COHO POINT

Preliminary Drainage Report

Prepared for:

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March 2021

2322.14464.01
Revision 1

REGISTERED PROFESSIONAL ENGINEER
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OREGON
NOVEMBER 12, [illegible]
MICHAEL S. GILLETTE
EXPIRES: 06/30/2021

TABLE OF CONTENTS

FIGURES

TABLES

EXECUTIVE SUMMARY

The proposed Coho Point development will construct a multi-story mixed use building (33,000 SF ground level) with associated landscape and pedestrian areas, and improvements to Dogwood park and the public ROW. The proposed development will also construct public sidewalks along SE Main St, SE Washington St, and along a portion of SE McLoughlin Blvd. The project is located at 11100 SE McLoughlin road in Milwaukie, Oregon.

The purpose of this report is to describe the stormwater management strategy being proposed for the Coho Point development. The design follows the standards and regulations developed by the City of Portland, which have been adopted by the City of Milwaukie. These regulations are identified in the City of Portland's Stormwater Management Manual, Bureau of Environmental Services, revised August 2016.

Stormwater from the hardscape and plaza areas around the building will be managed through permeable pavers and pervious concrete. Stormwater from the roof area will be treated in a planter facility located on the second-floor terrace. This facility will provide water quality treatment only. Detention is not proposed with this project since the discharge point is a storm-only pipe in Main St that outfalls to the river at Dogwood Park, roughly 500 ft away. The downstream conveyance system was reviewed, and it was confirmed the 25-yr storm event can be conveyed without surcharge.

I hereby certify that this Stormwater Management Report for the Coho Point development 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.

1.0 PROJECT OVERVIEW

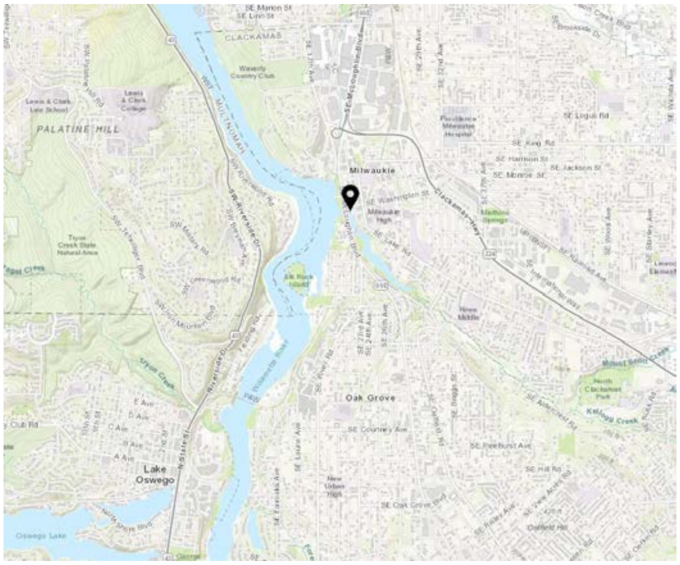
1.1 Project Description

The proposed Coho Point development will construct a multi-story mixed use building (33,000 SF ground level) with associated landscape and pedestrian areas, and improvements to Dogwood park and the public ROW. The proposed development will also construct public sidewalks along SE Main St, SE Washington St, and along a portion of SE McLoughlin Blvd

1.2 Location

The project is located at 11100 SE McLoughlin road in Milwaukie, Oregon.

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 proposed building will be constructed adjacent to the property and ROW lines on the west, north, and east sides. Additionally, the SW side of the building is bordered by Kellogg Creek, and the city is requiring a pedestrian connection along this side of the building to connect SE Main St to SE McLoughlin Blvd. Due to the size of the building and limited space on site, infiltration facilities are infeasible since they would need to be located too close to the building and would potentially undermine the foundation.

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

The project will be designed under Hierarchy Category 3. Since the discharge point is a storm only pipe that flows directly to the river, detention is not proposed. The downstream conveyance system was reviewed, and it was confirmed the 25-yr storm event can be conveyed without surcharge.

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

2.0 SITE CONDITIONS

2.1 Topography

Site slopes range from moderate to steep towards Kellogg Creek to the southwest. The highest elevation of 42 is located in the northeast property corner. The lowest elevation of 32 is located in the southeast property corner.

2.2 Climate

The site is located in Milwaukie, Oregon. There is a gradual change in seasons with defined seasonal characteristics. Average daily temperatures range from 41°F to 69°F. Average annual rainfall recorded in this area is 45 inches.

2.3 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 (See Appendix A: USGS Soils Map - Multnomah County). A hydrologic soil group is not assigned to this soil type.

2.4 Hydrology

Existing

The existing site contains an asphalt parking lot and 3,500 SF building at the northeast corner of the property. Runoff from the existing site generally sheet flows to the southwest to a catch basin which discharges directly to Kellogg Creek. Pollution reduction and flow control are not present on the existing site.

Proposed

Stormwater from the proposed development will be managed using permeable pavers and a stormwater planter on the second-floor terrace. The planter facility will provide water quality treatment only and discharge to the storm pipe in SE Main St. Since the discharge point is a storm only pipe that flows directly to the river, detention is not proposed. The downstream conveyance system was reviewed, and it was confirmed the 25-yr storm event can be conveyed without surcharge.

2.5 Basin Areas

Table 2-1 lists the basin areas under existing and proposed conditions (See Technical Appendix: Figure 1 – Existing Conditions and Figure 2 – Proposed Conditions). Note the proposed conditions site impervious area includes only the building roof. The pedestrian plaza will be constructed with permeable pavers or pervious concrete, and the walkway connecting SE Main St and SE McLoughlin Blvd will be constructed as an elevated steel grated walkway. Both the plaza and walkway are counted as pervious area.

The proposed public improvements along the frontages were not included in the below table since these areas are not routed to on-site storm facilities. The City of Milwaukie Main Street Improvement project recently installed new stormwater planter facilities that manage runoff from the public ROW in this area. The planters are assumed to have been designed to include the future sidewalks along the site frontages.

Table 2-1 Basin Areas

Site Condition	Impervious Area (ac)	Pervious Area (ac)	Total Area (ac)	Percent Impervious (%)
Existing	0.77	0.25	1.02	75.5%
Proposed	0.76	0.26	1.02	74.5%

3.0 WATER QUALITY

3.1 Design Guidelines

The project is designed under Hierarchy Category 3 and requires pollution reduction for stormwater management of the site.

