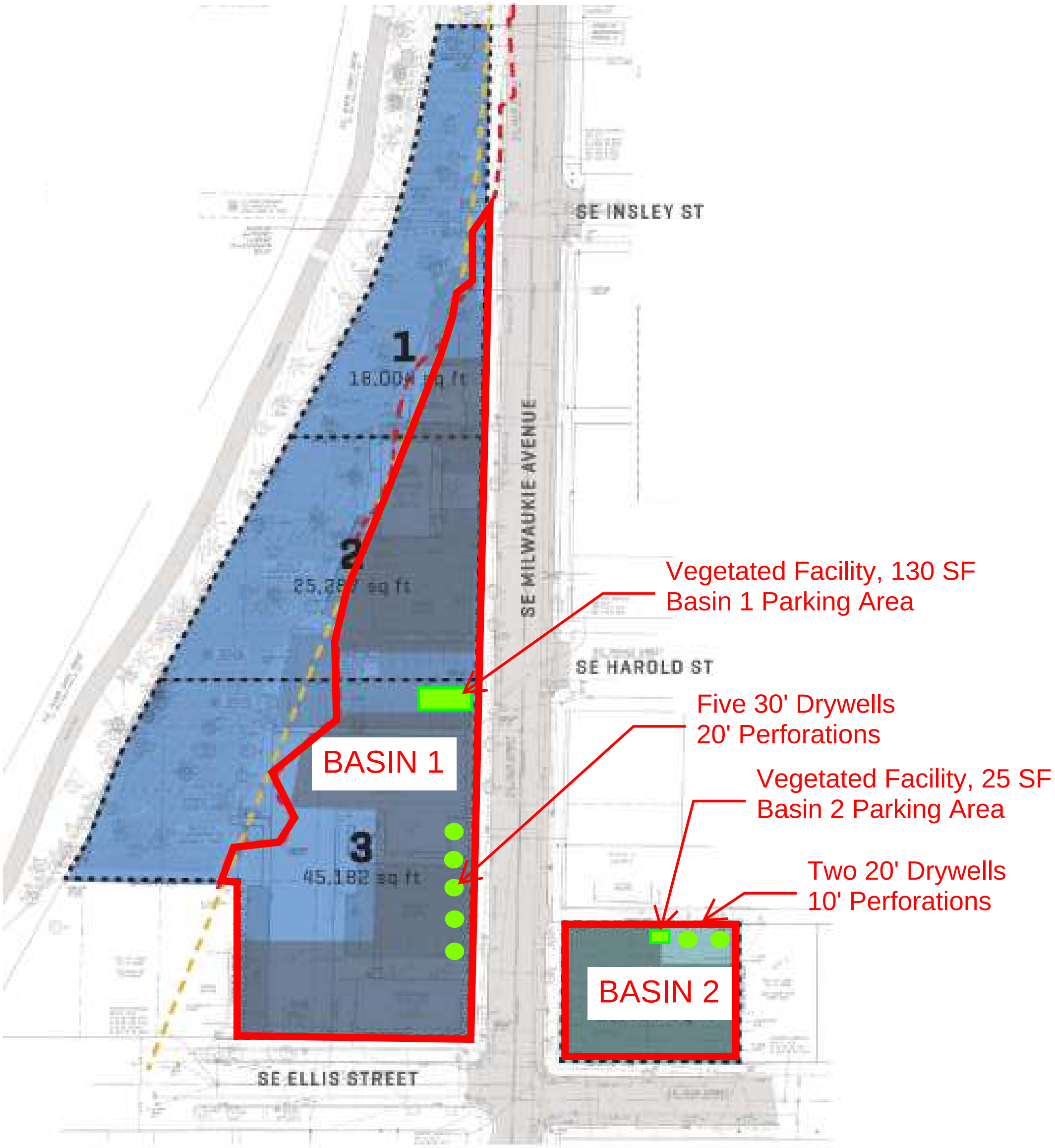
Parking Area Assumed at 25% of Impervious Area
Vegetated Facility Treatment Area: 0.23 Acres (10,020 SF)

Basin 2
Total Site Area: 0.23 Acres

Assuming 100% Impervious**
Pervious Area: 0.00 Acres
Impervious Area: 0.23 Acres

Parking Area Assumed at 25% of Impervious Area
Vegetated Facility Treatment Area: 0.057 Acres (2,505 SF)

* Area on Bluff steep slope and in environmental zone has been removed for conservative purposes.
**Assumed 100% impervious area for conservative purposes.



Vegetated Facility, 130 SF
Basin 1 Parking Area

Five 30' Drywells
20' Perforations

Vegetated Facility, 25 SF
Basin 2 Parking Area

Two 20' Drywells
10' Perforations

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PAC Report

Project Name Milwaukie Bluffs	Permit No. N/A	Created 9/20/21 3:33 PM
Project Address 5515 SE Milwaukie Ave Portland, OR 97202	Designer Korey Derrick	Last Modified 9/20/21 4:16 PM
	Company DOWL	Report Generated 9/20/21 4:16 PM

Project Summary

Milwaukie Bluffs Zoning Study

Catchment Name	Impervious Area (sq ft)	Native Soil Design Infiltration Rate	Hierarchy Category	Facility Type	Facility Config	Facility Size (sq ft)	Facility Sizing Ratio	PR Results	Flow Control Results
Basin 1 Parking	10020	0.00	2	Basin	D	130	2.4%	Pass	Not Used
Basin 2 Parking	2505	0.00	2	Basin	D	25	3.1%	Pass	Not Used

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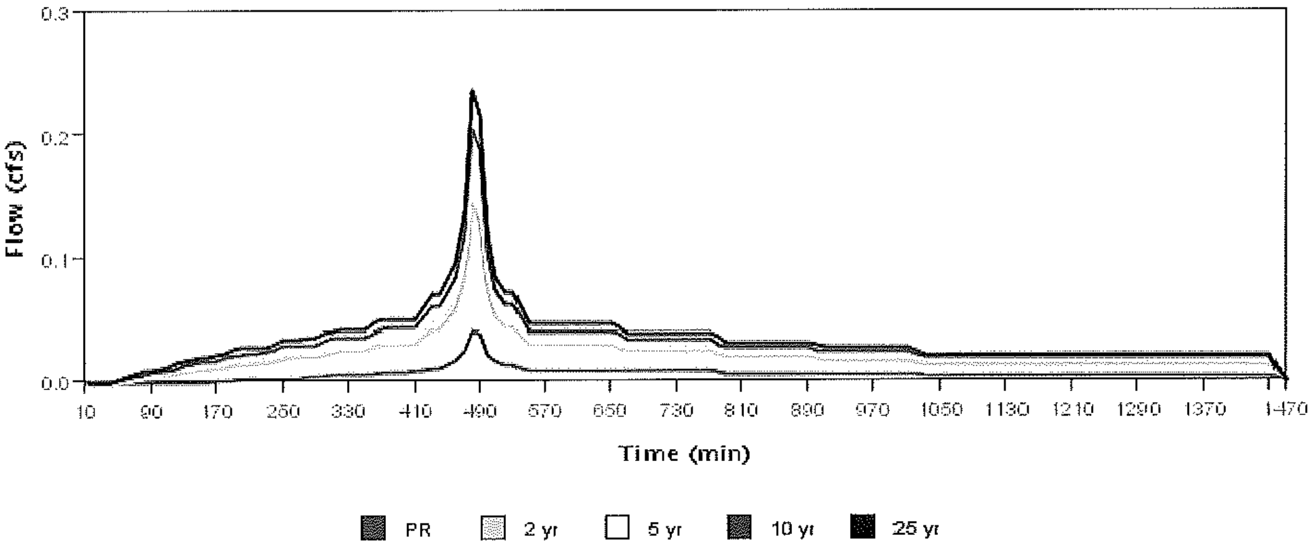
Catchment Basin 1 Parking

Site Soils & Infiltration Testing Data	Infiltration Testing Procedure	**Open Pit Falling Head**
	Native Soil Infiltration Rate (I_{test})	**0.00** ⚠
Correction Factor	CF_{test}	**2**
Design Infiltration Rates	Native Soil (I_{dsgn})	**0.00 in/hr** ⚠
	Imported Growing Medium	**2.00 in/hr**
Catchment Information	Hierarchy Category	**2**
	Hierarchy Description	**On-site infiltration through use of approved UIC facility**
	Pollution Reduction Requirement	**Pass**
	10-year Storm Requirement	**Pass or if Fail, disposal through separate approved UIC**
	Flow Control Requirement	**Pass or if Fail, disposal through separate approved UIC**
	Impervious Area	**10020 sq ft** **0.230 acre**
	Time of Concentration (Tc)	**5**
	Post-Development Curve Number (CN_{post})	**98**

⚠ Indicates value is outside of recommended range

SBUH Results



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	Peak Rate (cfs)	Volume (cf)
PR	0.041	523.573
2 yr	0.141	1813.078
5 yr	0.173	2228.363
10 yr	0.204	2644.27
25 yr	0.235	3060.569

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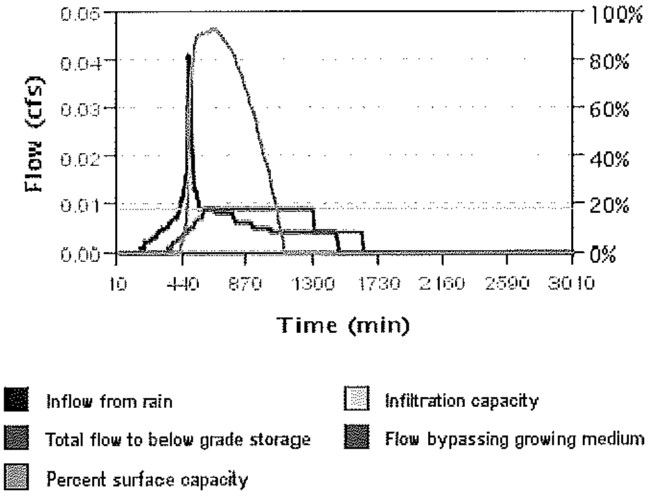
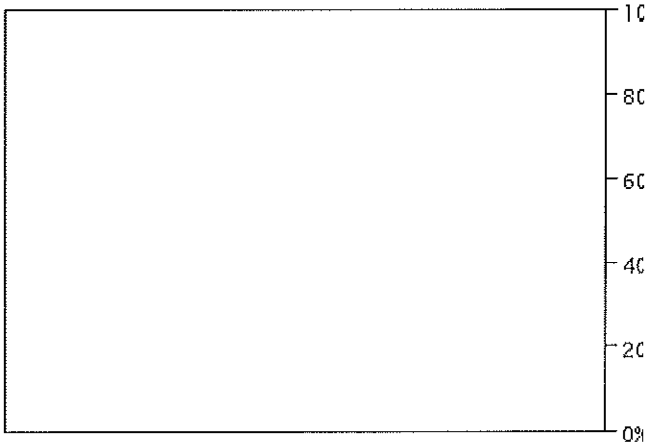
Facility Basin 1 Parking

Facility Details	Facility Type	**Basin**
	Facility Configuration	**D: Lined Facility with RS and Ud**
	Facility Shape	**Rectangle**
	Above Grade Storage Data	
	Bottom Area	**130 sq ft**
	Bottom Width	**8.00 ft**
	Side Slope	**3.0:1**
	Storage Depth 1	**6.0 in**
	Growing Medium Depth	**18 in**
	Freeboard Depth	**2.00 in**
	Surface Capacity at Depth 1	**84.4 cu ft**
	Design Infiltration Rate for Native Soil	**0.000 in/hr**
	Infiltration Capacity	**0.009 cfs**
Facility Facts	Total Facility Area Including Freeboard	**239.57 sq ft**
	Sizing Ratio	**2.4%**
Pollution Reduction Results	Pollution Reduction Score	**Pass**
	Overflow Volume	**522.610 cf**
	Surface Capacity Used	**94%**
10 Year Results	10 Year Score	**Fail**
	Overflow Volume	**2646.857 cf**
	Surface Capacity Used	**100%**


Pollution Reduction Event Surface Facility Modeling


Pollution Reduction Event Below Grade Modeling

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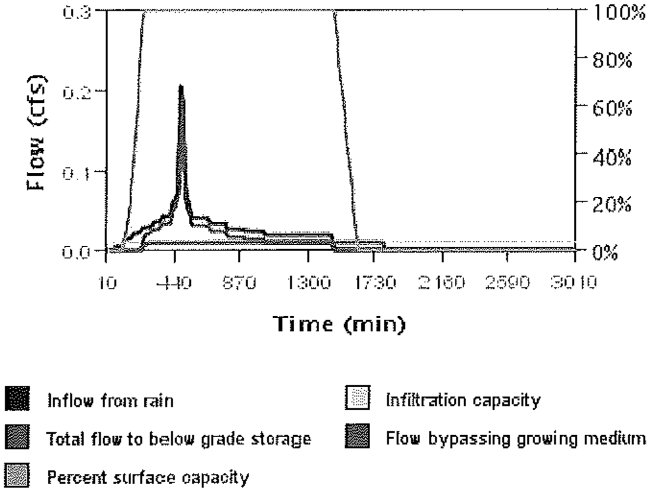
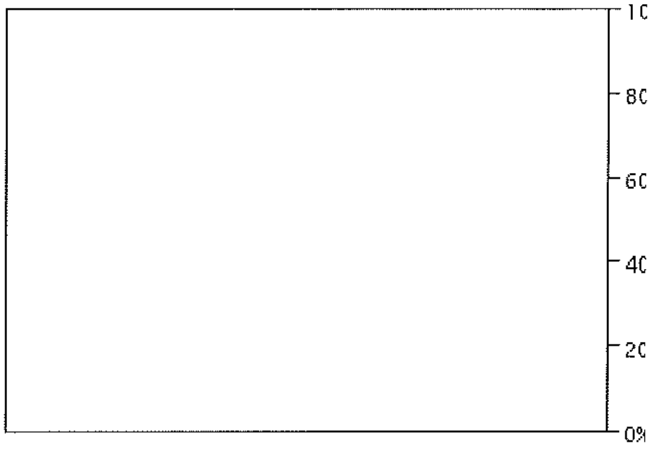
10 Year Event Surface Facility Modeling



10 Year Event Below Grade Modeling



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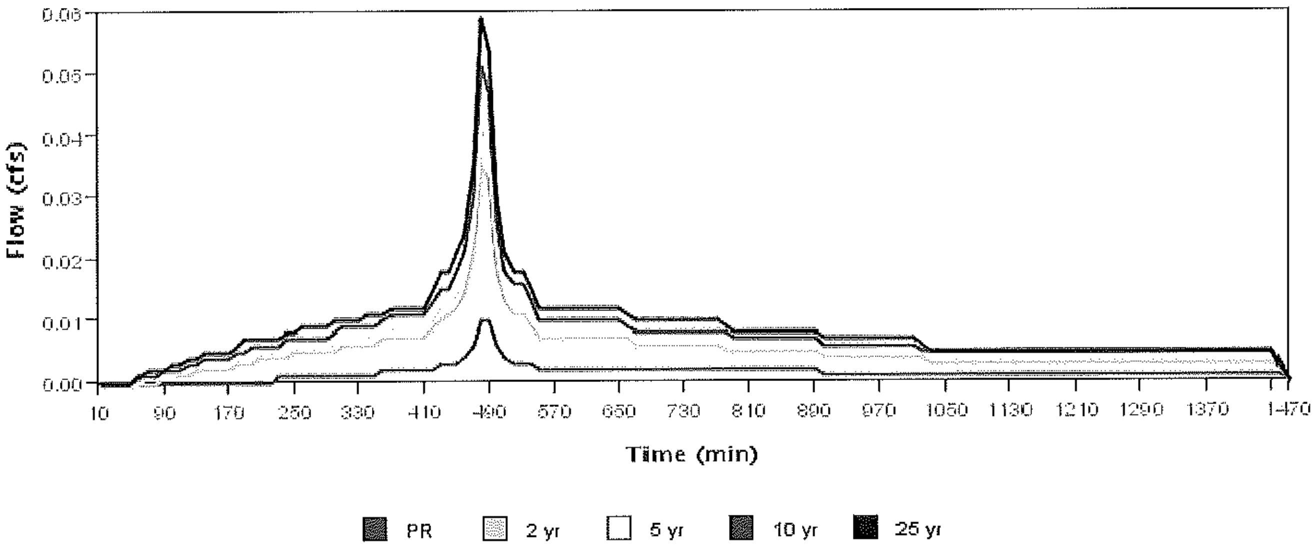
Catchment Basin 2 Parking

Site Soils & Infiltration Testing Data	Infiltration Testing Procedure	**Open Pit Falling Head**
	Native Soil Infiltration Rate (I_{test})	**0.00** ⚠
Correction Factor	CF_{test}	**2**
Design Infiltration Rates	Native Soil (I_{dsgn})	**0.00 in/hr** ⚠
	Imported Growing Medium	**2.00 in/hr**
Catchment Information	Hierarchy Category	**2**
	Hierarchy Description	**On-site infiltration through use of approved UIC facility**
	Pollution Reduction Requirement	**Pass**
	10-year Storm Requirement	**Pass or if Fail, disposal through separate approved UIC**
	Flow Control Requirement	**Pass or if Fail, disposal through separate approved UIC**
	Impervious Area	**2505 sq ft** **0.058 acre**
	Time of Concentration (Tc)	**5**
	Post-Development Curve Number (CN_{post})	**98**