3.2 Stormwater Planters

Stormwater from the building roof area will be treated with a vegetated stormwater planter designed using the Portland Presumptive Approach Calculator (PAC). 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. The planter facility is designed as follows:

- Freeboard = 3”
- Storage Depth = 6”
- Growing Medium Depth = 18”
- Underdrain Rock Depth = 12”

The planter was designed using PAC Facility Configuration D: Lined Facility with Rock Storage and Underdrain. The facility is lined due to its location on the second-floor terrace roof. Table 3-1 below shows a summary of the proposed planter facility. (See Technical Appendix: PAC Report). An overflow standpipe with a dome grate will be included to provide an emergency bypass route (See Technical Appendix: BES Detail SW-141).

Table 3-1 Planter Facility Summary

Basin ID	Impervious Area (sf)	Planter Bottom Area (sf)	Surface Capacity Used (%)	PAC Facility Type
Roof Area	33,052	614	87%	Planter (Flat)

3.3 Permeable Pavers and Concrete

The pedestrian plaza and will be constructed using permeable pavers and pervious concrete. The system is designed under the simplified approach and will include 6” of rock beneath the pavers per City of Portland BES detail SW-110. The pavers will replace the impervious surfaces at a 1:1 ratio; no other areas of the site will be managed by this system.

4.0 WATER QUANTITY

4.1 Design Overview

Detention is not proposed with this project since the discharge point is a storm-only pipe in Main St that outfalls to the river at Dogwood Park, roughly 500 ft away. The downstream conveyance system was reviewed, and it was confirmed the 25-yr storm event can be conveyed without surcharge.

The intent with this design is to get the proposed site runoff to the creek before runoff from the rest of the developed upstream areas makes its way downstream to the System 6 Outfall near Dogwood Park. The existing site currently discharges 0.77 ac of untreated impervious area directly to the creek through its own outfall on-site. The proposed development will remove this outfall, and instead route 0.76 ac of treated impervious area to the 30” city storm line which discharges at the System 6 Outfall roughly 400 LF southeast. Detaining the proposed site runoff would mean it releases to the city storm main at a similar time as the rest of the upstream areas, which would increase the potential for surcharge in this pipe during and after large storm events.

5.0 CONVEYANCE ANALYSIS

5.1 Design Overview

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.

5.2 Hydrologic Method

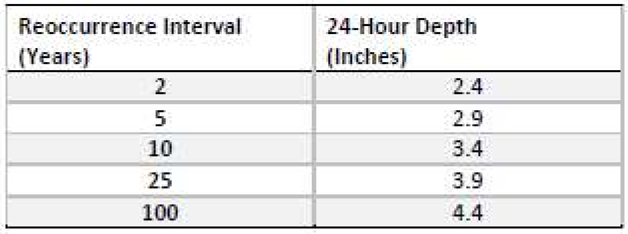
The Santa Barbara Urban Hydrograph (SBUH) method was used for this analysis. The SBUH method is based on the curve number (CN) approach and uses the Natural Resource Conservation Service’s (NRCS) equations for computing soil absorption and precipitation excess. The SBUH method converts the incremental 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.

5.3 Design Storm

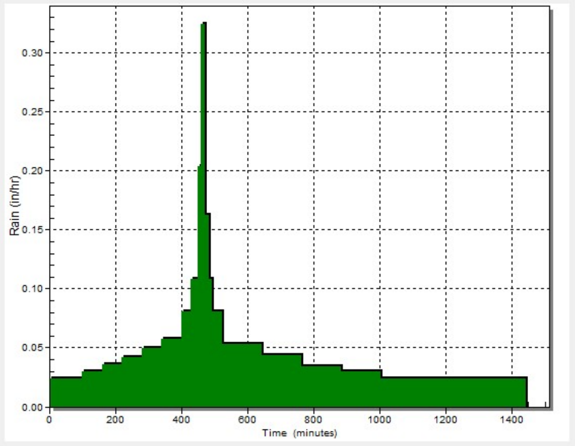
The rainfall distribution used within the City of Portland’s jurisdiction is the design storm of 24-hour duration based on the standard NRCS Type 1A rainfall distribution. Table 5-1 shows total precipitation depths for different storm events which were used for the type 1A 24-hour rainfall distribution in XPSWMM. A typical NRCS Type 1A 24-hour rainfall distribution is shown in Figure 5-1.

Table 5-1 Precipitation Depth



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

Figure 5-1 Type 1A Rainfall Distribution



5.4 System Performance

The City of Milwaukie provided DOWL with an XPSWMM model of the public conveyance system and the City Stormwater Master Plan dated January 2014. The public conveyance system draining to the System 6 Outfall was modeled to determine the system performance before and after the Coho Point development (See Technical Appendix: Pages from City of Milwaukie Stormwater Master Plan).

The city model was updated per the South Downtown Improvements project along SE Main St in 2018. The model was updated to show the new 30" storm line in SE Main St. Other than this update, no other changes to the city model were made. The total areas, percent impervious, curve numbers, and times of concentration remained the same since no other major developments/land use changes occurred in the upstream areas after the model was created in 2013. The composite curve number of the upstream areas is approximately 82, which is representative of a residential/commercial urban development.

Results from the model show the downstream conveyance system can adequately convey the 25-year storm event with no surcharge. A minimum of 5.41 ft of freeboard is maintained within the system through the 25-yr storm (see Technical Appendix: XPSWMM Results – Conveyance Tables).

A comparison of the existing vs proposed conditions shows only minor changes to the downstream conveyance system. The 30" storm line in SE Main St flows at 91% full just downstream of the Coho Point tie in. This is only a slight increase, as the storm line flows at 90% full under existing conditions (see Technical Appendix: XPSWMM Results – Conveyance Tables). The addition of the Coho Point area to the system does not cause surcharge in the SE Main St conveyance line during the 25-yr event.

6.0 SUMMARY

The design follows the standards and regulations developed by the City of Portland, which have been adopted by the City of Milwaukie. These regulations are identified in the City of Portland's Stormwater Management Manual, Bureau of Environmental Services, revised August 2016.

Stormwater from the hardscape and plaza areas around the building will be managed through permeable pavers. Stormwater from the roof area will be treated in a planter facility located on the second-floor terrace. This facility will provide water quality treatment only and discharge to the storm pipe in SE Main St. Since the discharge point is a storm only pipe that flows directly to the river, detention is not proposed. The downstream conveyance system was reviewed, and it was confirmed the 25-yr storm event can be conveyed without surcharge.

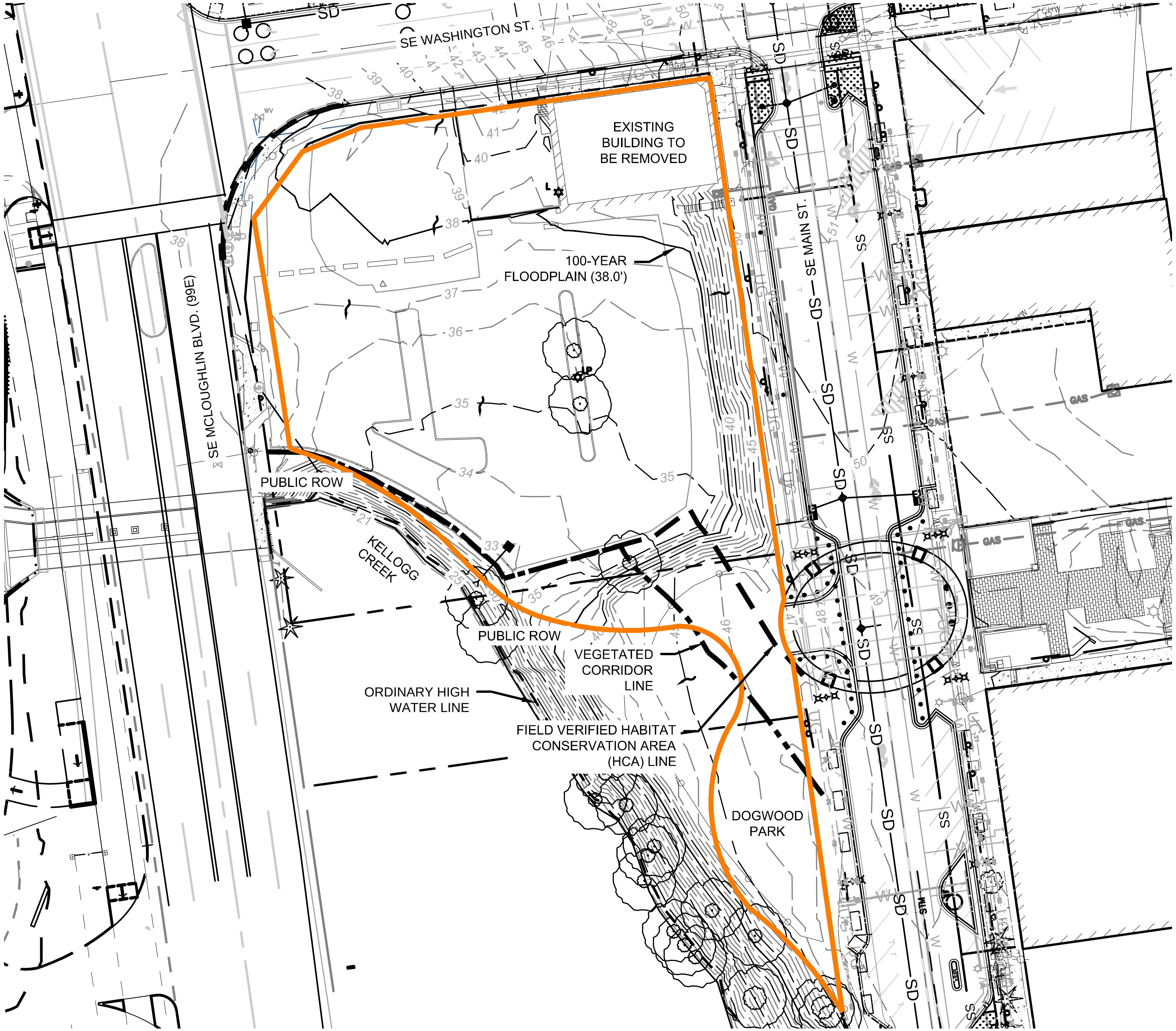
TECHNCIAL APPENDIX - SUPPORTING DATA

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

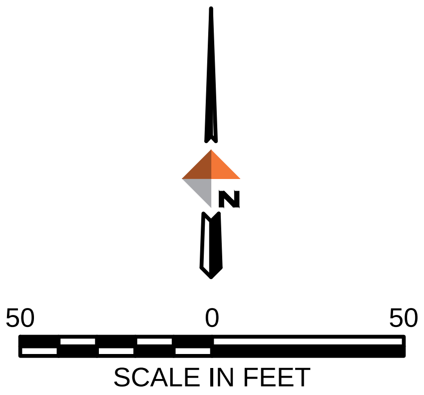
- PAC Report
- XPSWMM Results – Coho Point
    - Schematic
    - Dynamic Long Sections
    - Runoff Data
    - Conveyance Data

- City of Portland BES Standard Detail SW 141 – Lined Planter
- City of Milwaukie Stormwater Master Plan Basin Map
- City of Milwaukie: Pages SD01 – SD03 of the Main Street Reconstruction Plans – August 2018
- Composite Curve Number Calculation for Upstream Areas

- Soil Map - Multnomah County
- Geotechnical Report – GeoDesign – September 2018



Site Condition	Impervious Area (ac)	Pervious Area (ac)	Total Area (ac)	Percent Impervious (%)
Existing	0.77	0.25	1.02	75.5%



SCALE IN FEET

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Portland, Oregon 97205
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COHO POINT
EXISTING CONDITIONS
MILWAUKIE, OREGON

PROJECT	14464
DATE	12/17/2020

FIGURE 1

\\BIL-FS\BIL-projects\22\14464-01\40Study\drain\cad\14464-existing.dwg PLOT DATE 2021-1-5 11:40 SAVED DATE 2021-01-05 11:39 USER: mgillette

Site Condition	Impervious Area (ac)	Pervious Area (ac)	Total Area (ac)	Percent Impervious (%)
Proposed	0.76	0.26	1.02	74.5%

N

50 0 50
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COHO POINT
PROPOSED CONDITIONS
MILWAUKIE, OREGON

PROJECT 14464
DATE 12/17/2020

FIGURE 2

\\BIL-FS\BIL-projects\22\14464-01\40Study\drain\cad\14464-prop.dwg PLOT DATE 2021-1-6 12:58 SAVED DATE 2021-01-06 12:58 USER: mgillette

PAC Report

Project Name Coho Point	Permit No.	Created 12/16/20 12:36 PM
Project Address 11100 SE McLoughlin Rd Milwaukie, OR 97222	Designer Mike Gillette	Last Modified 12/16/20 2:23 PM
	Company DOWL	Report Generated 12/16/20 2:23 PM