⚠ Indicates value is outside of recommended range

SBUH Results



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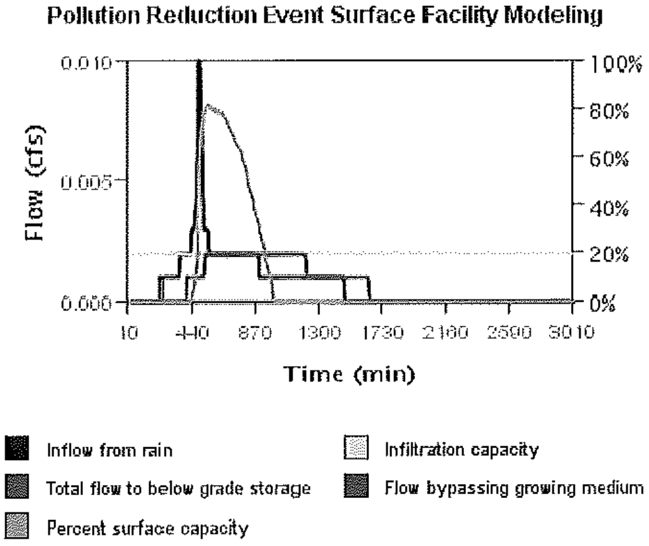
	Peak Rate (cfs)	Volume (cf)
PR	0.01	130.893
2 yr	0.035	453.269
5 yr	0.043	557.091
10 yr	0.051	661.068
25 yr	0.059	765.142

EXHIBIT B

Facility Basin 2 Parking

Facility Details	Facility Type	**Basin**
	Facility Configuration	**D: Lined Facility with RS and Ud**
	Facility Shape	**Rectangle**
	Above Grade Storage Data	
	Bottom Area	**25 sq ft**
	Bottom Width	**6.00 ft**
	Side Slope	**3.0:1**
	Storage Depth 1	**6.0 in**
	Growing Medium Depth	**18 in**
	Freeboard Depth	**2.00 in**
	Surface Capacity at Depth 1	**21.3 cu ft**
	Design Infiltration Rate for Native Soil	**0.000 in/hr**
	Infiltration Capacity	**0.002 cfs**
Facility Facts	Total Facility Area Including Freeboard	**78.23 sq ft**
	Sizing Ratio	**3.1%**
Pollution Reduction Results	Pollution Reduction Score	**Pass**
	Overflow Volume	**132.842 cf**
	Surface Capacity Used	**82%**
10 Year Results	10 Year Score	**Fail**
	Overflow Volume	**662.736 cf**
	Surface Capacity Used	**100%**


Pollution Reduction Event Surface Facility Modeling

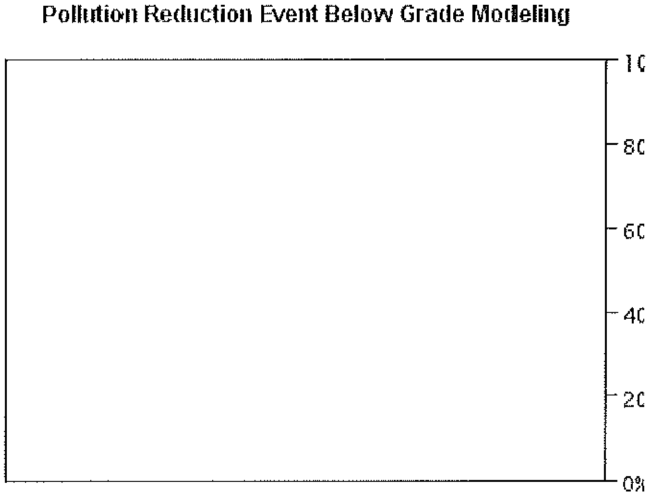
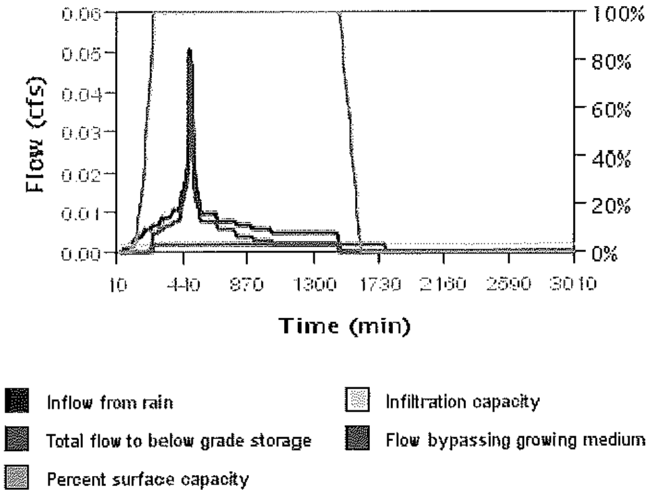

Pollution Reduction Event Below Grade Modeling

EXHIBIT B


10 Year Event Surface Facility Modeling

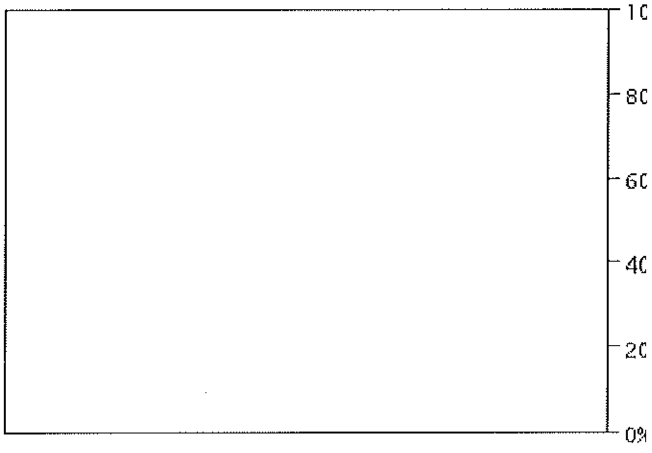

10 Year Event Below Grade Modeling

EXHIBIT B

United States
Department of
Agriculture

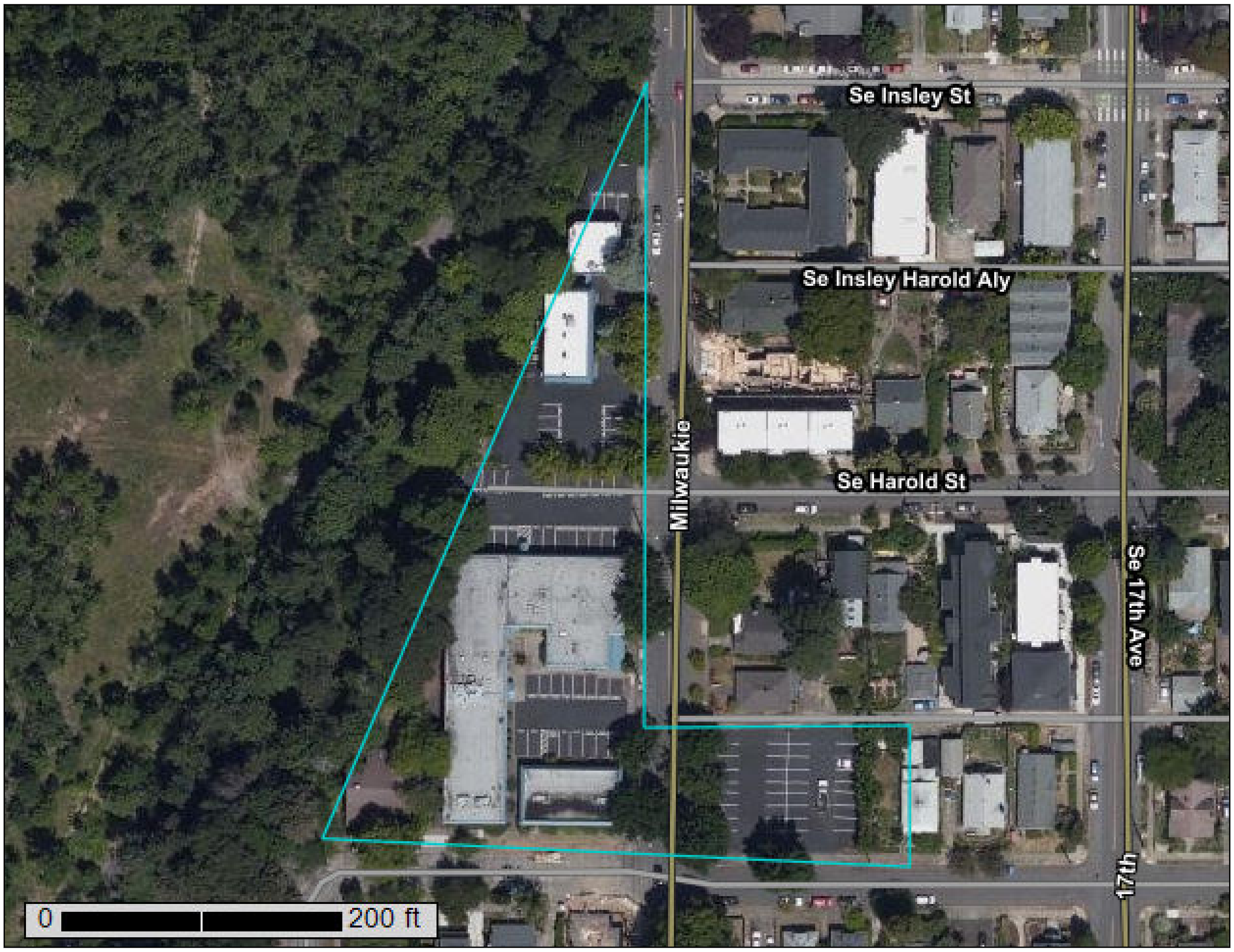
NRCS

Natural
Resources
Conservation
Service

A product of the National Cooperative Soil Survey, a joint effort of the United States Department of Agriculture and other Federal agencies, State agencies including the Agricultural Experiment Stations, and local participants

Custom Soil Resource Report for

Multnomah County Area, Oregon



September 2, 2021

EXHIBIT B

Soil Map

Map Scale: 1:994 if printed on A portrait (8.5" x 11") sheet.

Map projection: Web Mercator Corner coordinates: WGS84 Edge tics: UTM Zone 10N WGS84

EXHIBIT B

MAP LEGEND

Area of Interest (AOI)

- ☐ Area of Interest (AOI)

Soils

- ☐ Soil Map Unit Polygons
- Soil Map Unit Lines
- ■ Soil Map Unit Points

Special Point Features

- Blowout
- ☒ Borrow Pit
- Clay Spot
- ◇ Closed Depression
- Gravel Pit
- Gravelly Spot
- Landfill
- Lava Flow
- Marsh or swamp
- Mine or Quarry
- ◎ Miscellaneous Water
- ◎ Perennial Water
- Rock Outcrop
- + Saline Spot
- Sandy Spot
- Severely Eroded Spot
- ◇ Sinkhole
- Slide or Slip
- Sodic Spot
- Spoil Area
- Stony Spot
- Very Stony Spot
- Wet Spot
- △ Other
- Special Line Features

Water Features

- Streams and Canals

Transportation

- Rails
- Interstate Highways
- US Routes
- Major Roads
- Local Roads

Background

- Aerial Photography

MAP INFORMATION

The soil surveys that comprise your AOI were mapped at 1:20,000.

Warning: Soil Map may not be valid at this scale.

Enlargement of maps beyond the scale of mapping can cause misunderstanding of the detail of mapping and accuracy of soil line placement. The maps do not show the small areas of contrasting soils that could have been shown at a more detailed scale.

Please rely on the bar scale on each map sheet for map measurements.

Source of Map: Natural Resources Conservation Service
Web Soil Survey URL:
Coordinate System: Web Mercator (EPSG:3857)

Maps from the Web Soil Survey are based on the Web Mercator projection, which preserves direction and shape but distorts distance and area. A projection that preserves area, such as the Albers equal-area conic projection, should be used if more accurate calculations of distance or area are required.

This product is generated from the USDA-NRCS certified data as of the version date(s) listed below.

Soil Survey Area: Multnomah County Area, Oregon
Survey Area Data: Version 18, Jun 11, 2020

Soil map units are labeled (as space allows) for map scales 1:50,000 or larger.

Date(s) aerial images were photographed: Jun 13, 2019—Jul 25, 2019

The orthophoto or other base map on which the soil lines were compiled and digitized probably differs from the background imagery displayed on these maps. As a result, some minor shifting of map unit boundaries may be evident.

EXHIBIT B

Map Unit Legend

Map Unit Symbol	Map Unit Name	Acres in AOI	Percent of AOI
19E	Haploxerolls, steep	0.1	6.8%
50C	Urban land, 3 to 15 percent slopes	0.0	0.3%
51A	Urban land-Latourell complex, 0 to 3 percent slopes	1.0	54.4%
51B	Urban land-Latourell complex, 3 to 8 percent slopes	0.7	38.5%
Totals for Area of Interest		**1.9**	**100.0%**

Map Unit Descriptions

The map units delineated on the detailed soil maps in a soil survey represent the soils or miscellaneous areas in the survey area. The map unit descriptions, along with the maps, can be used to determine the composition and properties of a unit.

A map unit delineation on a soil map represents an area dominated by one or more major kinds of soil or miscellaneous areas. A map unit is identified and named according to the taxonomic classification of the dominant soils. Within a taxonomic class there are precisely defined limits for the properties of the soils. On the landscape, however, the soils are natural phenomena, and they have the characteristic variability of all natural phenomena. Thus, the range of some observed properties may extend beyond the limits defined for a taxonomic class. Areas of soils of a single taxonomic class rarely, if ever, can be mapped without including areas of other taxonomic classes. Consequently, every map unit is made up of the soils or miscellaneous areas for which it is named and some minor components that belong to taxonomic classes other than those of the major soils.

Most minor soils have properties similar to those of the dominant soil or soils in the map unit, and thus they do not affect use and management. These are called noncontrasting, or similar, components. They may or may not be mentioned in a particular map unit description. Other minor components, however, have properties and behavioral characteristics divergent enough to affect use or to require different management. These are called contrasting, or dissimilar, components. They generally are in small areas and could not be mapped separately because of the scale used. Some small areas of strongly contrasting soils or miscellaneous areas are identified by a special symbol on the maps. If included in the database for a given area, the contrasting minor components are identified in the map unit descriptions along with some characteristics of each. A few areas of minor components may not have been observed, and consequently they are not mentioned in the descriptions, especially where the pattern was so complex that it was impractical to make enough observations to identify all the soils and miscellaneous areas on the landscape.

The presence of minor components in a map unit in no way diminishes the usefulness or accuracy of the data. The objective of mapping is not to delineate pure taxonomic classes but rather to separate the landscape into landforms or landform segments that have similar use and management requirements. The delineation of such segments on the map provides sufficient information for the development of resource plans. If intensive use of small areas is planned, however, onsite investigation is needed to define and locate the soils and miscellaneous areas.

An identifying symbol precedes the map unit name in the map unit descriptions. Each description includes general facts about the unit and gives important soil properties and qualities.

Soils that have profiles that are almost alike make up a *soil series*. Except for differences in texture of the surface layer, all the soils of a series have major horizons that are similar in composition, thickness, and arrangement.

Soils of one series can differ in texture of the surface layer, slope, stoniness, salinity, degree of erosion, and other characteristics that affect their use. On the basis of such differences, a soil series is divided into *soil phases*. Most of the areas shown on the detailed soil maps are phases of soil series. The name of a soil phase commonly indicates a feature that affects use or management. For example, Alpha silt loam, 0 to 2 percent slopes, is a phase of the Alpha series.

Some map units are made up of two or more major soils or miscellaneous areas. These map units are complexes, associations, or undifferentiated groups.

A *complex* consists of two or more soils or miscellaneous areas in such an intricate pattern or in such small areas that they cannot be shown separately on the maps. The pattern and proportion of the soils or miscellaneous areas are somewhat similar in all areas. Alpha-Beta complex, 0 to 6 percent slopes, is an example.

An *association* is made up of two or more geographically associated soils or miscellaneous areas that are shown as one unit on the maps. Because of present or anticipated uses of the map units in the survey area, it was not considered practical or necessary to map the soils or miscellaneous areas separately. The pattern and relative proportion of the soils or miscellaneous areas are somewhat similar. Alpha-Beta association, 0 to 2 percent slopes, is an example.

An *undifferentiated group* is made up of two or more soils or miscellaneous areas that could be mapped individually but are mapped as one unit because similar interpretations can be made for use and management. The pattern and proportion of the soils or miscellaneous areas in a mapped area are not uniform. An area can be made up of only one of the major soils or miscellaneous areas, or it can be made up of all of them. Alpha and Beta soils, 0 to 2 percent slopes, is an example.

Some surveys include *miscellaneous areas*. Such areas have little or no soil material and support little or no vegetation. Rock outcrop is an example.

EXHIBIT B

Multnomah County Area, Oregon

19E—Haploxerolls, steep

Map Unit Setting

National map unit symbol: 228v
Elevation: 50 to 400 feet
Mean annual precipitation: 40 to 60 inches
Mean annual air temperature: 50 to 54 degrees F
Frost-free period: 165 to 210 days
Farmland classification: Not prime farmland

Map Unit Composition

Haploxerolls and similar soils: 80 percent
Minor components: 2 percent
Estimates are based on observations, descriptions, and transects of the mapunit.

Description of Haploxerolls

Setting

Landform: Terraces
Landform position (three-dimensional): Riser
Down-slope shape: Linear
Across-slope shape: Linear
Parent material: Sandy and silty alluvium

Typical profile

H1 - 0 to 11 inches: sandy loam
H2 - 11 to 39 inches: sandy loam
H3 - 39 to 60 inches: gravelly loamy sand

Properties and qualities

Slope: 30 to 70 percent
Depth to restrictive feature: More than 80 inches
Drainage class: Well drained
Capacity of the most limiting layer to transmit water (Ksat): Moderately high to high (0.20 to 5.95 in/hr)
Depth to water table: About 36 to 72 inches
Frequency of flooding: None
Frequency of ponding: None
Available water supply, 0 to 60 inches: Moderate (about 8.5 inches)

Interpretive groups

Land capability classification (irrigated): None specified
Land capability classification (nonirrigated): 6e
Hydrologic Soil Group: A
Hydric soil rating: No

Minor Components

Aquolls, seeps

Percent of map unit: 2 percent
Landform: Depressions
Hydric soil rating: Yes

50C—Urban land, 3 to 15 percent slopes

Map Unit Setting

National map unit symbol: 22bw
Elevation: 50 to 100 feet
Farmland classification: Not prime farmland

Map Unit Composition

Urban land: 100 percent
Estimates are based on observations, descriptions, and transects of the mapunit.

Description of Urban Land

Interpretive groups

Land capability classification (irrigated): None specified
Land capability classification (nonirrigated): 8
Hydric soil rating: No

51A—Urban land-Latourell complex, 0 to 3 percent slopes

Map Unit Setting

National map unit symbol: 22bx
Elevation: 50 to 400 feet
Mean annual precipitation: 40 to 60 inches
Mean annual air temperature: 52 to 54 degrees F
Frost-free period: 165 to 210 days
Farmland classification: Not prime farmland

Map Unit Composition

Urban land: 50 percent
Latourell and similar soils: 40 percent
Estimates are based on observations, descriptions, and transects of the mapunit.

Description of Urban Land

Interpretive groups

Land capability classification (irrigated): None specified
Land capability classification (nonirrigated): 8
Hydric soil rating: No

Description of Latourell

Setting

Landform: Terraces
Landform position (three-dimensional): Tread

Down-slope shape: Linear
Across-slope shape: Convex
Parent material: Medium textured alluvium

Typical profile

H1 - 0 to 16 inches: loam
H2 - 16 to 56 inches: loam
H3 - 56 to 66 inches: very gravelly sandy loam

Properties and qualities

Slope: 0 to 3 percent
Depth to restrictive feature: More than 80 inches
Drainage class: Well drained
Capacity of the most limiting layer to transmit water (Ksat): Moderately high to high (0.57 to 1.98 in/hr)
Depth to water table: More than 80 inches
Frequency of flooding: None
Frequency of ponding: None
Available water supply, 0 to 60 inches: High (about 10.3 inches)

Interpretive groups

Land capability classification (irrigated): 1
Land capability classification (nonirrigated): 1
Hydrologic Soil Group: B
Hydric soil rating: No

51B—Urban land-Latourell complex, 3 to 8 percent slopes

Map Unit Setting

National map unit symbol: 22by
Elevation: 50 to 400 feet
Mean annual precipitation: 40 to 60 inches
Mean annual air temperature: 52 to 54 degrees F
Frost-free period: 165 to 210 days
Farmland classification: Not prime farmland

Map Unit Composition

Urban land: 50 percent
Latourell and similar soils: 40 percent
Estimates are based on observations, descriptions, and transects of the mapunit.

Description of Urban Land

Interpretive groups

Land capability classification (irrigated): None specified
Land capability classification (nonirrigated): 8
Hydric soil rating: No

EXHIBIT B

Description of Latourell

Setting

Landform: Terraces
Landform position (three-dimensional): Tread
Down-slope shape: Linear
Across-slope shape: Convex
Parent material: Medium textured alluvium

Typical profile

H1 - 0 to 16 inches: loam
H2 - 16 to 56 inches: loam
H3 - 56 to 66 inches: very gravelly sandy loam

Properties and qualities

Slope: 3 to 8 percent
Depth to restrictive feature: More than 80 inches
Drainage class: Well drained
Capacity of the most limiting layer to transmit water (Ksat): Moderately high to high (0.57 to 1.98 in/hr)
Depth to water table: More than 80 inches
Frequency of flooding: None
Frequency of ponding: None
Available water supply, 0 to 60 inches: High (about 10.3 inches)

Interpretive groups

Land capability classification (irrigated): 2e
Land capability classification (nonirrigated): 2e
Hydrologic Soil Group: B
Hydric soil rating: No

EXHIBIT B

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EXHIBIT B

Table A-2. Curve Numbers for Urban Areas

Cover type and hydrological condition	Average percent impervious area	Curve Numbers by Hydrologic Soil Group			
		A	B	C	D
Open Space (lawns, parks, golf courses, cemeteries, etc.):					
Poor condition (grass cover <50%)		68	79	86	89
Fair condition (grass cover 50-75%)		49	69	79	84
Good condition (grass cover >75%)		39	61	74	80
Impervious Area:					
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	98	98
Paved; open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	93
Urban Districts:					
Commercial and business	85	85	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	82
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

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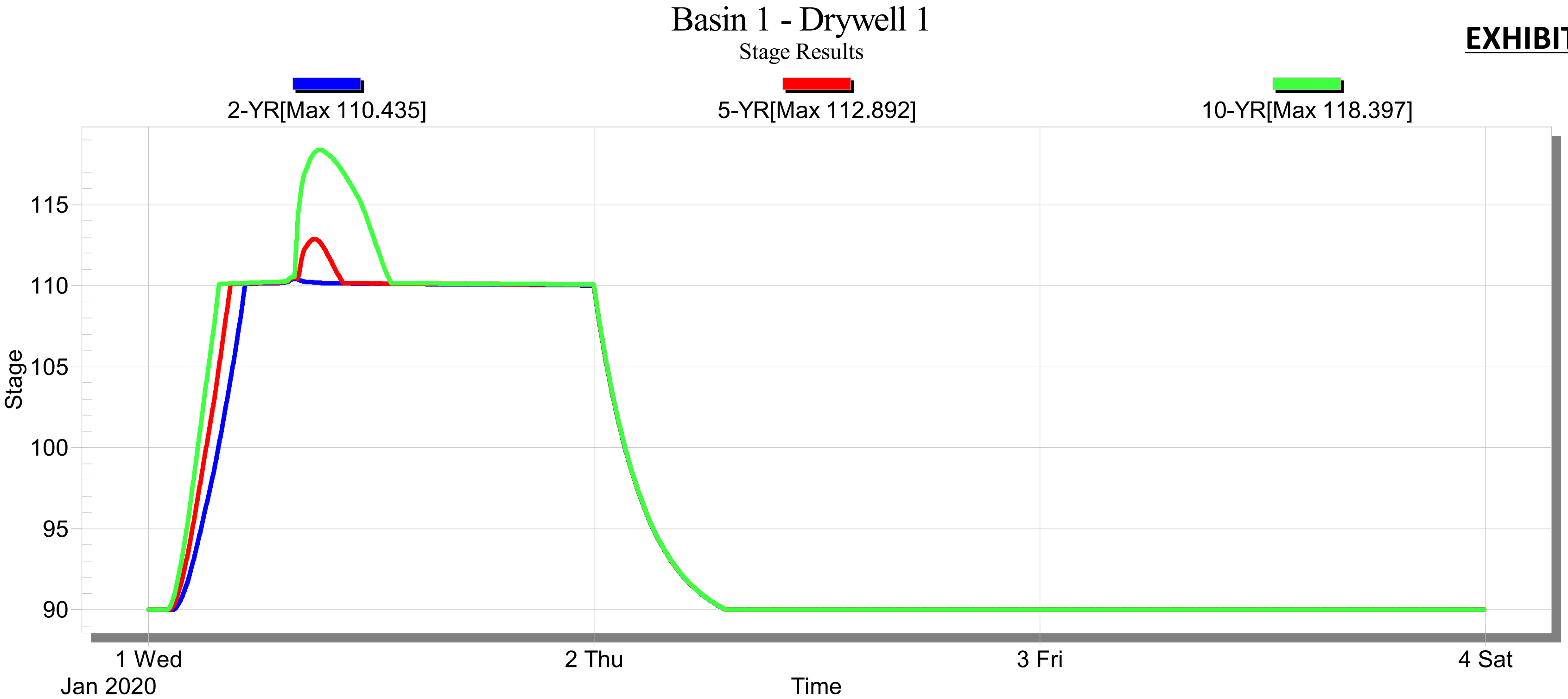
EXHIBIT B


Basin 1 - Drywell 1
Stage Results
2-YR[Max 110.435]
5-YR[Max 112.892]
10-YR[Max 118.397]
Stage
115
110
105
100
95
90
1 Wed
Jan 2020
2 Thu
3 Fri
4 Sat
Time

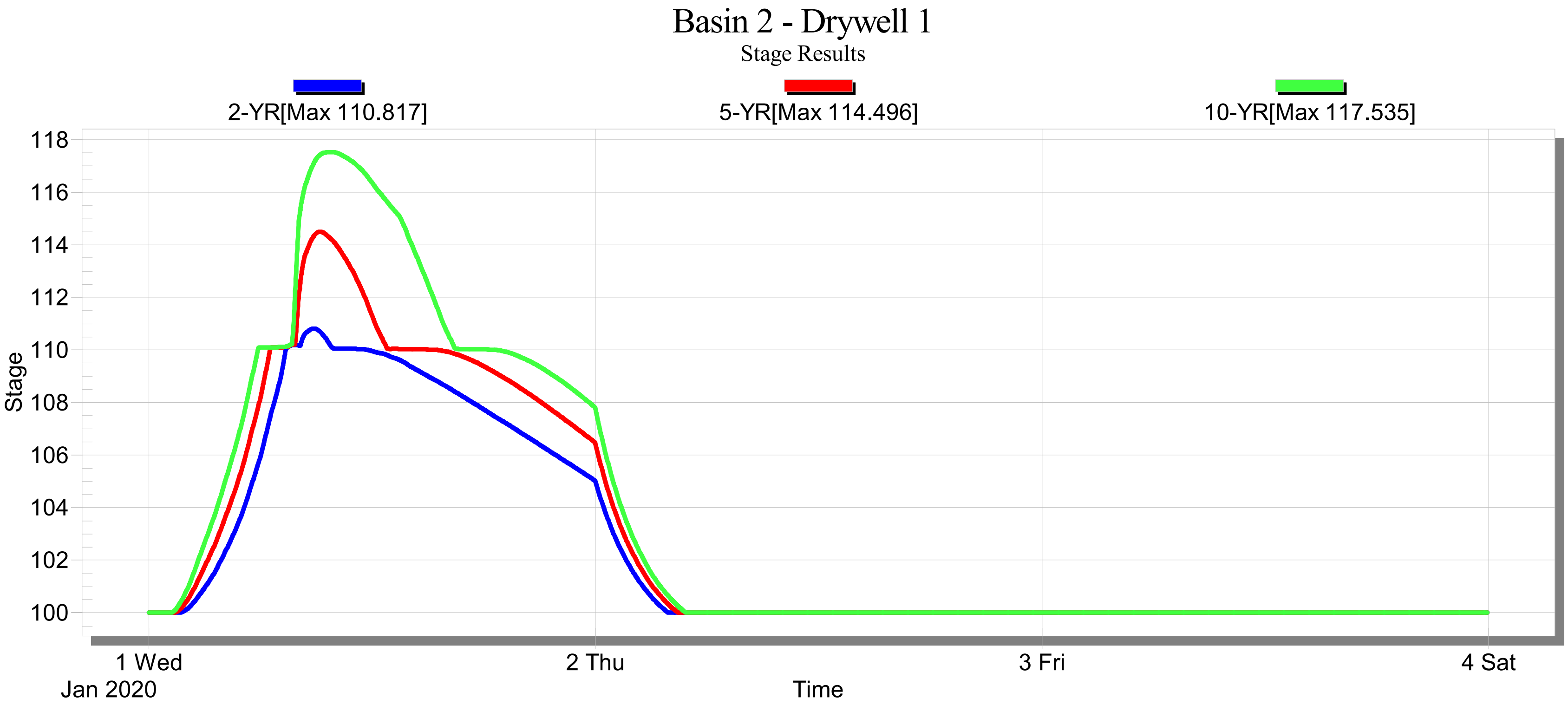

Basin 2 - Drywell 1
Stage Results
2-YR[Max 110.817]
5-YR[Max 114.496]
10-YR[Max 117.535]
Stage
118
116
114
112
110
108
106
104
102
100
1 Wed
Jan 2020
2 Thu
3 Fri
4 Sat
Time

EXHIBIT B

GEODESIGN INC
AN NV5 COMPANY

REPORT OF GEOTECHNICAL ENGINEERING SERVICES

Proposed RM4 Development
5415 and 5425 SE Milwaukie Avenue
Portland, Oregon

For
Columbia Capital Group LLC
May 15, 2020

GeoDesign Project: CCGPDX-2-01

EXHIBIT B

GeoDesign Inc
AN NV5 COMPANY

May 15, 2020

Columbia Capital Group LLC
PO Box 14667
Portland, OR 97293

Attention: Blaine Whitney

Report of Geotechnical Engineering Services
Proposed RM4 Development
5415 and 5425 SE Milwaukie Avenue
Portland, Oregon
GeoDesign Project: CCGPDX-2-01

GeoDesign, Inc. is pleased to submit this report of geotechnical engineering for the proposed RM4 development located at 5415 and 5425 SE Milwaukie Avenue in Portland, Oregon. Our services for this project were conducted in accordance with our proposal dated December 20, 2020.

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

Sincerely,

GeoDesign, Inc.

Brett A. Shipton, P.E., G.E.
Principal Engineer

GJS:BAS:kt
Attachments
One copy submitted (via email only)
Document ID: CCGPDX-2-01-051520-geor.docx

EXHIBIT B

EXECUTIVE SUMMARY

Based on our understanding of the project and the results of our explorations, laboratory testing, and analyses, it is our opinion that the proposed project can be constructed at the site. Since foundation loads are not know at this time, re-evaluation of the recommendations may be necessary. The primary geotechnical considerations for the project are summarized as follows:

- Shallow foundations should be offset by a distance of 30 feet from the top of the existing slope. We recommend that footings within 30 feet of the existing top of slope be supported on a deep foundation system, or ground that has been improved with rammed aggregate piers or DSMC foundations, that transfer foundation loads to the underlying very dense gravel encountered at a depth of approximately 20 feet BGS.
- Depending on foundation loads, foundations located at a distance greater than 30 feet from the top of the existing slope can either bear on the existing native silt and clay, on structural fill overlying the native silt and clay, or on a deep foundation system.
- Where undocumented fill exists under proposed slab-on-grade floors, the fill should be evaluated during construction to identify soft, loose, or deleterious material. Soft, loose, or deleterious material should be removed and replaced with structural fill prior to installation of base rock for slab-on-grade floors.
- The native soil is generally suitable for use as structural fill during periods of dry weather, provided it is properly moisture conditioned. Moisture conditioning the soil will typically consist of drying the soil to within 3 percent of the optimum moisture content required for compaction. Moisture conditioning is expected to require significant time and effort. It will not be possible to adequately compact native soil during the rainy season or any period of prolonged wet weather.
- The native soil is easily disturbed during the wet season or when wet of optimum moisture content. Subgrade protection will be necessary if construction occurs during the wet season. Wet, sensitive subgrade should be anticipated.

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    4.2 Subsurface Conditions 2
    4.3 Infiltration Testing 3
    4.4 Slope Stability 4
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APPENDICES

Appendix A

Field Explorations A-1

Laboratory Testing A-1

Exploration Key Table A-1

Soil Classification System Table A-2

Boring Logs Figures A-1 - A-2

Atterberg Limits Test Results Figure A-3

Consolidation Test Results Figure A-4

Summary of Laboratory Data Figure A-5

SPT Hammer Calibration

Appendix B

Surveyor's Plot Map of Site Provided by Columbia Capital Group B-1

Plot Map

EXHIBIT B

ACRONYMS AND ABBREVIATIONS

AC	asphalt concrete
AOS	apparent opening size
ASCE	American Society of Civil Engineers
ASTM	American Society for Testing and Materials
BGS	below ground surface
DSMC	deep soil mix column
g	gravitational acceleration (32.2 feet/second^2)
H:V	horizontal to vertical
IBC	International Building Code
MCE	maximum considered earthquake
OSHA	Occupational Safety and Health Administration
OSSC	Oregon Standard Specifications for Construction (2018)
pcf	pounds per cubic foot
PGA	peak ground acceleration
psf	pounds per square foot
psi	pounds per square inch
SOSSC	State of Oregon Structural Specialty Code
SPT	standard penetration test
USGS	U.S. Geological Survey

EXHIBIT B

1.0 INTRODUCTION

GeoDesign, Inc. is pleased to submit this report of geotechnical engineering services for the proposed RM4 development at 5415 and 5425 SE Milwaukie Avenue in Portland, Oregon. Figure 1 shows the site relative to existing physical features. Figure 2 shows the current site layout and our approximate exploration locations. The exploration logs are presented in Appendix A. Acronyms and abbreviations used herein are defined above, immediately following the Table of Contents.

2.0 PROJECT UNDERSTANDING

The site is currently occupied by two office buildings with two and three stories, respectively, and associated AC parking lots. The existing structures and parking lots will be demolished to accommodate the new development.

The proposed development consists of a seven-story, residential apartment building with no basement. The building will have a footprint of approximately 14,100 square feet and consist of five floors of wood-framed construction over two floors of concrete construction. Structural loads were not available at the time of this report. We have assumed that maximum column loads will be up to 800 kips and wall loads will be up to 16 kips per foot. Floor slab loads are assumed to be less than 100 psf. We have assumed that cuts and fills will be less than approximately 5 feet each.