Project Summary

Mixed Use Building

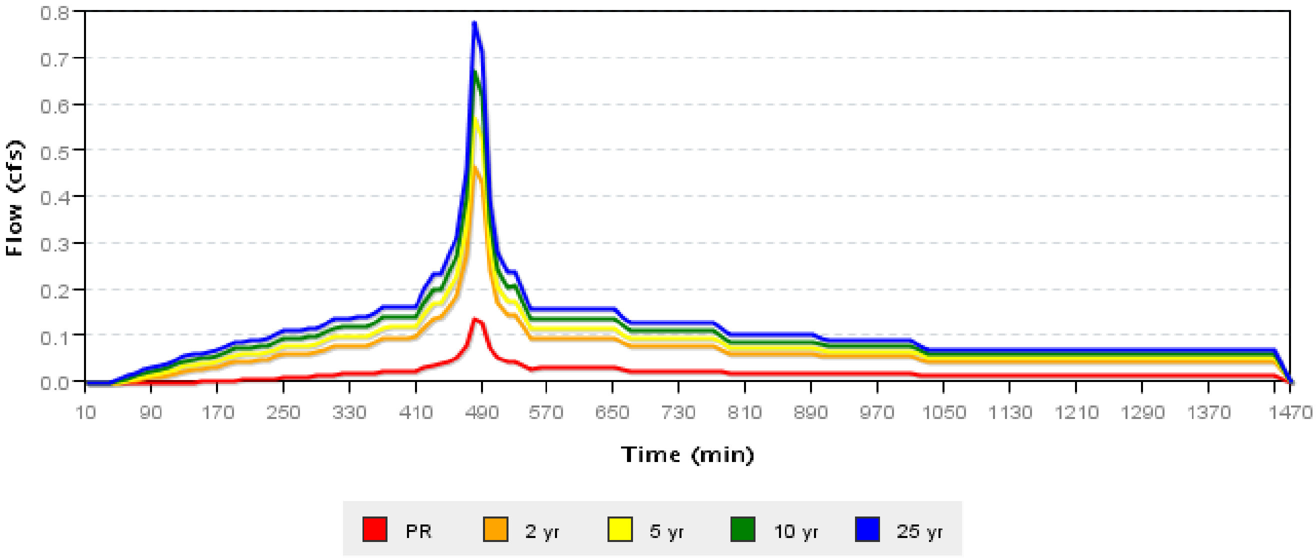
Catchment Name	Impervious Area (sq ft)	Native Soil Design Infiltration Rate	Hierarchy Category	Facility Type	Facility Config	Facility Size (sq ft)	Facility Sizing Ratio	PR Results	Flow Control Results
Roof	33052	0.00	3	Planter (Flat)	D	614	1.9%	Pass	Not Used

Catchment Roof

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	**3**
	Disposal Point	**A**
	Hierarchy Description	**Off-site flow to drainageway, river, or storm-only pipe system**
	Pollution Reduction Requirement	**Pass**
	10-year Storm Requirement	**N/A**
	Flow Control Requirement	**N/A**
	Impervious Area	**33052 sq ft** **0.759 acre**
	Time of Concentration (Tc)	**5**
	Pre-Development Curve Number (CN_{pre})	**72**
	Post-Development Curve Number (CN_{post})	**98**

⚠ Indicates value is outside of recommended range

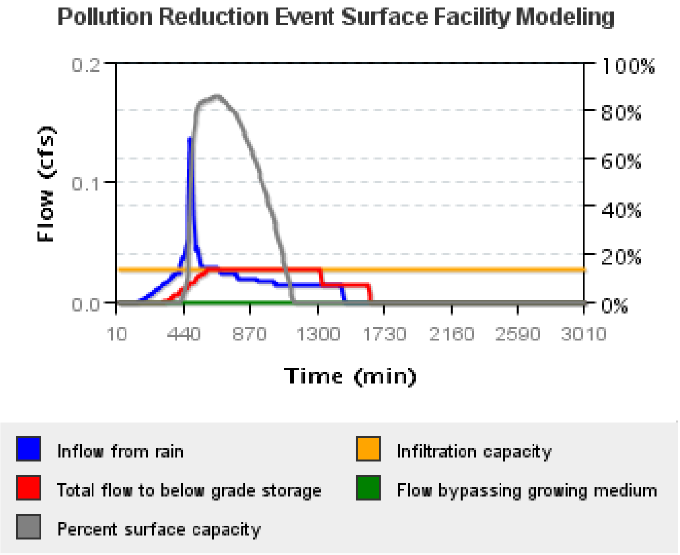
SBUH Results

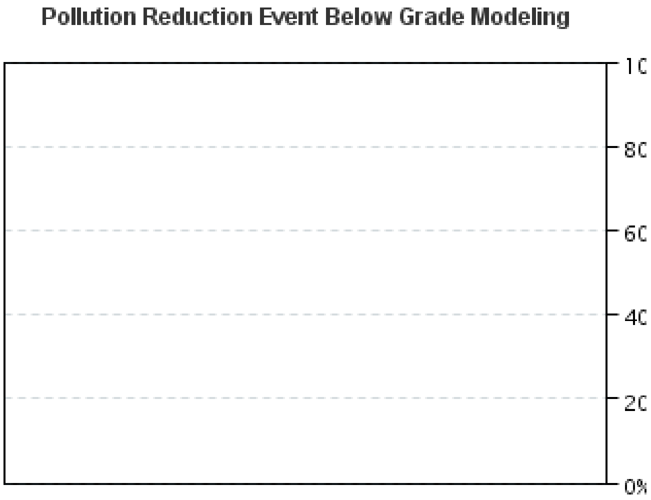


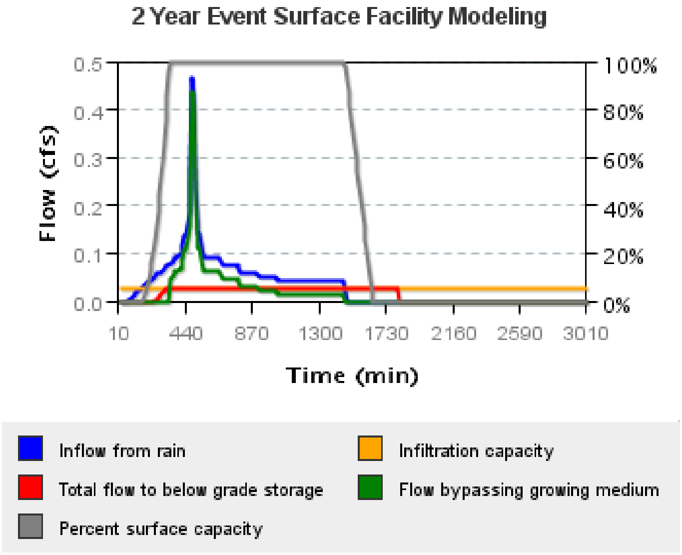
	Pre-Development Rate and Volume		Post-Development Rate and Volume	
	Peak Rate (cfs)	Volume (cf)	Peak Rate (cfs)	Volume (cf)
PR	0	1.906	0.136	1727.06
2 yr	0.041	1315.219	0.467	5980.623
5 yr	0.095	2063.685	0.57	7350.485
10 yr	0.157	2908.71	0.673	8722.397
25 yr	0.226	3829.634	0.775	10095.602