3.0 SCOPE OF SERVICES

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

- Reviewed readily available, published geologic data and our in-house files for existing information on subsurface conditions in the site vicinity.
- Coordinated and managed the field investigation, including locating utilities and scheduling subcontractors and GeoDesign field staff.
- Explored subsurface conditions at the site by drilling two borings to depths of 31.5 and 35.1 feet BGS.
- Maintained continuous logs of the explorations and collected soil samples at representative intervals.
- Performed infiltration testing at a depth of 10 feet BGS in one of the borings in accordance with City of Portland standards.
- Completed the following laboratory testing on soil samples collected from the explorations:
    - Fifteen moisture content determinations in general accordance with ASTM D2216
    - Three particle-size analyses in general accordance with ASTM D1140
    - One Atterberg limits test in general accordance with ASTM D4318
    - One consolidation test in general accordance with ASTM D2435

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- Provided recommendations for site preparation and grading, including demolition, temporary and permanent slopes, shoring, fill placement criteria, suitability of on-site soil for fill, subgrade preparation, and wet weather construction.
- Provided an evaluation of groundwater conditions at the site and general recommendations for dewatering during construction and subsurface drainage.
- Provided foundation support recommendations, including foundation types, allowable bearing pressures, lateral resistance parameters, and settlement estimates.
- Provided recommendations for use in design of conventional retaining walls, including backfill and drainage requirements and lateral earth pressures.
- Provided recommendations for slope stability.
- Provide seismic design recommendations in accordance with the procedures outlined in the 2019 SOSSC.
- Prepared this geotechnical engineering report that presents our findings, conclusions, and recommendations.

4.0 SITE DESCRIPTION

4.1 SURFACE CONDITIONS

The approximately 1.07-acre site is bound by SE Milwaukie Avenue to the east, commercial property to the south, and the Oaks Bottom Wildlife Refuge to the north and west. The site is currently occupied by two office buildings with two and three stories, respectively, and associated AC parking lots.

The developed portion of the site is relatively flat. From the west edge of the developed area the site slopes downward at an approximately 1.2H:1V grade to a spur of the Springwater Trail to the west. Based on an historical surveyor's plot map provided by you and presented in Appendix B, we understand that the elevation of the existing parking areas ranges from 93 to 95 feet (City of Portland datum) and the elevation of the Springwater Trail to the west ranges from 37 to 63 feet (City of Portland datum).

4.2 SUBSURFACE CONDITIONS

We explored subsurface conditions at the site by drilling two borings (B-1 and B-2) to depths of 31.5 and 35.1 feet BGS. The approximate exploration locations are shown on Figure 2. A description of the subsurface exploration and laboratory testing programs, the exploration logs, and laboratory test results are presented in Appendix A.

The soil conditions encountered during our subsurface explorations generally consist of silt and clay over sand and gravel. Boring B-1 encountered undocumented fill to a depth of 7.5 feet BGS. Pavement layers consisting of 2.3 to 2.8 inches of AC over 5.8 to 6.3 inches of aggregate base were encountered at the surface of both borings. A detailed description of the soil encountered on site is presented below.

4.2.1 Fill

Undocumented fill consisting of silt with varying proportions of sand is present directly below the AC and aggregate base sections in boring B-1 and extends to a depth of approximately

7.5 feet BGS. SPT data indicate that the fill is very stiff in consistency, and laboratory data indicates that the fill had a moisture content ranging between 28 and 29 percent at the time of our explorations.

4.2.2 Silt and Clay

Native fine-grained soil consisting of clay and sandy silt was encountered beneath the fill in boring B-1 and the AC and aggregate base in boring B-2 and extends to a depth of approximately 10 feet BGS. SPT data indicates that the native fine-grained material is stiff to very stiff in consistency. Laboratory testing indicates that the fine-grained material had a moisture content ranging from 26 to 38 percent at the time of our explorations.

4.2.3 Silty Sand

Silty sand underlies the silt and clay layers and extends to depths between 19.5 and 20.5 feet BGS. SPT data indicates that the silty sand is medium dense to very dense in relative density. Laboratory testing indicates that the silty sand had a moisture content ranging from 16 to 23 percent at the time of our explorations.

4.2.4 Gravel

Gravel with variable silt and sand content underlies the silty sand and extends to the maximum explored depth in both borings. SPT data indicates that the gravel is very dense in relative density. Laboratory data indicates that the gravel had a moisture content ranging from 9 to 13 percent at the time of our explorations.

4.2.5 Groundwater

Groundwater levels could not be observed during drilling due to mud rotary drilling methods. According to the estimated depth to groundwater mapping published by USGS (Snyder, 2008), the regional groundwater table is estimated to be at a depth of approximately 36 to 40 feet BGS. The depth to groundwater may fluctuate in response to prolonged rainfall, seasonal changes, changes in surface topography, and other factors not observed during this study.

4.3 INFILTRATION TESTING

Infiltration testing was completed to assist in the evaluation of infiltration rates at the site. Infiltration testing was conducted in general accordance with the 2016 City of Portland *Stormwater Management Manual.*

The test was performed at a depth of 10 feet BGS in boring B-2. The infiltration test was performed using the encased falling-head test method and was conducted in a 6-inch-diameter pipe. After soaking the soil under a constant head of water, infiltration rates were measured under low-head conditions of approximately 1 foot to 2 feet of water. Fines content testing was conducted on a representative soil sample collected at the depth of the infiltration test. The fines content test results are presented in Appendix A. Table 1 summarizes the results of infiltration testing and fines content determination.

Table 1. Field Measured Infiltration Rate

Location	Depth (feet BGS)	Material	Infiltration Rate[1] (inches per hour)	Fines Content[2] (percent)
B-2	10	Silty Sand	8.2	22

1. Infiltration rates are not factored.
2. Fines content: material passing the U.S. Standard No. 200 sieve

The infiltration rate presented in Table 1 is unfactored. Correction factors should be applied to the measured infiltration rate to account for soil variations and the potential for long-term clogging due to siltation and buildup of organic material. The City of Portland *Stormwater Management Manual* recommends a minimum allowable factor of safety of 2 be used for the encased falling-head test method.

4.4 SLOPE STABILITY

The global stability of the existing slope was analyzed using the SLOPE/W (version 10) application developed by Geo-Slope International, Ltd. Our analyses used the limit equilibrium Morgenstern-Price method. The slope geometry was estimated from an historical surveyor's plot map provided by you (see Appendix B). Soil parameters for the embankment were estimated based on our explorations.

Seismic pressures were determined using the Mononabe-Okabe method and a pseudo-static load corresponding to a horizontal seismic load coefficient of 0.20. The vertical component of seismic load was assumed to be zero.

Based on our analysis, the stability of the existing slope satisfies a minimum factor of safety of 1.5 for static conditions and 1.1 for seismic conditions.

Final design loads for the proposed building were not available at the time of this report. Once final design loads have been determined, we recommend that additional slope stability analyses may be required to model the effect of the proposed building on the nearby slope.

5.0 SITE DEVELOPMENT RECOMMENDATIONS

5.1 SITE PREPARATION

5.1.1 Demolition

Demolition includes the complete removal of the existing structures, concrete footings, pavement, utilities, and various other former site improvements that may be encountered during construction. We recommend that all abandoned underground vaults, underground storage tanks, septic tanks, manholes, utility lines, foundation elements, and other subsurface structures that are beneath new structural components be entirely removed.

Voids resulting from the removal of improvements should be backfilled with compacted structural fill, as discussed in the "Structural Fill" section. Utility lines abandoned under new structural components should be completely removed and backfilled with structural fill. Firm

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subgrade should be exposed at the bottom of the excavations before backfilling, and the sides of the temporary excavations should be sloped at a minimum of 1H:1V.

Demolished material should be transported off site for disposal. Soft soil encountered during site preparation should be replaced with structural fill.

5.1.2 Clearing

Stripping and clearing shall be completed to remove vegetation within the development. Limited areas of exposed soil are currently present in the proposed work area. Trees and shrubs should be removed from all new pavement and improvement areas. In addition, root balls should be grubbed out to the depth of roots greater than ½ inch in diameter, which could exceed 3 feet BGS for some of the larger trees. Depending on the method used to remove root balls, considerable disturbance and loosening of the subgrade could occur during site grubbing. We recommend that soil disturbed during grubbing operations be removed to expose firm, undisturbed subgrade. The resulting excavations should be backfilled with structural fill. Stripped material should be transported off site for disposal or as directed by owner.

5.1.3 Undocumented Fill

Fill was encountered in the borings. It appears that it was placed as engineered fill based on its consistency. This should be verified during construction. All soft or loose material should be removed and replaced with structural fill in the influence zone of foundations. Soil processing, including moisture conditioning and the removal of roots, cobbles, and other deleterious material from the soil, may be required to use the excavated material as structural fill. Compaction should be performed as described in the "Structural Fill" section.

5.1.4 Wet Weather Construction

Trafficability of the soil may be difficult during or after extended wet periods or when the moisture content of the surface soil is more than a few percentage points above optimum. When wet, the surficial fine-grained soil is easily disturbed and may provide inadequate support for construction equipment. If construction occurs during the wet season or wet subgrade is present, site preparation activities may need to be accomplished using track-mounted equipment, loading removed material into trucks supported on granular haul roads, or working progressively across the site over unexposed surfaces.

The base rock thickness for floor slab and pavement areas is intended to support post-construction design traffic loads. This design base rock thickness may not support construction traffic or pavement construction when the subgrade soil is wet. Accordingly, if construction is planned for periods when the subgrade soil is wet, staging and haul roads with increased thicknesses of base rock will be required. The location and number of staging and haul road areas, as well as the required thickness of granular material, will vary with the contractor's sequencing of a project and the type/frequency of construction equipment. Based on our experience, between 12 and 18 inches of imported granular material is generally required in staging areas and between 18 and 24 inches in haul roads areas. Stabilization material may be used as a substitute, provided the top 4 inches of material consists of imported granular material. The actual thickness will depend on the contractor's means and methods and, accordingly, should be the contractor's responsibility.

The imported granular material and stabilization material should meet the specifications in the "Structural Fill" section. We recommend that a geotextile fabric be placed as a barrier between the subgrade and imported granular material in areas of repeated construction traffic. The geotextile should have a minimum Mullen burst strength of 250 psi for puncture resistance and an AOS between U.S. Standard No. 70 and No. 100 sieves.

5.1.5 Subgrade Evaluation

After demolition, clearing, and the required site preparation have been completed, we recommend proof rolling the subgrade with a fully loaded dump truck or similar sized rubber tire construction equipment to identify areas of excessive yielding. Proof rolling should be observed by a member of our geotechnical staff who will evaluate the subgrade. If areas of excessive yielding are identified, the material should be excavated and replaced with compacted material recommended for structural fill. Areas that are too small for proof rolling or that appear to be too wet and soft to support proof rolling equipment should be evaluated by probing and prepared in accordance with recommendations presented in the "Construction Considerations" section.

5.2 CONSTRUCTION CONSIDERATIONS

We do not anticipate that the groundwater table will be encountered during construction. However, perched groundwater may be present during the wet season or after periods of precipitation. Consequently, dewatering may be required to control perched groundwater if present. We anticipate that perched groundwater flow, if encountered, will diminish over time and can be addressed using sumps and pumps internal to the excavation.

We recommend placing a layer of stabilization material over the subgrade that will be exposed to construction traffic to protect it during wet weather. The contractor has control of the construction schedule and equipment and, therefore, should be responsible for selecting the appropriate working blanket and thickness. However, it is our opinion that a 12-inch-thick section should be sufficient for light staging areas and an 18-inch-thick blanket should be sufficient for areas subject to heavy construction traffic. Stabilization material should consist of well-graded gravel, crushed gravel, or crushed rock with a minimum particle size of 3 inches and less than 5 percent by dry weight passing the U.S. Standard No. 4 sieve. Stabilization material should be placed in one lift.

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

5.3 EXCAVATION

5.3.1 General

Conventional earthmoving equipment in proper working condition should be capable of making necessary excavations for site cuts and utilities. Trench cuts should stand vertical to a depth of approximately 4 feet. Open excavation techniques may be used to excavate trenches with depths between 4 and 10 feet BGS, provided the walls of the excavation are cut at a slope of

1H:1V or flatter and groundwater seepage is not present. These recommendations are based on the assumption that surcharge loads will not be present within H feet, where H is the depth of the trench.

If shoring is used, we recommend that the type and design of the shoring system be the responsibility of the contractor, who is in the best position to choose a system that fits the overall plan of operation.

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

5.3.2 Dewatering

Dewatering may be required to maintain dry working conditions for trench excavations. Dewatering systems are best designed by the contractor. A sump located within the trench excavations may remove accumulated water, depending on the amount and persistence of water seepage and the length of time the trench is left open. Flow rates for dewatering are likely to vary depending on location, soil type, and the season during which the excavation occurs. Dewatering systems should be capable of adapting to variable flows.

If groundwater is present in the base of trench excavations, we recommend over-excavating the trench by a minimum of 6 inches and placing trench stabilization material in the base. Specifications for stabilization material are provided in the "Structural Fill" section.

5.4 SHORING

Temporary support for shallow excavations can be provided by installing soldier piles with timber lagging or cantilevered shoring. Cantilevered shoring should be designed to resist an active lateral earth pressure that has an equivalent fluid pressure of 35 pcf, with an available passive earth pressure at the inside base of the excavation of 350 pcf (applied over 3 soldier pile diameters) above the level of groundwater and 180 pcf below the level of groundwater. These values do not include surcharged-induced earth pressures. The lateral earth pressures listed above are unfactored.

If the surface at the top of the shoring is sloped, the recommended lateral earth pressures should be increased as indicated in Table 2.

Table 2. Recommended Lateral Earth Pressures

Inclination at Top of Shoring (H:V)	Percent Increase in Lateral Earth Pressure (percent)
1:1	200
1.5:1	165
2:1	150

We recommend a vertical live load of 250 psf be applied at the surface of the retained soil where the wall shoring retains roadways.

5.5 MATERIALS

5.5.1 Structural Fill

5.5.1.1 General

Structural fill includes fill beneath foundations, slabs, pavement, any other areas intended to support structures, or within the influence zones of structures. Structural fill should be free of organic and other deleterious materials and, in general, should consist of particles no larger than 3 inches in diameter. A brief characterization of some of the acceptable materials and our recommendations for their use as structural fill are provided below.

5.5.1.2 On-Site Fine-Grained Soil

The near-surface soil at the site consists primarily of fine-grained silt and clay materials. The on-site fine-grained material is typically expected to have a moisture content higher than optimum for compaction. Moisture conditioning of fine-grained soil can typically only be completed during the summer dry season with sufficient surface area to spread and condition the soil. Considering that significant fill is not expected for the site and there will be little or no area available to moisture condition the soil, on-site soil is not recommended for use as structural fill at the site.

5.5.1.3 Imported Granular Material

Imported granular material used for structural fill should be pit- or quarry-run rock, crushed rock, or crushed gravel and sand. Imported granular material should be fairly well graded between coarse and fine material, should have less than 5 percent by dry weight passing the U.S. Standard No. 200 sieve, and should have at least two mechanically fractured faces.

When used as structural fill, imported granular 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.

5.5.1.4 Trench Backfill

Trench backfill for the utility pipe base and pipe zone should consist of well-graded, durable, crushed, granular material with a maximum particle size of ¾ inch and less than 5 percent by dry weight passing the U.S. Standard No. 200 sieve. The material should be free of roots, organic material, and other unsuitable material. Backfill for the pipe base and pipe zone should be compacted to at least 90 percent of the maximum dry density, as determined by ASTM D1557, or as recommended by the pipe manufacturer.

Within building, pavement, and other structural areas, trench backfill placed above the pipe zone should consist of imported granular material as specified above. The backfill should be compacted to at least 92 percent of the maximum dry density, as determined by ASTM D1557, at depths greater than 2 feet below the finished subgrade and 95 percent of the maximum dry density, as determined by ASTM D1557, within 2 feet of finished subgrade. In all other areas, trench backfill above the pipe zone should be compacted to at least 90 percent of the maximum dry density, as determined by ASTM D1557.

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5.5.1.5 Aggregate Base Rock

Imported granular material used as base rock for building floor slabs and pavement should consist of ¾- or 1½-inch-minus material. The aggregate should have less than 5 percent by dry weight passing the U.S. Standard No. 200 sieve and at least two fractured faces. The aggregate base should be compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D1557.

5.5.1.6 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. We recommend the select granular wall backfill be separated from general fill, native soil, and/or topsoil using a geotextile fabric that meets the specifications provided below for drainage geotextiles.

The wall backfill should be compacted to a minimum of 95 percent of the maximum dry density, as determined by ASTM D1557. However, backfill located within a horizontal distance of 3 feet from a retaining wall should only be compacted to approximately 90 percent of the maximum dry density, as determined by ASTM D1557. Backfill placed within 3 feet of 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.

5.5.1.7 Recycled Material

AC, conventional concrete, and oversized rock may be used as fill if they are processed to meet the requirements for their intended use and do not pose an environmental concern. Processing includes crushing and screening, grinding in place, or other methods to meet the requirements for structural fill as described above. The processed material should be fairly well graded and not contain metal, organic material, or other deleterious material. The processed material may be mixed with on-site soil or imported fill to assist in achieving the gradation requirements. Processed recycled fill should have a maximum particle size of 4 inches.

Recycled granular fill material is generally not suitable for the top 4 inches of floor slab base rock. We also caution that excavation through recycled material that is placed as structural fill may be difficult. In addition, these excavations may also be prone to raveling and caving.

5.5.1.8 Drain Rock

Drain rock should consist of open-graded, angular, granular material with a maximum particle size of 2 inches. The material should be free of roots, organic material, and other unsuitable material and should have less than 2 percent by dry weight passing the U.S. Standard No. 200 sieve (washed analysis).

5.5.1.9 Stabilization Material

Stabilization material should consist of pit- or quarry-run rock, crushed rock, or crushed gravel and sand that consists of 4- to 6-inch-minus material. It should have less than 5 percent by dry weight passing the U.S. Standard No. 4 sieve and at least two mechanically fractured faces. The material should be free of organic and other deleterious materials. Stabilization material should be placed in one lift and compacted to a firm condition.

Where the stabilization material is used to stabilize soft subgrade beneath pavement or construction haul roads, a geotextile should be placed as a barrier between the soil subgrade and the imported granular material. The geotextile fabric should meet the specifications provided below for subgrade geotextiles. Geotextile is not required where stabilization material is used at the base of utility trenches.

5.5.2 Geotextile Fabric

5.5.2.1 Separate Geotextile Fabric

A separation geotextile fabric can be placed as a barrier between silty subgrade and granular material in staging areas, haul road areas, or in areas of repeated construction traffic. The subgrade geotextile should meet the requirements in OSSC 02320 (Geosynthetics) for subgrade geotextiles and be installed in conformance with OSSC 00350 (Geosynthetic Installation).

5.5.2.2 Drainage Geotextile Fabric

Drain rock and other granular material used for subsurface drains should be wrapped in a geotextile fabric that meets the specifications provided in OSSC 00350 (Geosynthetic Installation) and OSSC 02320 (Geosynthetics) for drainage geotextiles and installed in conformance with OSSC 00350 (Geosynthetic Installation).

5.6 PERMANENT CUT AND FILL SLOPES

Permanent cut and fill slopes in the site soil should be inclined no steeper than 2H:1V. Upslope buildings, access roads, and pavement should be set back a minimum of 5 feet from the crest of such slopes.

5.7 EROSION CONTROL

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

6.0 FOUNDATION SUPPORT RECOMMENDATIONS

Foundation loads were not established at the time of this report. For preliminary design purpose we recommend footings within 30 feet from the top of the existing slope be supported a deep foundation system that transfers the load to the gravel encountered at a depth of approximately 20 feet BGS. Ground improvement such as rammed aggregate piers or DSMC foundations may also be feasible.

Depending on foundation loads, foundations located at a distance greater than 30 feet from the top of the existing slope should be supported on spread footings bearing on the existing native silt and clay, on granular pads overlying the native silt and clay, or on rammed aggregate piers or DSMC foundations. The recommended foundation type for different design loads is presented in Table 3.

Table 3. Recommended Foundation Type Based on Design Load (locations greater than 30 feet from existing top of slope)

Design Load	Recommended Foundation Type
0 to 400 kips	Shallow spread footings bearing on native silt or clay
400 to 600 kips	Shallow spread footings bearing on granular pads overlying native silt or clay
600 to 800 kips	Spread footings bearing on ground improvement or deep foundations

Our recommendations for specific foundation types are presented in the following sections. GeoDesign should be contacted to re-evaluate these recommendations once the final design of the proposed building has been determined.

6.1 SHALLOW SPREAD FOOTINGS

6.1.1 Bearing Capacity

For design loads up to 400 kips, we recommend that shallow spread footings bear on undisturbed native silt and clay. For design loads between 400 and 600 kips, we recommend that shallow spread footings bear on a minimum 24-inch-thick granular pad underlain by undisturbed native silt and clay. Footings bearing on native silt and clay or native silt and clay on structural fill should be proportioned for a maximum allowable soil bearing pressure of 3,500 psf. An allowable bearing capacity of 3,500 psf can also be used for footing bearing on granular pads. This is a net bearing pressure; the weight of the footing and overlying backfill can be ignored in calculating footing sizes. The recommended allowable bearing pressures apply to the total of dead and long-term live loads and may be increased by 50 percent for short-term loads, such as those resulting from wind or seismic forces.

The planned structure can be supported by isolated column and continuous wall footings. We recommend that isolated column and continuous wall footings have minimum widths of 24 and 18 inches, respectively. The bottom of exterior footings and wall footings should be founded at least 18 inches below the lowest adjacent grade. Interior column footings should be founded at least 12 inches below the base of the adjacent floor slab.

6.1.2 Settlement

Based on the anticipated foundation loads, post-construction settlement of new footings founded on shallow spread footings bearing on undisturbed native silt and clay or on granular pads as recommended is anticipated to be less than 1 inch. Differential settlement between similarly

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loaded, newly constructed foundation elements should be approximately one-half of the total settlement. If grading plans or structural loads change, we should be contacted to perform additional settlement analyses.

6.2 RAMMED AGGREGATE PIERS

If foundation elements are located within 30 feet of the existing top of slope or if the proposed design loads for spread footings are greater than 600 kips, we recommend that the proposed building can be supported on spread footings bearing on improved ground. Ground improvement can consist of rammed aggregate piers or DSMC foundations that extend to the underlying gravel. Rammed aggregate piers or DSMC foundations are typically proprietary systems designed and constructed by specialty contractors. We have assumed rammed aggregate piers will be used for design of the proposed building.

We recommend the specialty contractor obtain the structural loads and settlement requirements from the project structural engineer and use this information to design the aggregate pier foundation. The contractor can use the information in this report and, if necessary, should conduct additional explorations if the geotechnical information is insufficient.

Installing the aggregate piers may require drilling through sandy soil in perched groundwater zones. Saturated, sandy soil or gravel below the groundwater table will be prone to raveling and running conditions. If groundwater is encountered, casing might be required to advance the auger excavations for the aggregate piers.

We recommend GeoDesign be allowed to review the final design and proposed installation method for the selected system. A representative of our firm should be present during installation of the aggregate piers to confirm that soil conditions are as anticipated and to observe data collected during installation. We should also observe any test pier installation and load testing that may be performed.

6.2.1 Spread Footings Bearing on Rammed Aggregate Piers

Rammed aggregate piers could result in an allowable bearing pressure of 6,000 to 7000 psf. A one-third increase in allowable bearing pressure is also typical for such systems when resisting short-term loads such as wind and seismic forces. Rammed aggregate pier design should consider foundation settlement mitigation of adjacent building foundations.

6.2.2 Settlement

Shallow foundations bearing on rammed aggregate piers should experience post-construction settlement of less than 1 inch. Differential settlement of up to one-half of the total settlement magnitude can be expected between adjacent footings with similar loads. We expect settlement will occur during construction as loads are applied.

6.3 DSMCs

DSMCs are constructed by mixing cement slurry with the on-site soil to form columns of improved ground that extend down to a suitable bearing stratum. DSMCs are created using a specialty drill rig that injects cement slurry into the ground during the drilling process. Paddles along the shaft mix the soil and cement slurry together until a relatively uniform column of soil and cement is formed. DSMCs typically vary in diameter between 36 and 60 inches. Spoils generated during the installation can be used on site as structural fill or hauled off site. Typically, the amount of spoils generated is approximately 30 percent of the volume of the cement slurry.

DSMCs should extend through the fill and bear on the very dense gravel. Based on explorations at the site, the gravel contact is located at a depth of approximately 20 feet BGS. Based on our experience, spread footings supported on DSMC ground improvement can be sized using an allowable bearing pressure of 6,000 psf. This can be increased by one-third when considering transient loads such as wind and seismic forces.

We recommend that a 12-inch-thick layer of compacted angular crushed rock be placed between the top of the DSMC ground improvement and the bottom of footings. Specific depths of the DSMCs will be dependent on individual column loads.

The system can be designed such that the total foundation settlement will not exceed 1 inch for spread footings bearing on DSMC ground improvement. Differential settlement of up to ½ inch is typical between similarly loaded footings.

6.4 CAST-IN-PLACE CONCRETE PILES

Cast-in-place concrete piles can be used to transfer foundation loads to the dense gravel unit encountered at a depth of approximately 20 feet BGS.

6.4.1 Axial Capacity

Axial downward and uplift capacity profiles for 24- and 30-inch-diameter cast-in-place concrete piles are shown on Figure 3. The allowable capacity is based on a safety factor of 3 for compression.

We can provide the design team with lateral pile response curves when the pile/shaft size has been selected.

6.4.2 Quality Control

The drilled concrete piles should be installed with suitable alignment tolerances. Lateral alignment should be within tolerances determined by the structural engineer, considering the structural design of the pile/shaft connection with the structure. Steel reinforcement cages should be installed with a vertical alignment within 2 percent of plumb or as required by the structural engineer.

Due to the presence of granular material and groundwater, the use of full-depth casing, drilling mud, or a combination thereof may be required to maintain an open hole with a clean base. If continuous flight auger methods of installation are employed, the shafts should be drilled in a single stroke without delays during the installation process.

The base of the excavated pile/shaft cavity should be relatively free of excess debris resulting from shaft excavation. This will require a cleanout barrel or bucket to be turned at the base of the excavation when the desired design depths are achieved.

Quality of pile/shaft construction will be critical to provide acceptable settlement behavior. At a minimum, we recommend the following quality control measures:

- The hole bottom should be thoroughly cleaned to ensure that all loose material has been removed.
- A qualified technician should be present during construction to verify subsurface conditions are as interpreted and the general design intentions are met during construction.

Based on the anticipated foundation loads, post-construction settlement of cast-in-place concrete piles is anticipated to be less than ½ inch.

6.5 LATERAL RESISTANCE

Lateral loads can be resisted by passive earth pressure on sides of the footings and by friction on the base of the footings. We recommend that a friction coefficient of 0.35 be used to compute the frictional resistance for footings bearing on native site soil and a friction coefficient of 0.45 be used to compute the frictional resistance for footings bearing on granular pads or rammed aggregate piers. Frictional resistance should be ignored beneath pile caps.

An equivalent fluid unit weight of 350 pcf is recommended to compute passive earth pressure acting on footings constructed in direct contact with compacted structural fill or native soil. This value is based on the assumptions that the adjacent confining structural fill or native soil is level and that groundwater remains below the base of the footing. The top 1 foot of soil should be neglected when calculating lateral earth pressures unless the foundation area is covered with pavement or is inside a building.

6.6 DIFFERENTIAL SETTLEMENT

Since the building may be supported on more than one foundation type, differential settlement between foundation system may approach 1 inch. Drilled shafts that bear on gravel may not experience much settlement, while spread footings could experience up to 1 inch of total settlement. We recommend that GeoDesign be consulted to evaluate foundation settlement when foundation loads have been established.

6.7 SLABS ON GRADE

Satisfactory subgrade support for building floor slabs supporting floor loads of up to 100 psf can be obtained, provided the subgrade is prepared in accordance with the "Site Preparation" section. A minimum 6-inch-thick layer of base rock should be placed and compacted over the prepared subgrade to assist as a capillary break. The base rock should be crushed rock or crushed gravel and sand meeting the requirements outlined in the "Structural Fill" section. The imported granular material should be placed in one lift and compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D1557. Floor slab base rock should be replaced if it becomes contaminated with excessive fines (greater than 5 percent by dry weight passing the U.S. Standard No. 200 sieve).

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

7.0 PERMANENT RETAINING STRUCTURES

Permanent retaining structures free to rotate slightly around the base should be designed for active earth pressures using an equivalent fluid unit pressure of 35 pcf. If retaining walls are restrained against rotation during backfilling, they should be designed for an at-rest earth pressure of 55 pcf. These values are based on the assumption that (1) the backfill is level, (2) the backfill is drained, and (3) the wall is less than 10 feet in height. Seismic lateral forces can be calculated using a dynamic force equal to $7H^2$ pounds per linear foot of wall, where H is the wall height. The seismic force should be applied as a distributed load with the centroid located at 0.6H from the wall base. Footings for retaining walls should be designed as recommended for shallow foundations.

Drains consisting of a perforated drainpipe wrapped in a geotextile filter should be installed behind retaining walls. The pipe should be embedded in a zone of coarse sand or gravel containing less than 2 percent by dry weight passing the U.S. Standard No. 200 sieve and should outlet to a suitable discharge.

8.0 DRAINAGE CONSIDERATIONS

8.1 GENERAL

We recommend that roof drains be connected to a tightline leading to storm drain facilities. Pavement surfaces and open space areas should be sloped such that surface water runoff is collected and routed to suitable discharge points. We also recommend that ground surfaces adjacent to the building be sloped to facilitate positive drainage away from the building.

8.2 TEMPORARY

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

8.3 SURFACE

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

EXHIBIT B

9.0 SEISMIC CONSIDERATIONS

9.1 DESIGN PARAMETERS

Seismic design will be performed in accordance with ASCE 7-16, which is prescribed by the 2019 SOSSC. A Site Class C designation can be used to compute design levels of ground shaking. We obtained these parameters from the ASCE 7 hazard tool (ASCE, 2018).

Table 4. Seismic Design Parameters

Parameter	Short Period (T_s = 0.2 second)	1 Second Period (T_1 = 1.0 second)
MCE Spectral Acceleration, S	S_s = 0.888 g	S_1 = 0.393 g
Site Class	C	
Site Coefficient, F	F_a = 1.2	F_v = 1.5
Adjusted Spectral Acceleration, S_M	S_{MS} = 1.065 g	S_{M1} = 0.590 g
Design Spectral Response Acceleration Parameters, S_D	S_{DS} = 0.710 g	S_{D1} = 0.393 g
Design Spectral PGA (2/3 MCE)	0.28 g	

Liquefaction is not considered a hazard under design levels of ground shaking.

The SOSSC requires that buildings over six stories in height or buildings with an aggregate floor area of 60,000 feet or more will require a site-specific hazard evaluation. Once the final building size has been determined, GeoDesign can provide a site-specific hazard evaluation of the site if required.

9.2 SEISMIC HAZARDS

9.2.1 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 soil with low silt and clay content is the most susceptible to liquefaction. Silty soil with low plasticity is moderately susceptible to liquefaction under relatively higher levels of ground shaking.

Based on subsurface conditions, laboratory testing, and analysis, it is our opinion that the subsurface soil at the site is not susceptible to liquefaction.

9.2.2 Lateral Spreading

Lateral spreading is a liquefaction-related seismic hazard and occurs on gently sloping or flat sites underlain by liquefiable sediment adjacent to an open face (such as riverbanks). Liquefied soil adjacent to open faces will tend to flow, resulting in surface cracking and lateral displacement towards the open face. The magnitude of lateral spread decreases with distance from the open face. Because the soil at the site is not liquefiable, lateral spreading is not considered a site hazard under design levels of ground shaking.

9.2.3 Fault Rupture

According to USGS, faults are not mapped beneath the site. The closest mapped fault is the Portland Hills fault, which is mapped approximately 0.17 mile to the east. Consequently, it is our opinion that the probability of surface fault rupture beneath the site is low.

10.0 OBSERVATION OF CONSTRUCTION

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

11.0 LIMITATIONS

We have prepared this report for use by Columbia Capital Group LLC and their consultants for this project. The data and report can be used for estimating purposes, but our report, conclusions, and interpretations should not be construed as a warranty of the subsurface conditions and are not applicable to other sites.

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

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

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

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

EXHIBIT B

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

Sincerely,

GeoDesign, Inc.

Gregory J. Schaertl, P.E.
Senior Project Engineer

Brett A. Shipton, P.E., G.E.
Principal Engineer

REGISTERED PROFESSIONAL ENGINEER
56207
OREGON
July 11, 2000
BRETT A. SHIPTON
EXPIRES: 6/30/20

REFERENCES

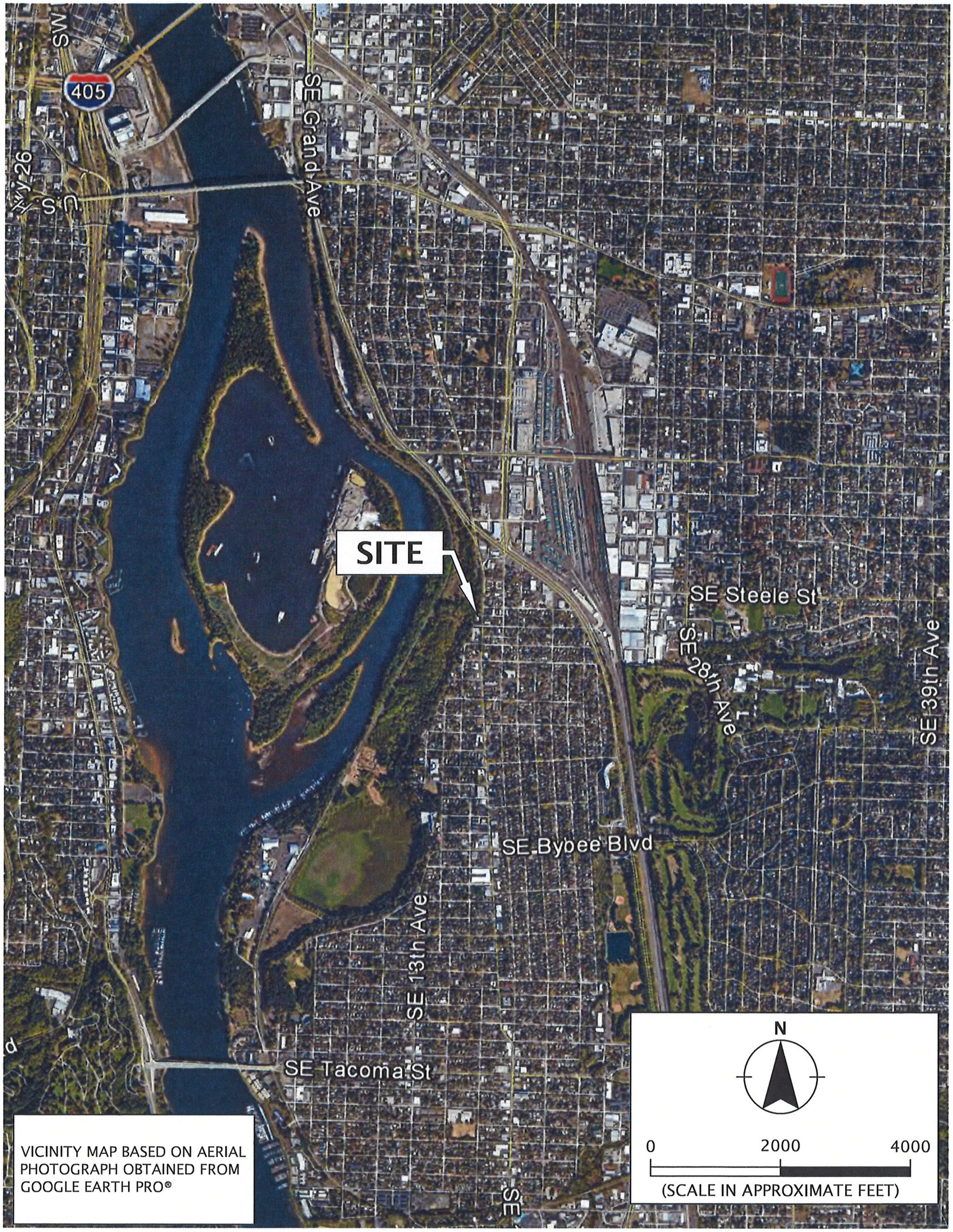
ASCE, 2018. *ASCE 7 Hazard Tool.* Obtained from website https://asce7hazardtool.online/. Accessed on March 19, 2020.