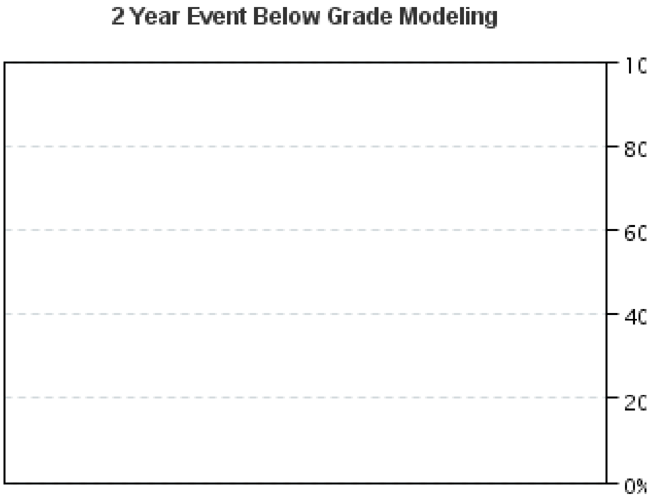
Facility Roof

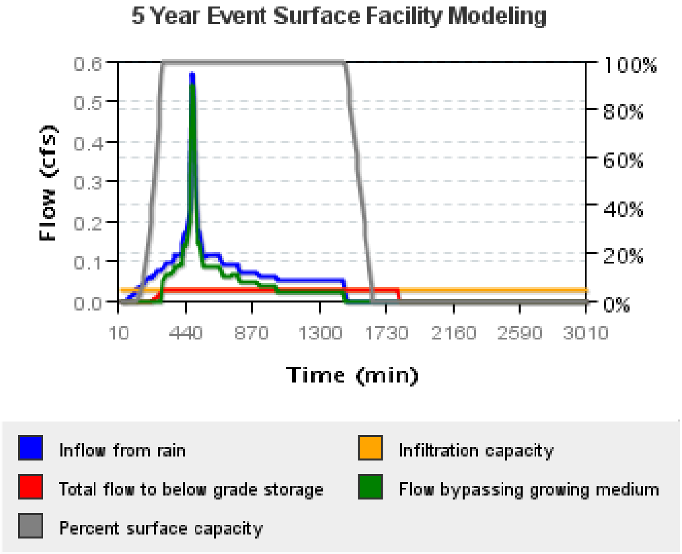
Facility Details	Facility Type	**Planter (Flat)**
	Facility Configuration	**D: Lined Facility with RS and Ud**
	Facility Shape	**Planter**
	Above Grade Storage Data	
	Bottom Area	**614 sq ft**
	Bottom Width	**10.00 ft**
	Storage Depth 1	**6.0 in**
	Growing Medium Depth	**18 in**
	Surface Capacity at Depth 1	**307.0 cu ft**
	Design Infiltration Rate for Native Soil	**0.000 in/hr**
	Infiltration Capacity	**0.028 cfs**
Facility Facts	Total Facility Area Including Freeboard	**614.00 sq ft**
	Sizing Ratio	**1.9%**
Pollution Reduction Results	Pollution Reduction Score	**Pass**
	Overflow Volume	**1725.068 cf**
	Surface Capacity Used	**87%**
Flow Control Results	Flow Control Score	**Not Used**
	Overflow Volume	**8758.447 cf**
	Surface Capacity Used	**100%**

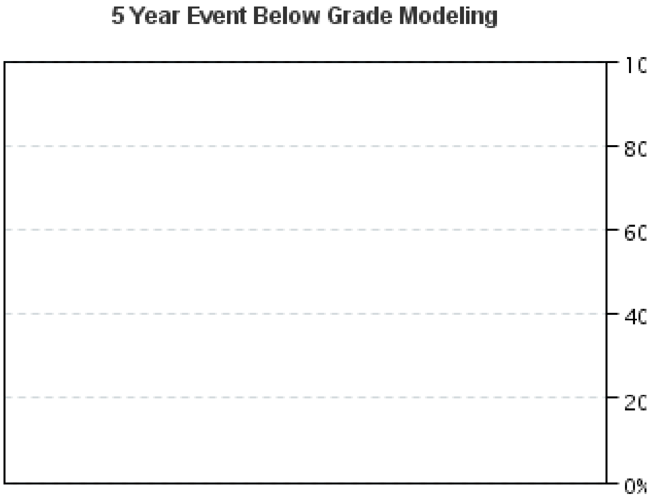

Pollution Reduction Event Surface Facility Modeling

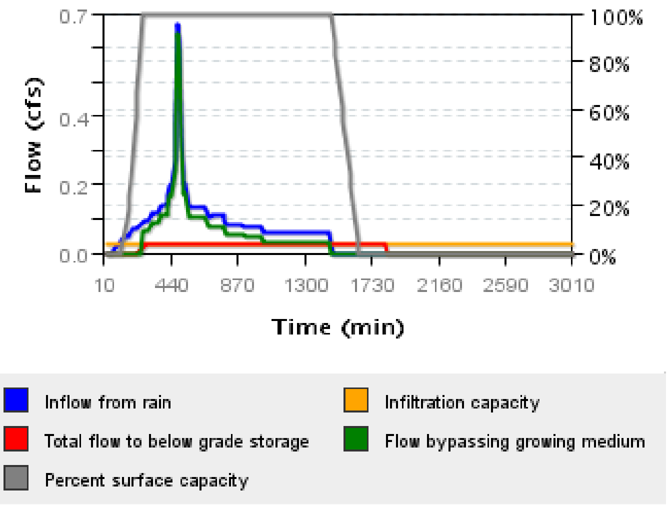

Pollution Reduction Event Below Grade Modeling


2 Year Event Surface Facility Modeling

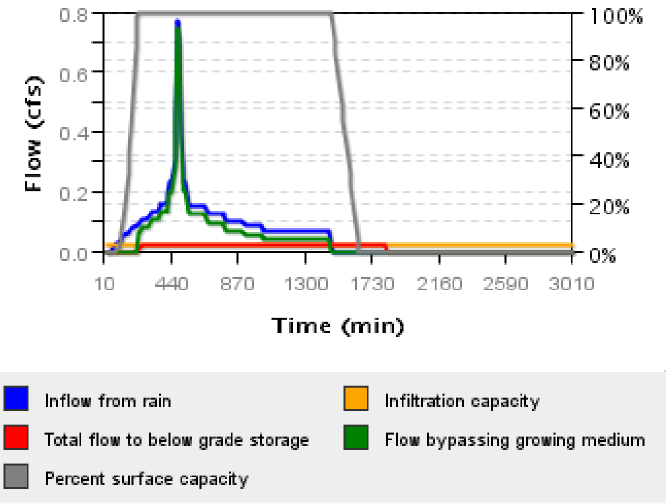

2 Year Event Below Grade Modeling

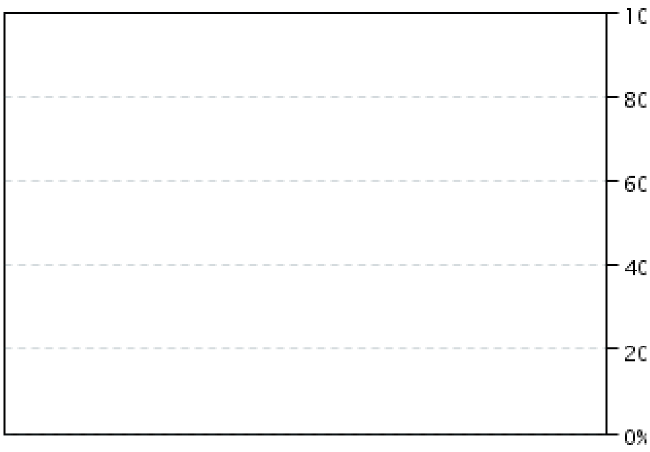

5 Year Event Surface Facility Modeling


5 Year Event Below Grade Modeling


10 Year Event Surface Facility Modeling


10 Year Event Below Grade Modeling


25 Year Event Surface Facility Modeling


25 Year Event Below Grade Modeling

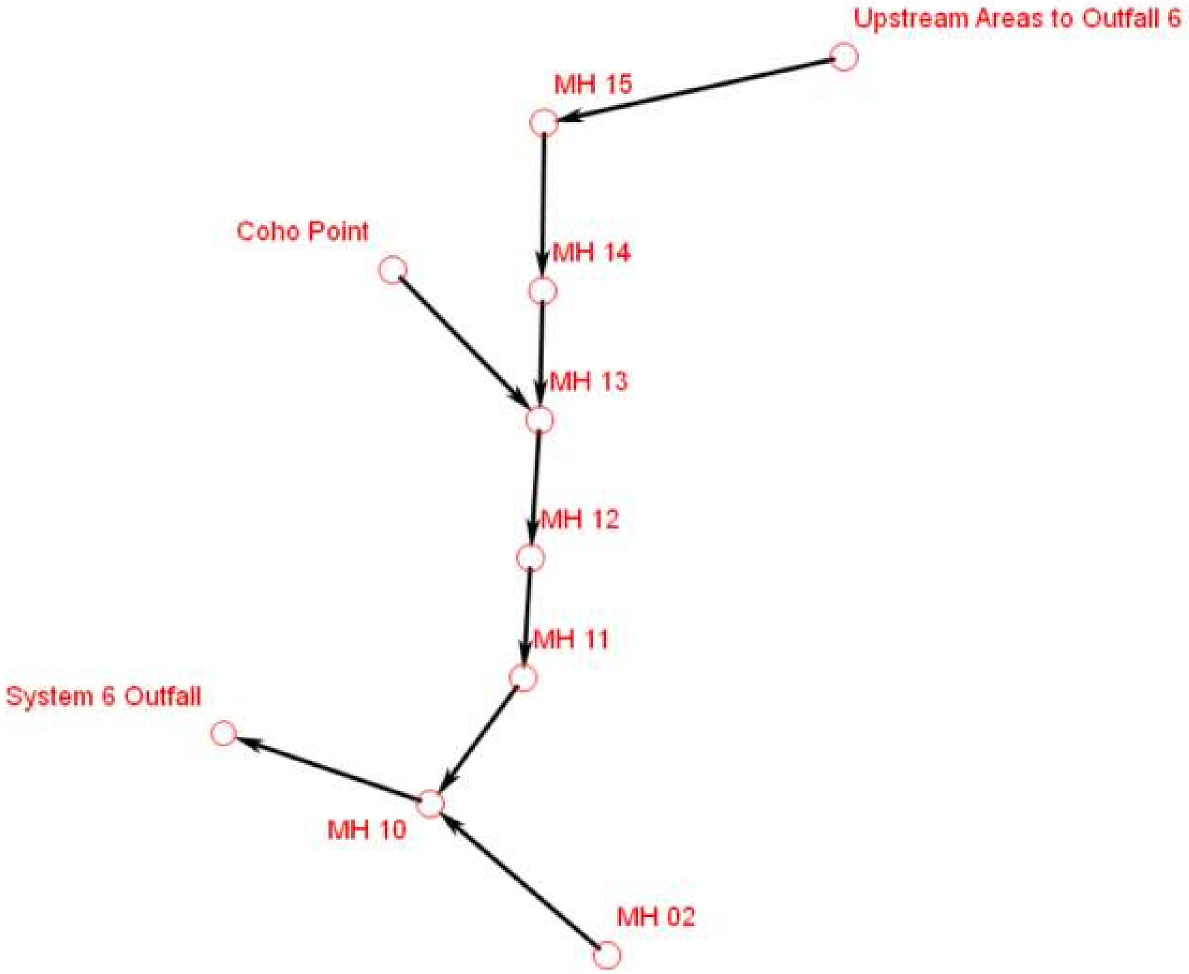
XPSWMM Results – Coho Point

Schematic

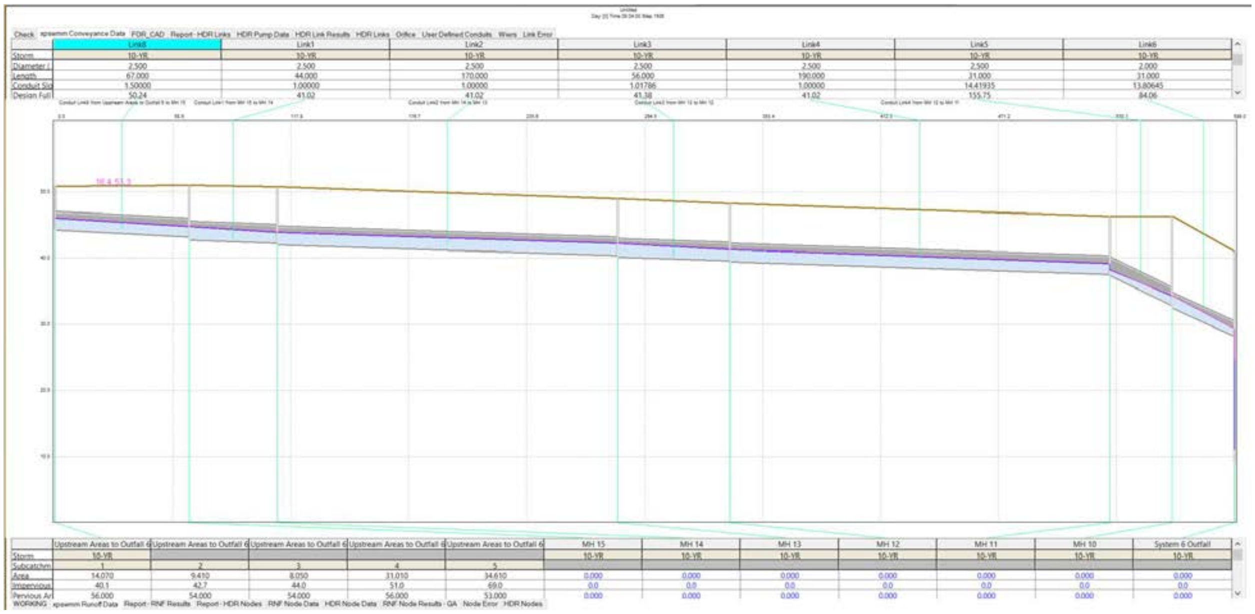
The “Upstream Areas to Outfall 6” node includes data from nodes 41069, 41065, 41032, 4119, and 21101 of the City of Milwaukie model. The “MH 02” node includes data from nodes 41020 and 41011 of the City of Milwaukie model. Basin areas, percent impervious, Tc, and CNs were all input into the below model to accurately represent the System 6 Outfall drainage basin.

The links shown below were modeled per pages SD01 – SD03 of the South Downtown Improvements plans dated August 14, 2018. These sheets are included below.