Snyder, Daniel T., 2008. *Estimated Depth to Ground Water and Configuration of the Water Table in the Portland, Oregon Area.* Prepared in cooperation with the City of Portland, the City of Gresham, Clackamas County's Water Environment Services, and Multnomah County.
U.S. Geological Survey Scientific Investigations Report 2008-5059.

EXHIBIT B

FIGURES

EXHIBIT B



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

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VICINITY MAP

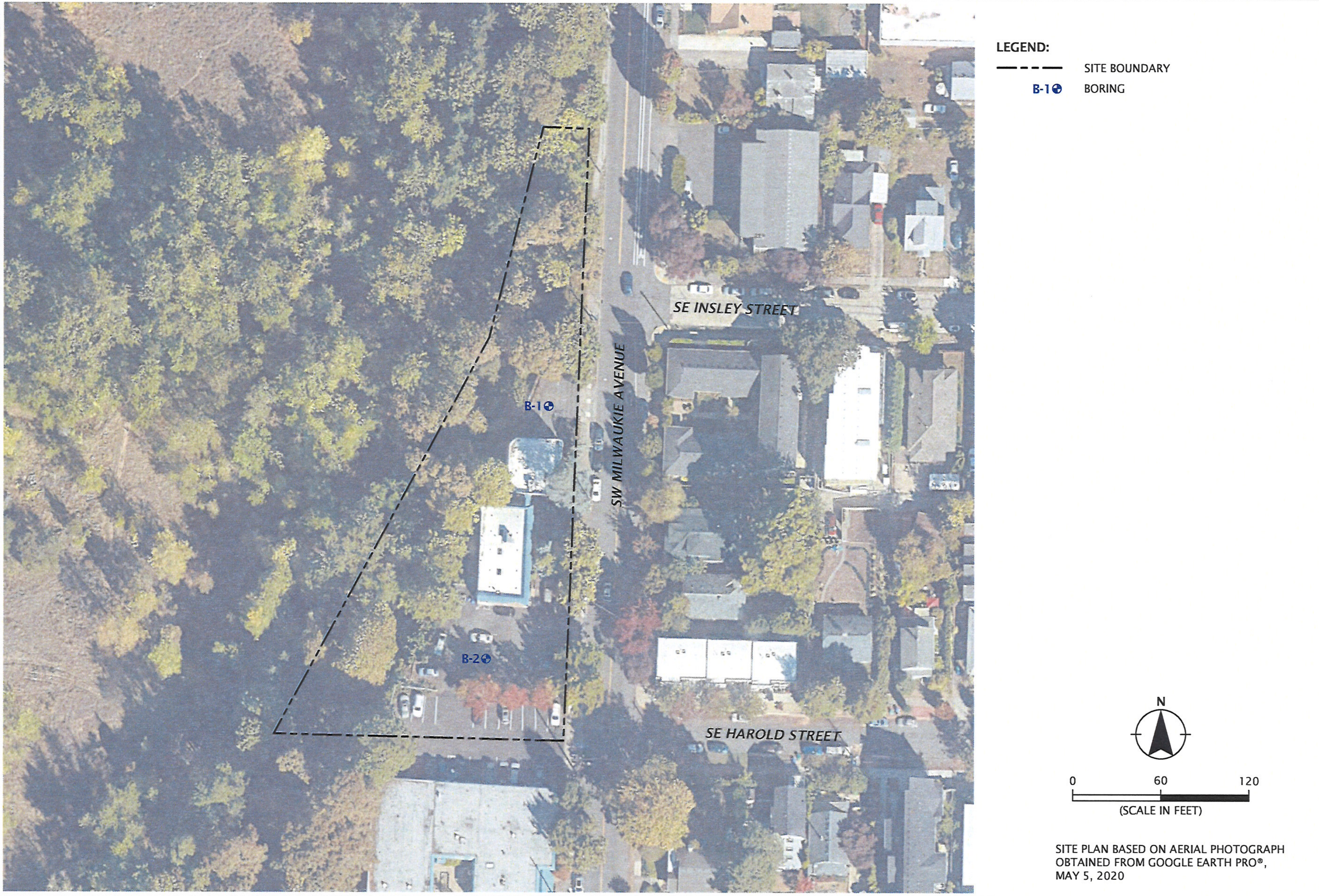
MAY 2020

PROPOSED RM4 DEVELOPMENT
PORTLAND, OR

FIGURE 1

Printed By: mmiller | Print Date: 5/5/2020 8:26:11 AM
File Name: J:\A-D\CCGPDX\CCGPDX-2\CCGPDX-2-01\Figures\CAD\CCGPDX-2-01-VM01.dwg | Layout: FIGURE 1

EXHIBIT B

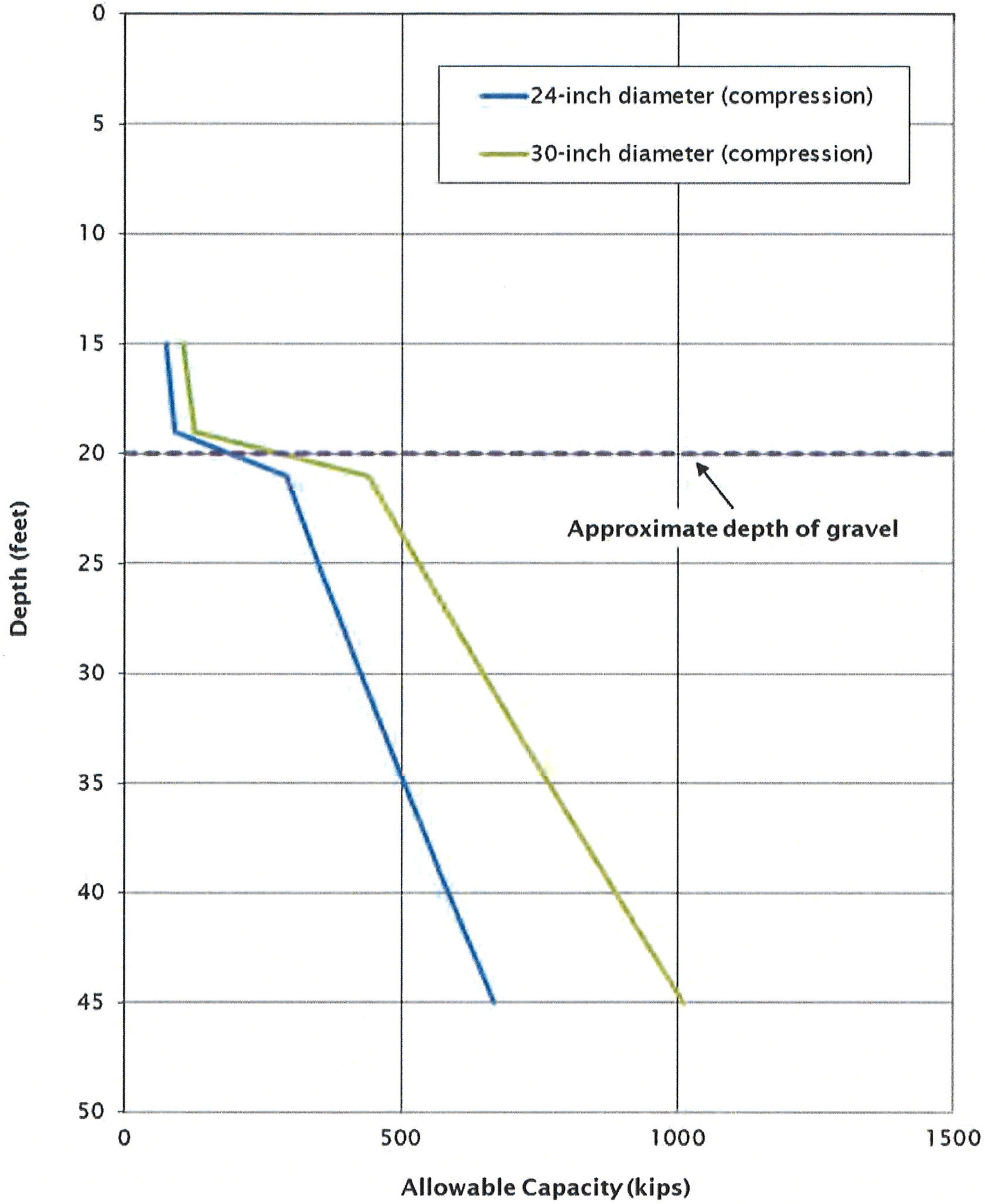


SITE PLAN BASED ON AERIAL PHOTOGRAPH
OBTAINED FROM GOOGLE EARTH PRO®,
MAY 5, 2020

GEODESIGN INC AN NV5 COMPANY	CCGPDX-2-01	SITE PLAN	
	MAY 2020	PROPOSED RM4 DEVELOPMENT PORTLAND, OR	FIGURE 2

Printed By: mmiller | Print Date: 5/5/2020 8:26:17 AM
File Name: J:\A-D\CCGPDX\CCGPDX-2\CCGPDX-2-01\Figures\CAD\CCGPDX-2-01-SP01.dwg | Layout: FIGURE 2

EXHIBIT B



Notes:
Safety Factor = 3.0 (compression)
Minimum embedment of 5 feet into gravel

CCGPDX-2-01-F3.docx Print Date: 5/14/20

GEODESIGN INC AN NV5 COMPANY	CCGPDX-2-01	**AXIAL CAPACITY FOR CAST-IN-PLACE CONCRETE PILES**	
	MAY 2020	PROPOSED RM4 DEVELOPMENT PORTLAND, OR	**FIGURE 3**

EXHIBIT B

APPENDIX A

EXHIBIT B

APPENDIX A

FIELD EXPLORATIONS

GENERAL

We explored subsurface conditions at the site by drilling two borings (B-1 and B-2) to depths of 31.5 and 35.1 feet BGS. Drilling services were provided by Western States Soil Conservation, Inc. of Hubbard, Oregon, using hollow-stem auger and mud rotary drilling methods. The exploration logs are presented in this appendix. The locations of the explorations are shown on Figure 2. The exploration locations were determined in the field by pacing and taping from surveyed existing site features. Their locations should be considered accurate only to the degree implied by the methods used.

SOIL SAMPLING

Samples were collected from the borings using a 1½-inch-inside diameter (SPT) split-spoon sampler in general accordance with ASTM D1586. The split-spoon samplers were driven into the soil with a 140-pound hammer free-falling 30 inches. The samplers were driven a total distance of 18 inches. The number of blows required to drive the sampler the final 12 inches is recorded on the boring logs, unless otherwise noted. Relatively undisturbed samples were obtained using a standard Shelby tube in general accordance with ASTM D1587. Sampling methods and intervals are shown on the exploration logs.

The average efficiency of the automatic SPT hammer used by Western States Soil Conservation, Inc. was 82.2 percent. The calibration testing results are presented at the end of this appendix.

SOIL CLASSIFICATION

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

LABORATORY TESTING

CLASSIFICATION

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

MOISTURE CONTENT

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

EXHIBIT B

ATTERBERG LIMITS

Atterberg limits testing was performed on a select soil sample in general accordance with ASTM D4318. Atterberg limits include the liquid limit, plastic limit, and the plasticity index of soil. These index properties are used to classify soil and for correlation with other engineering properties of soil. The test results are presented in this appendix.

PARTICLE-SIZE ANALYSIS

Particle-size analysis was performed on select soil samples in general accordance with ASTM D1140. This test is a quantitative determination of the amount of material finer than the U.S. Standard No. 200 sieve expressed as a percentage of soil weight. The test results are presented in this appendix.

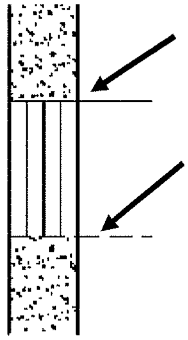
CONSOLIDATION TESTING

Consolidation testing was performed on a select relatively undisturbed soil sample in general accordance with ASTM D2435. The test measures the volume change of a soil sample under predetermined loads. The test results are presented in this appendix.

EXHIBIT B

SYMBOL	SAMPLING DESCRIPTION
▮	Location of sample collected in general accordance with ASTM D1586 using Standard Penetration Test with recovery
▮	Location of sample collected using thin-wall Shelby tube or Geoprobe® sampler in general accordance with ASTM D1587 with recovery
▤	Location of sample collected using Dames & Moore sampler and 300-pound hammer or pushed with recovery
▤	Location of sample collected using Dames & Moore sampler and 140-pound hammer or pushed with recovery
▮	Location of sample collected using 3-inch-O.D. California split-spoon sampler and 140-pound hammer with recovery
⊠	Location of grab sample
▒	Rock coring interval
▽	Water level during drilling
▼	Water level taken on date shown

Graphic Log of Soil and Rock Types



Observed contact between soil or rock units (at depth indicated)

Inferred contact between soil or rock units (at approximate depths indicated)

GEOTECHNICAL TESTING EXPLANATIONS

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

ENVIRONMENTAL TESTING EXPLANATIONS

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

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EXPLORATION KEY

TABLE A-1

EXHIBIT B

RELATIVE DENSITY - COARSE-GRAINED SOIL

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

CONSISTENCY - FINE-GRAINED SOIL

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

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

MOISTURE CLASSIFICATION

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

ADDITIONAL CONSTITUENTS

Secondary granular components or other materials such as organics, man-made debris, etc.

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

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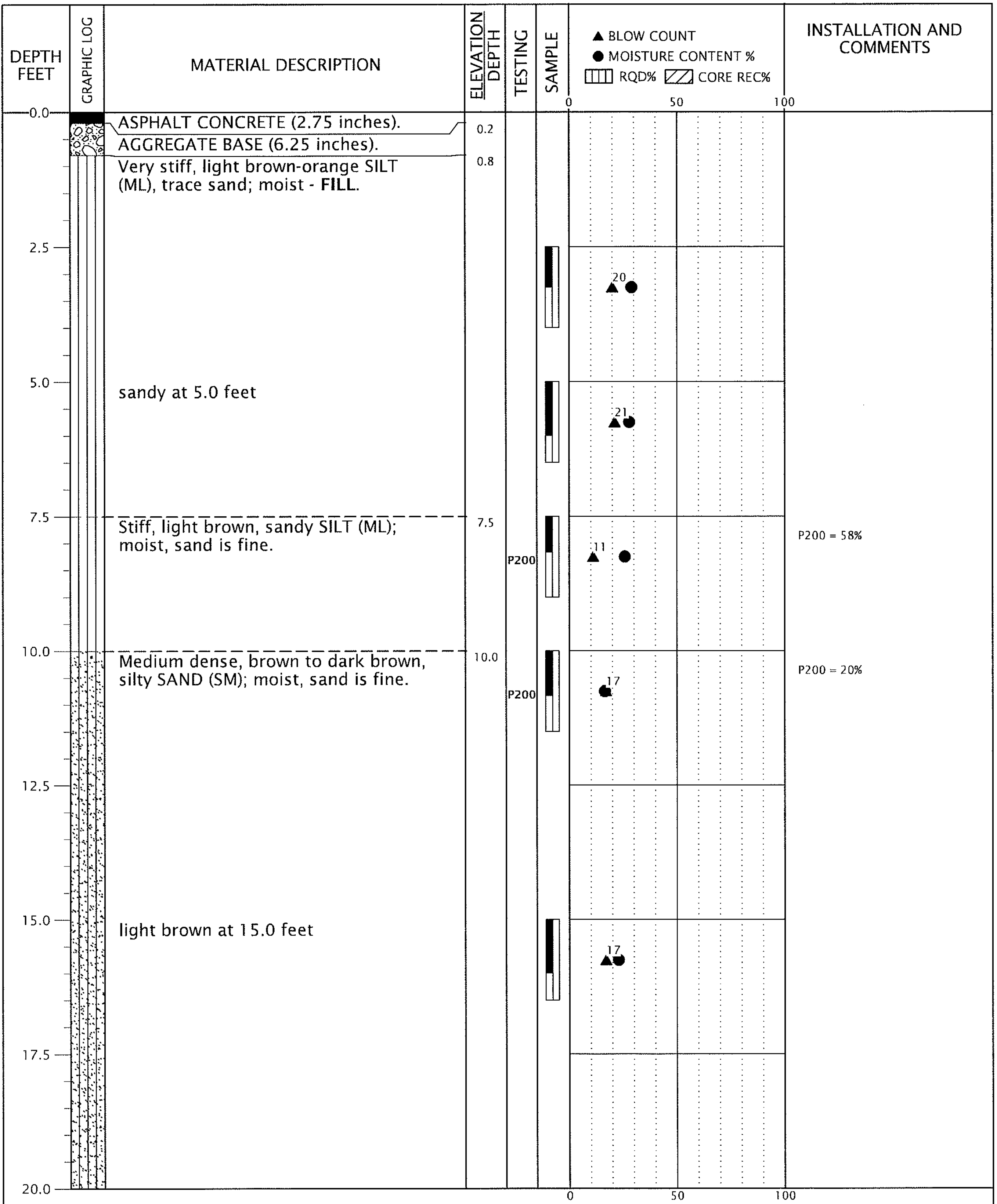
SOIL CLASSIFICATION SYSTEM

TABLE A-2

EXHIBIT B



DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	BLOW COUNT	INSTALLATION AND COMMENTS
0.0	ASPHALT CONCRETE (2.75 inches).	0.2			
	AGGREGATE BASE (6.25 inches).	0.8			
	Very stiff, light brown-orange SILT (ML), trace sand; moist - **FILL**.				
2.5				20	
5.0	sandy at 5.0 feet			21	
7.5	Stiff, light brown, sandy SILT (ML); moist, sand is fine.	7.5	P200	11	P200 = 58%
10.0	Medium dense, brown to dark brown, silty SAND (SM); moist, sand is fine.	10.0	P200	17	P200 = 20%
12.5					
15.0	light brown at 15.0 feet			17	
17.5					
20.0					

DRILLED BY: Western States Soil Conservation, Inc.