The proposed conditions model is shown below. The existing conditions model is the same, but with the Coho Point node turned off since the site currently discharges straight to the creek.



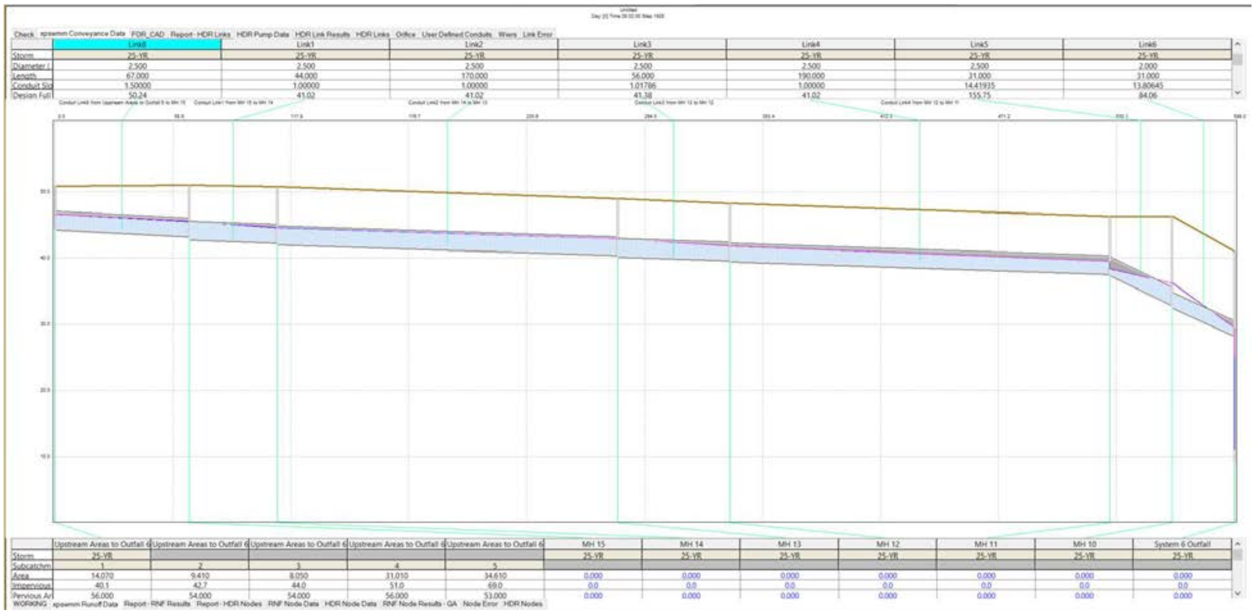
Existing Conditions – 10yr storm event



	Link8	Link1	Link2	Link3	Link4	Link5	Link6
Storm	10-YR	10-YR	10-YR	10-YR	10-YR	10-YR	10-YR
Diameter (	2.500	2.500	2.500	2.500	2.500	2.500	2.000
Length	67.000	44.000	170.000	56.000	190.000	31.000	31.000
Conduit Slo	1.50000	1.00000	1.00000	1.01786	1.00000	14.41935	13.80645
Design Full	50.24	41.02	41.02	41.38	41.02	155.75	84.06

	Upstream Areas to Outfall 6	Upstream Areas to Outfall 6	Upstream Areas to Outfall 6	Upstream Areas to Outfall 6	Upstream Areas to Outfall 6	MH 15	MH 14	MH 13	MH 12	MH 11	MH 10	System 6 Outfall
Storm	10-YR					10-YR	10-YR	10-YR	10-YR	10-YR	10-YR	10-YR
Subcatchm	1	2	3	4	5							
Area	14.070	9.410	8.050	31.010	34.610	0.000	0.000	0.000	0.000	0.000	0.000	0.000
Impervious	40.1	42.7	44.0	51.0	69.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Pervious Ar	56.000	54.000	54.000	56.000	53.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