LOGGED BY: J. Heidgerken

COMPLETED: 03/11/20

BORING METHOD: mud rotary (see document text)

BORING BIT DIAMETER: 4 7/8 inches

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MAY 2020

BORING B-1

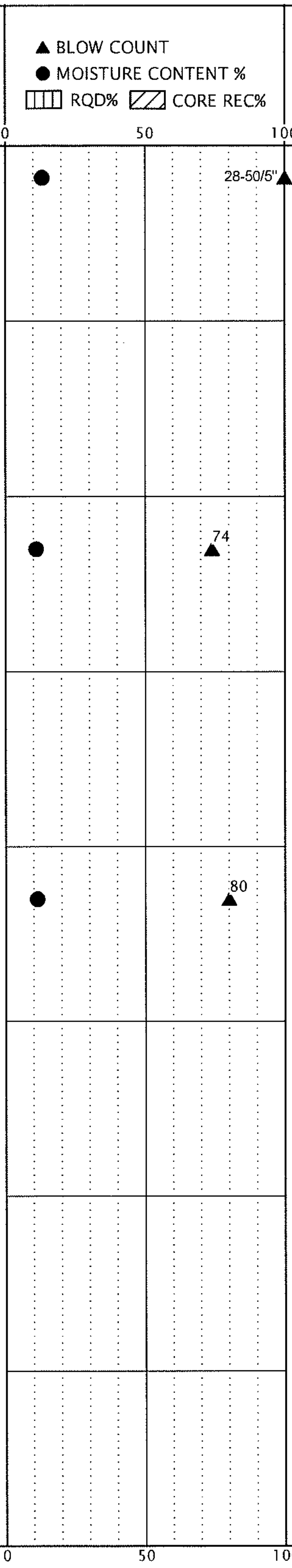
PROPOSED RM4 DEVELOPMENT
PORTLAND, OR

FIGURE A-1

BORING LOG - GDI-NV5 - 1 PER PAGE CCGPDX-2-01-B1_2.GPJ GDI_NV5.GDT PRINT DATE: 5/13/20:KM:KT

EXHIBIT B

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	▲ BLOW COUNT	INSTALLATION AND COMMENTS
20.0	dense to very dense at 20.0 feet					
	Very dense, gray GRAVEL with sand (GP), trace silt; moist.	20.5		▮	28-50/5"	
22.5						
25.0	Very dense, brown-gray GRAVEL with silt and sand (GP-GM), trace clay; moist.	25.0		▮	74	
27.5						
30.0				▮	80	
	Exploration completed at a depth of 31.5 feet. Hammer efficiency factor is 82.2 percent.	31.5				Surface elevation was not measured at the time of exploration.
32.5						
35.0						
37.5						
40.0						



DRILLED BY: Western States Soil Conservation, Inc.

LOGGED BY: J. Heidgerken

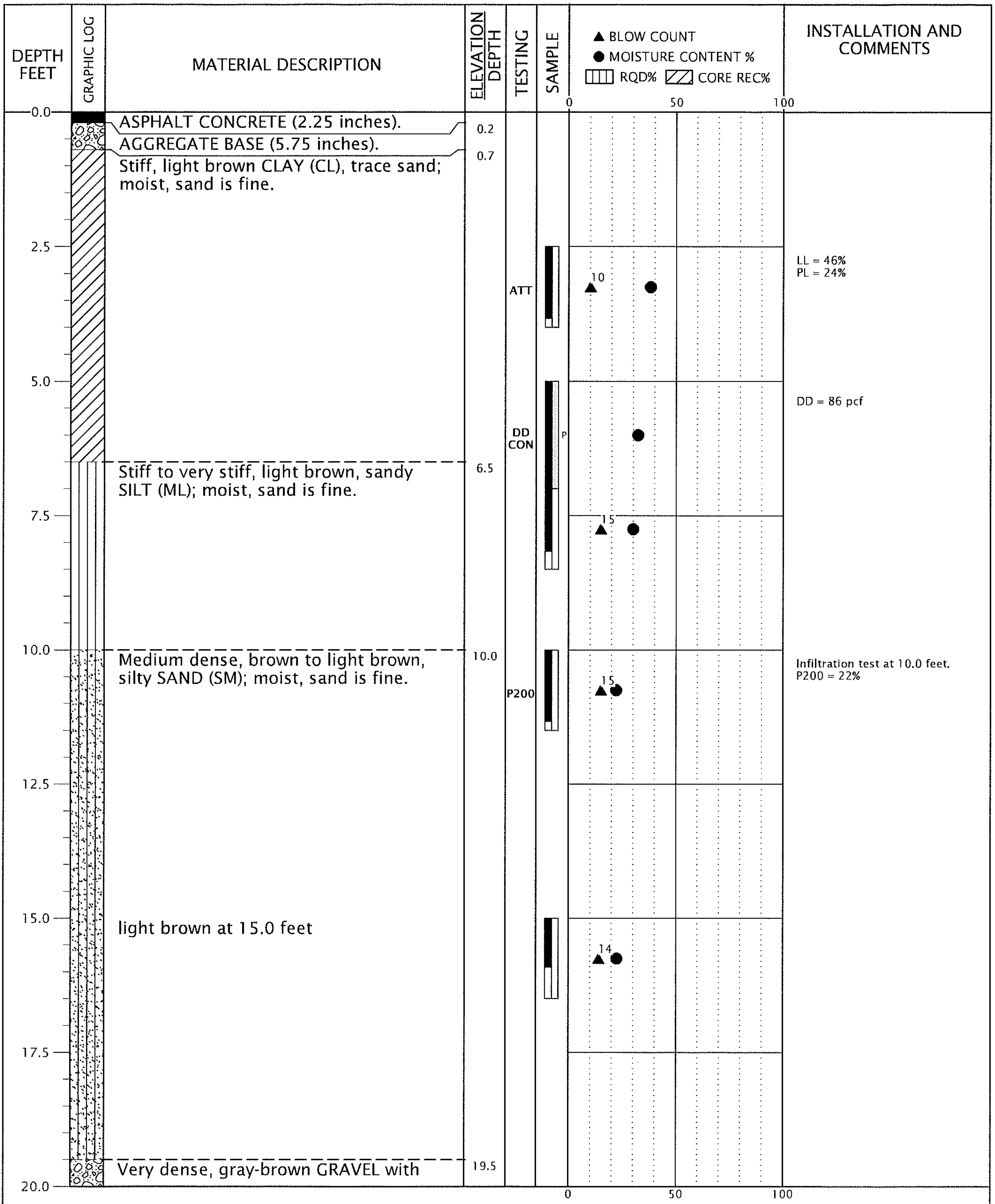
COMPLETED: 03/11/20

BORING METHOD: mud rotary (see document text)

BORING BIT DIAMETER: 4 7/8 inches

GEODESIGN INC AN NV5 COMPANY	CCGPDX-2-01	**BORING B-1** (continued)	
	MAY 2020	PROPOSED RM4 DEVELOPMENT PORTLAND, OR	**FIGURE A-1**

EXHIBIT B



MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	▲ BLOW COUNT	INSTALLATION AND COMMENTS
ASPHALT CONCRETE (2.25 inches).	0.2			
AGGREGATE BASE (5.75 inches).	0.7			
Stiff, light brown CLAY (CL), trace sand; moist, sand is fine.				
		ATT	10	LL = 46% PL = 24%
		DD CON		DD = 86 pcf
Stiff to very stiff, light brown, sandy SILT (ML); moist, sand is fine.	6.5			
			15	
Medium dense, brown to light brown, silty SAND (SM); moist, sand is fine.	10.0	P200	15	Infiltration test at 10.0 feet. P200 = 22%
light brown at 15.0 feet			14	
Very dense, gray-brown GRAVEL with	19.5			

DRILLED BY: Western States Soil Conservation, Inc.
LOGGED BY: J. Heidgerken
COMPLETED: 03/11/20

BORING METHOD: mud rotary and hollow-stem auger (see document text)
BORING BIT DIAMETER: 4 7/8 inches and 10 inches

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MAY 2020

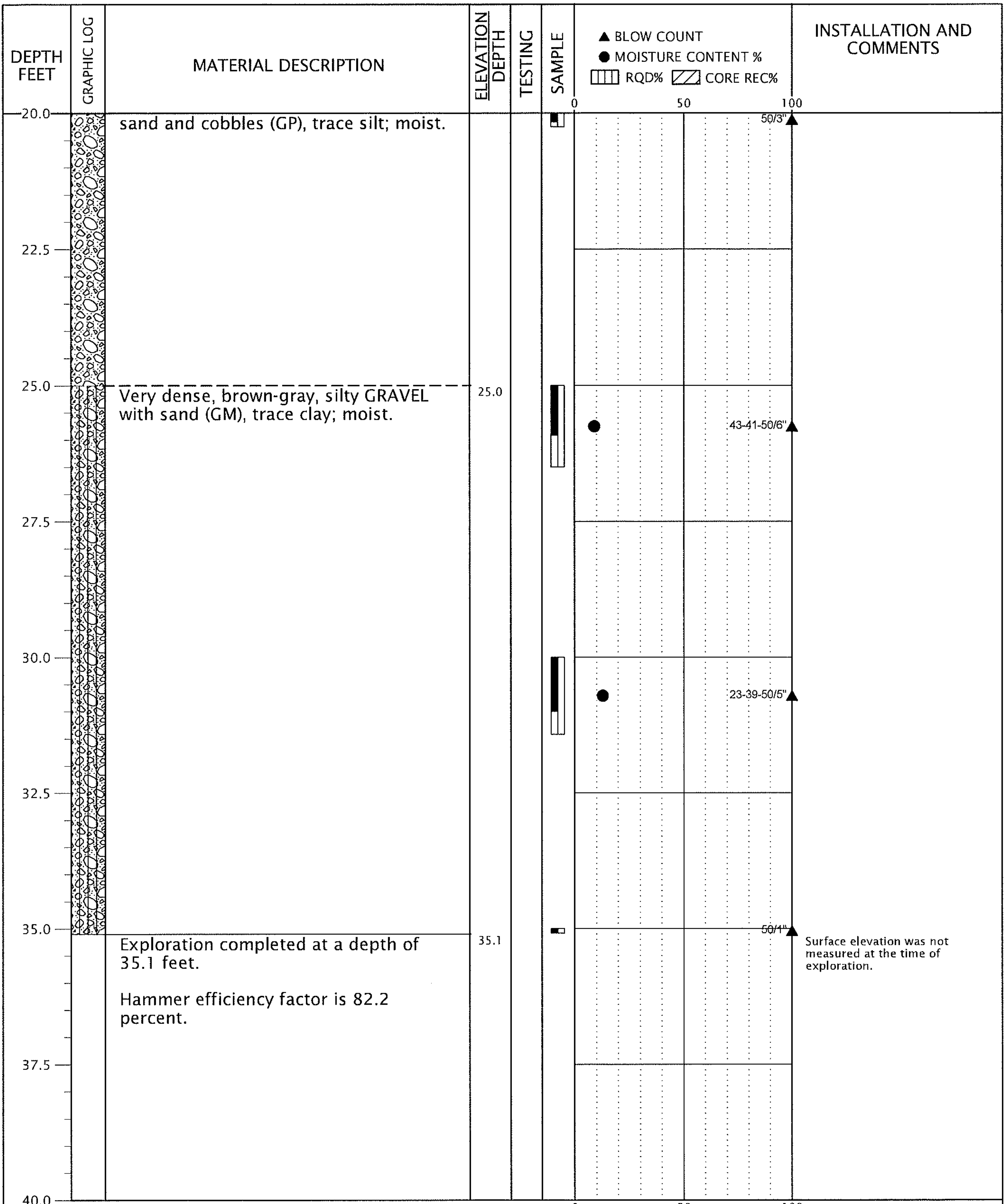
BORING B-2

PROPOSED RM4 DEVELOPMENT
PORTLAND, OR

FIGURE A-2

BORING LOG - GDI-NV5 - 1 PER PAGE CCGPDX-2-01-B1_2.GPJ GDI_NV5.GDT PRINT DATE: 5/13/20:KM:KT

EXHIBIT B



DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	BLOW COUNT	INSTALLATION AND COMMENTS
20.0	sand and cobbles (GP), trace silt; moist.		50/3"	
25.0	Very dense, brown-gray, silty GRAVEL with sand (GM), trace clay; moist.	25.0	43-41-50/6"	
30.0			23-39-50/5"	
35.0	Exploration completed at a depth of 35.1 feet. Hammer efficiency factor is 82.2 percent.	35.1	50/1"	Surface elevation was not measured at the time of exploration.

DRILLED BY: Western States Soil Conservation, Inc.

LOGGED BY: J. Heidgerken

COMPLETED: 03/11/20

BORING METHOD: mud rotary and hollow-stem auger (see document text)

BORING BIT DIAMETER: 4 7/8 inches and 10 inches

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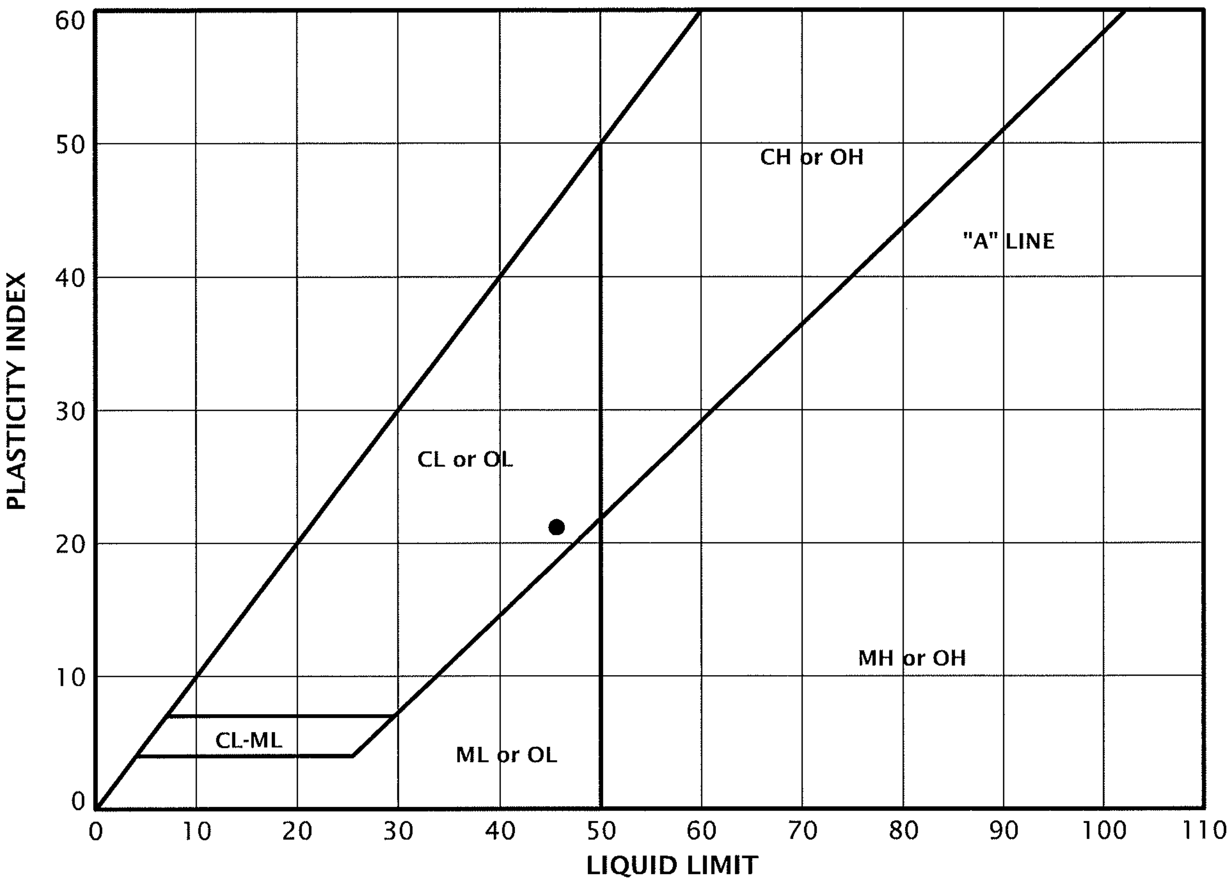
MAY 2020

BORING B-2
(continued)

PROPOSED RM4 DEVELOPMENT
PORTLAND, OR

FIGURE A-2

EXHIBIT B



KEY	EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	MOISTURE CONTENT (PERCENT)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
●	B-2	2.5	38	46	24	22

ATTERBERG_LIMITS 7 CCGPDX-2-01-B1_2.GPJ GEODESIGN.GDT PRINT DATE: 5/1/20:KM

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CCGPDX-2-01

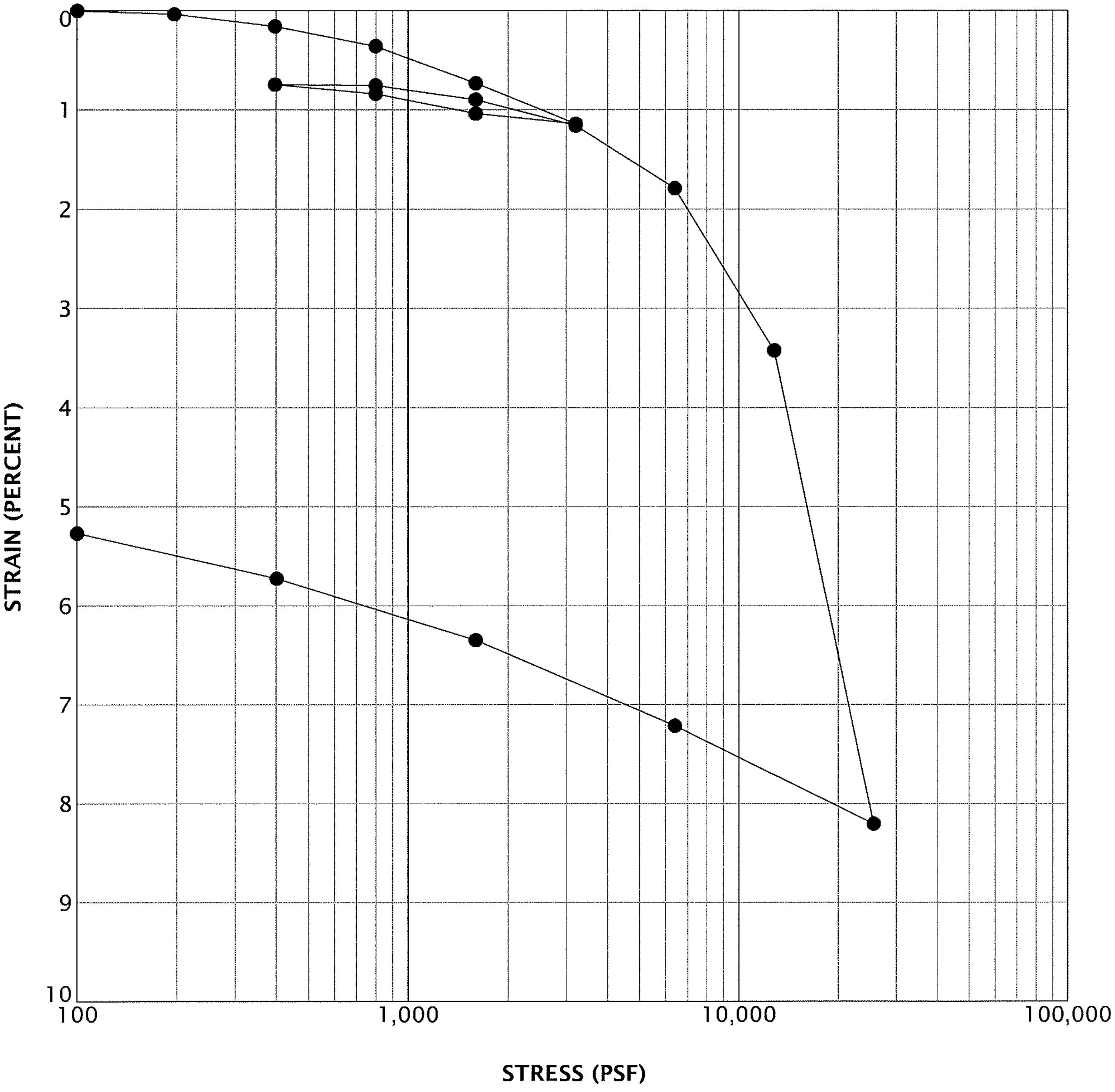
MAY 2020

ATTERBERG LIMITS TEST RESULTS

PROPOSED RM4 DEVELOPMENT
PORTLAND, OR

FIGURE A-3

EXHIBIT B



KEY	EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	MOISTURE CONTENT (PERCENT)	DRY DENSITY (PCF)
●	B-2	5.0	32	86

CONSOL_STRAIN_100K CCGPDX-2-01-B1_2.GPJ GEODESIGN.GDT PRINT DATE: 5/14/20:KT

GEODESIGN INC AN NV5 COMPANY	CCGPDX-2-01	**CONSOLIDATION TEST RESULTS**	
	MAY 2020	PROPOSED RM4 DEVELOPMENT PORTLAND, OR	**FIGURE A-4**

EXHIBIT B

SAMPLE INFORMATION			MOISTURE CONTENT (PERCENT)	DRY DENSITY (PCF)	SIEVE			ATTERBERG LIMITS		
EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	ELEVATION (FEET)			GRAVEL (PERCENT)	SAND (PERCENT)	P200 (PERCENT)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
B-1	2.5		29							
B-1	5.0		28							
B-1	7.5		26				58			
B-1	10.0		16				20			
B-1	15.0		23							
B-1	20.0		13							
B-1	25.0		11							
B-1	30.0		11							
B-2	2.5		38					46	24	22
B-2	5.0		32	86						
B-2	7.0		30							
B-2	10.0		22				22			
B-2	15.0		22							
B-2	25.0		9							
B-2	30.0		13							

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CCGPDX-2-01

MAY 2020

SUMMARY OF LABORATORY DATA

PROPOSED RM4 DEVELOPMENT
PORTLAND, OR

FIGURE A-5

EXHIBIT B

Pile Dynamics, Inc.
SPT Analyzer Results

RIG#1
PDA-S Ver. 2017.22 - Printed: 1/4/2019

Summary of SPT Test Results

Project: WSSC-8-04, Test Date: 12/27/2018

EMX: Maximum Energy

ETR: Energy Transfer Ratio - Rated

Start Depth ft	Final Depth ft	**N Value**	**N60 Value**	Average EMX ft-lb	Average ETR %
25.00	26.50	**0**	**0**	0.00	0.0
30.00	31.50	**31**	**42**	267.77	76.5
35.00	36.50	**34**	**46**	305.69	87.3
40.00	41.50	**0**	**0**	0.00	0.0
			Overall Average Values:	287.61	82.2
			Standard Deviation:	32.43	9.3
			Overall Maximum Value:	387.60	110.7
			Overall Minimum Value:	234.46	67.0

EXHIBIT B

APPENDIX B

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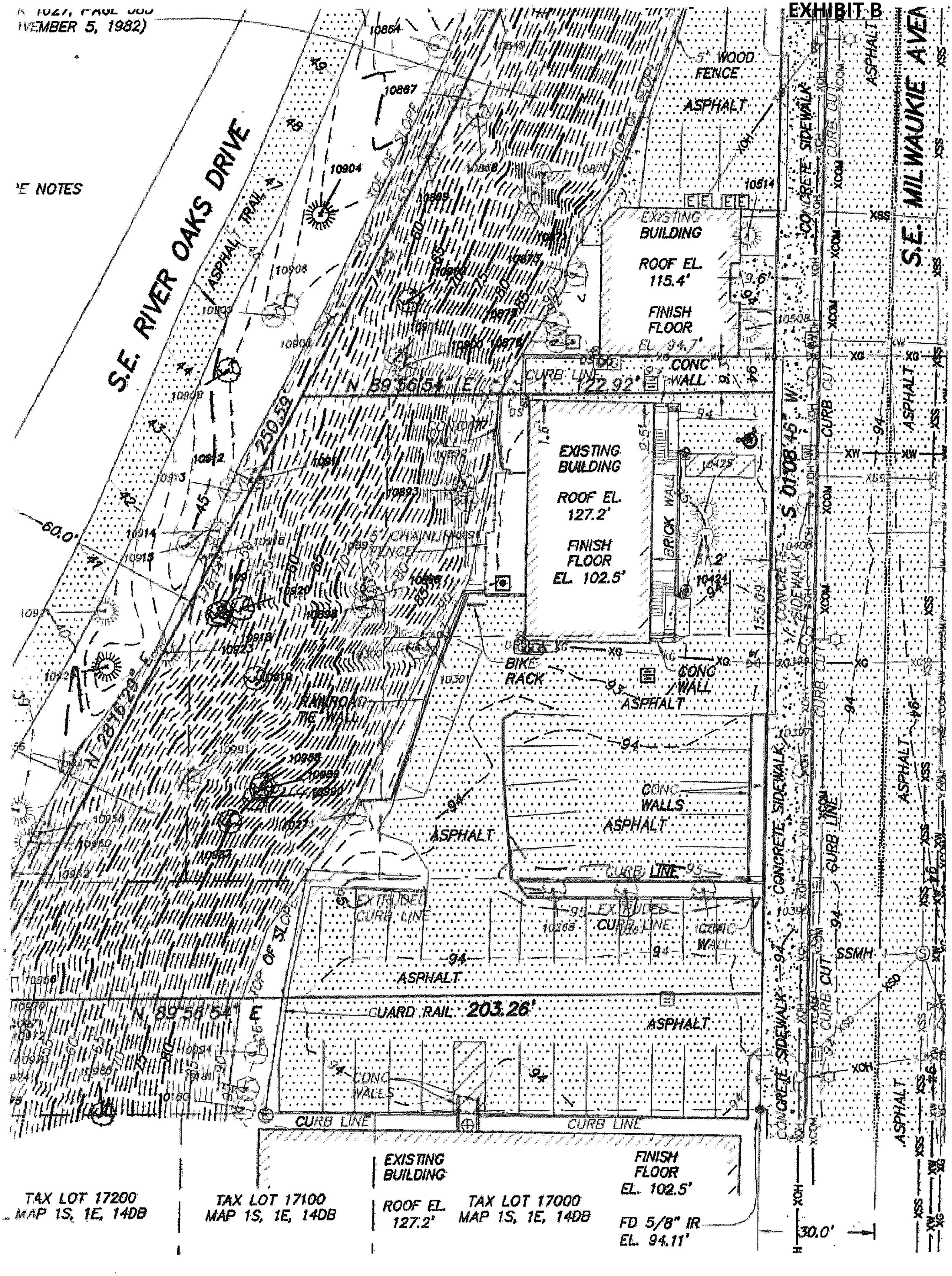
SURVEYOR'S PLOT MAP OF SITE PROVIDED BY COLUMBIA CAPITAL GROUP

EXHIBIT B



K 1027, PAGE 585
VEMBER 5, 1982)
E NOTES
S.E. RIVER OAKS DRIVE
ASPHALT TRAIL
S.E. MILWAUKIE AVE
5' WOOD FENCE
ASPHALT
EXISTING BUILDING
ROOF EL. 115.4'
FINISH FLOOR EL. 94.7'
CONC WALL
CURB LINE
N 89°56'54" E
122.92'
EXISTING BUILDING
ROOF EL. 127.2'
FINISH FLOOR EL. 102.5'
BRICK WALL
5' CHAINLINK FENCE
CONCRETE SIDEWALK
CURB CUT
S 01°08'46" W
155.09
250.59
N 28°16'29" E
60.0'
RAILROAD TIE WALL
BIKE RACK
CONC WALLS
TOP OF SLOPE
TOE OF SLOPE
EXTRUDED CURB LINE
GUARD RAIL
N 89°56'54" E
203.26'
SSMH
CONC WALLS
CURB LINE
EXISTING BUILDING
ROOF EL. 127.2'
FINISH FLOOR EL. 102.5'
TAX LOT 17200 MAP 1S, 1E, 14DB
TAX LOT 17100 MAP 1S, 1E, 14DB
TAX LOT 17000 MAP 1S, 1E, 14DB
FD 5/8" IR EL. 94.11'
30.0'
XSS
XW
XG
XOH
XCOM

EXHIBIT B

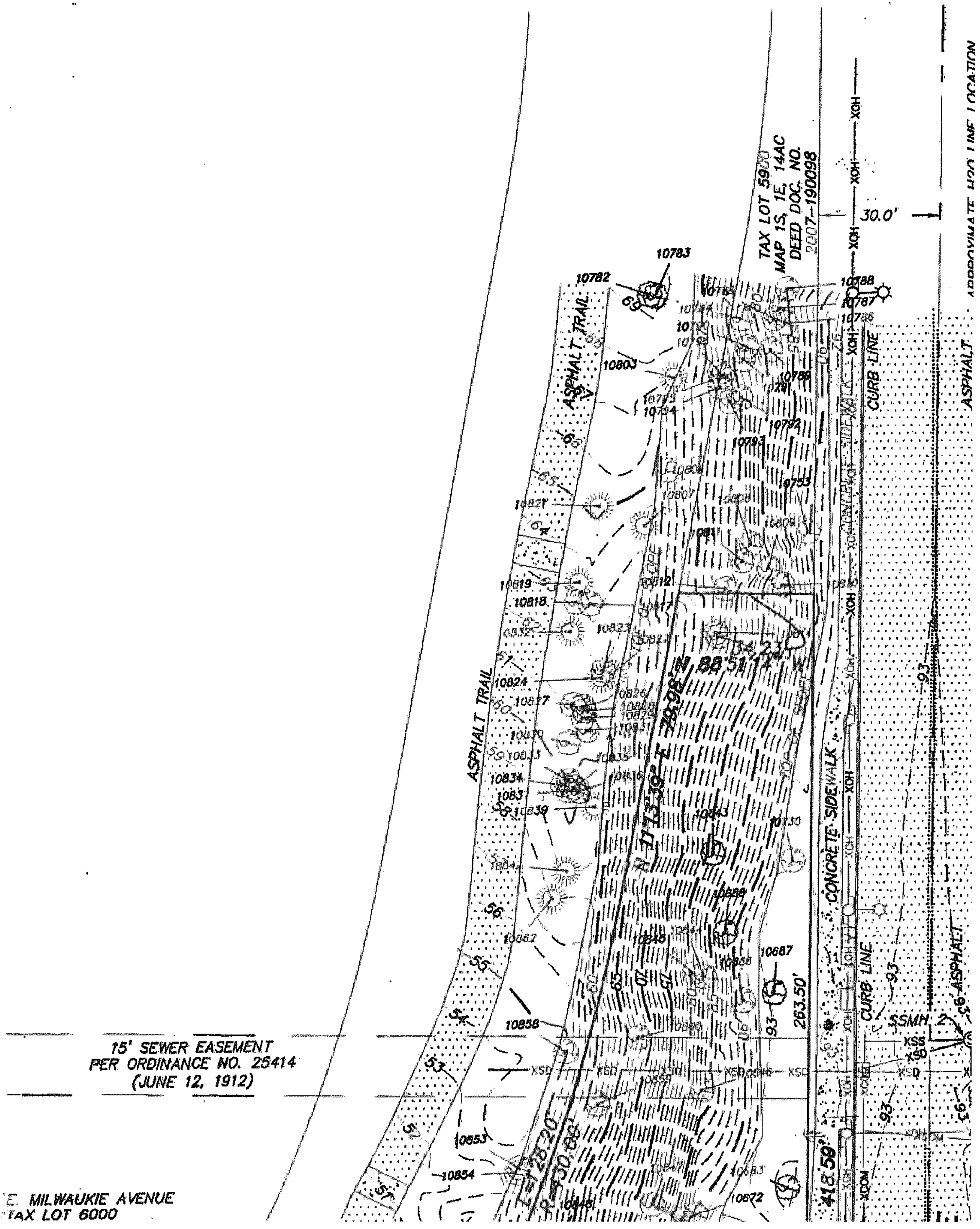

TAX LOT 5900
MAP 1S, 1E, 14AC
DEED DOC. NO.
2007-190098
30.0'
APPROXIMATE H2O LINE LOCATION
ASPHALT
CURB LINE
CONCRETE SIDEWALK
ASPHALT TRAIL
ASPHALT TRAIL
N 88°51'12" W
84.23'
N 11°13'39" E
79.98'
263.50'
418.59'
SSMH 2
XSS
XSD
XOH
XCOM
10782
10783
10788
10787
10803
10821
10819
10818
10824
10827
10834
10858
10853
10854
10887
10872
15' SEWER EASEMENT
PER ORDINANCE NO. 25414
(JUNE 12, 1912)
E. MILWAUKIE AVENUE
TAX LOT 6000

EXHIBIT B

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