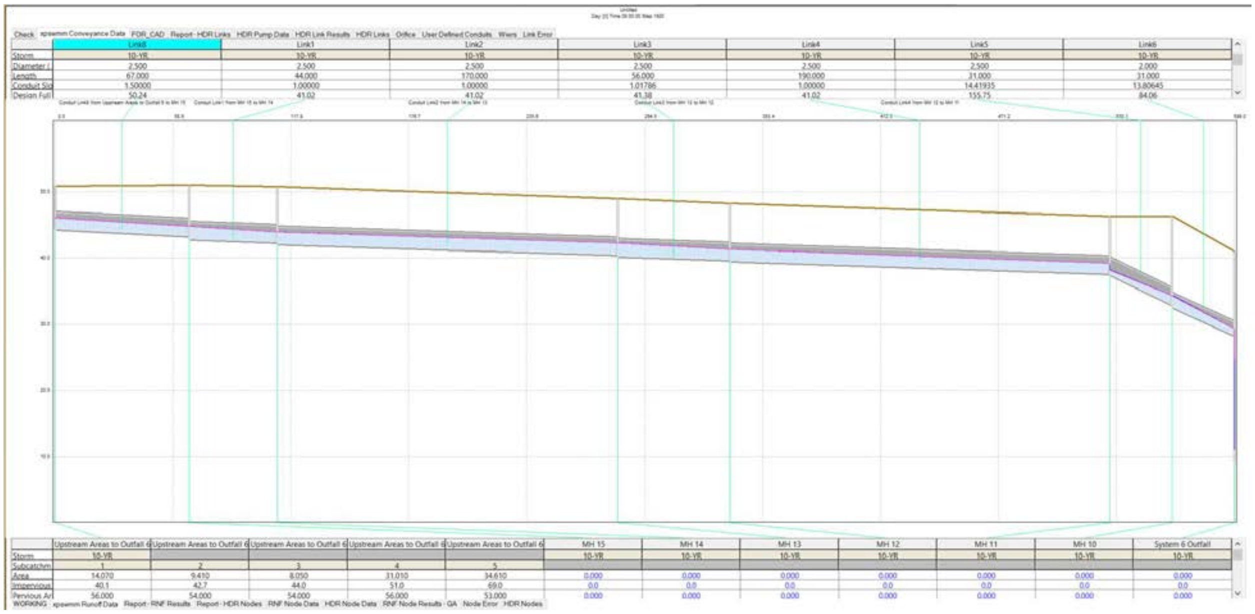
Existing Conditions – 25yr storm event



	Link8	Link1	Link2	Link3	Link4	Link5	Link6
Storm	25-YR	25-YR	25-YR	25-YR	25-YR	25-YR	25-YR
Diameter (	2.500	2.500	2.500	2.500	2.500	2.500	2.000
Length	67.000	44.000	170.000	56.000	190.000	31.000	31.000
Conduit Slo	1.50000	1.00000	1.00000	1.01786	1.00000	14.41935	13.80645
Design Full	50.24	41.02	41.02	41.38	41.02	155.75	84.06

	Upstream Areas to Outfall 6	Upstream Areas to Outfall 6	Upstream Areas to Outfall 6	Upstream Areas to Outfall 6	Upstream Areas to Outfall 6	MH 15	MH 14	MH 13	MH 12	MH 11	MH 10	System 6 Outfall
Storm	25-YR					25-YR	25-YR	25-YR	25-YR	25-YR	25-YR	25-YR
Subcatchm	1	2	3	4	5							
Area	14.070	9.410	8.050	31.010	34.610	0.000	0.000	0.000	0.000	0.000	0.000	0.000
Impervious	40.1	42.7	44.0	51.0	69.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Pervious Ar	56.000	54.000	54.000	56.000	53.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

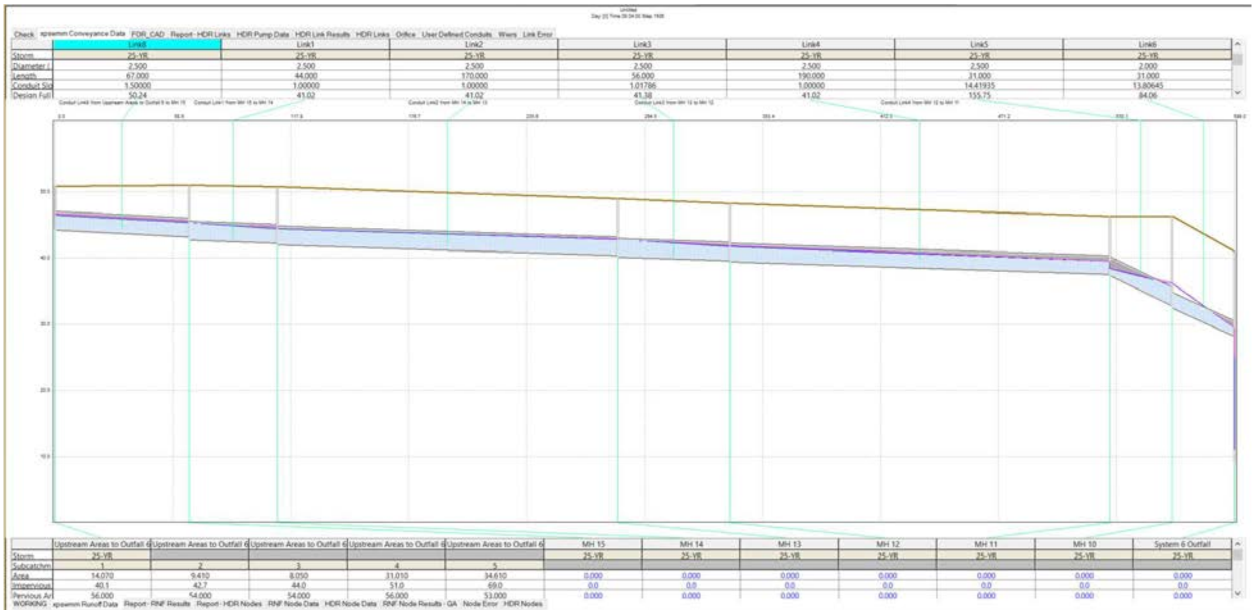
Proposed Conditions – 10yr storm event



	Link8	Link1	Link2	Link3	Link4	Link5	Link6
Storm	10-YR	10-YR	10-YR	10-YR	10-YR	10-YR	10-YR
Diameter (	2.500	2.500	2.500	2.500	2.500	2.500	2.000
Length	67.000	44.000	170.000	56.000	190.000	31.000	31.000
Conduit Slo	1.50000	1.00000	1.00000	1.01786	1.00000	14.41935	13.80645
Design Full	50.24	41.02	41.02	41.38	41.02	155.75	84.06

	Upstream Areas to Outfall 6	Upstream Areas to Outfall 6	Upstream Areas to Outfall 6	Upstream Areas to Outfall 6	Upstream Areas to Outfall 6	MH 15	MH 14	MH 13	MH 12	MH 11	MH 10	System 6 Outfall
Storm	10-YR					10-YR	10-YR	10-YR	10-YR	10-YR	10-YR	10-YR
Subcatchm	1	2	3	4	5							
Area	14.070	9.410	8.050	31.010	34.610	0.000	0.000	0.000	0.000	0.000	0.000	0.000
Impervious	40.1	42.7	44.0	51.0	69.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Pervious Ar	56.000	54.000	54.000	56.000	53.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

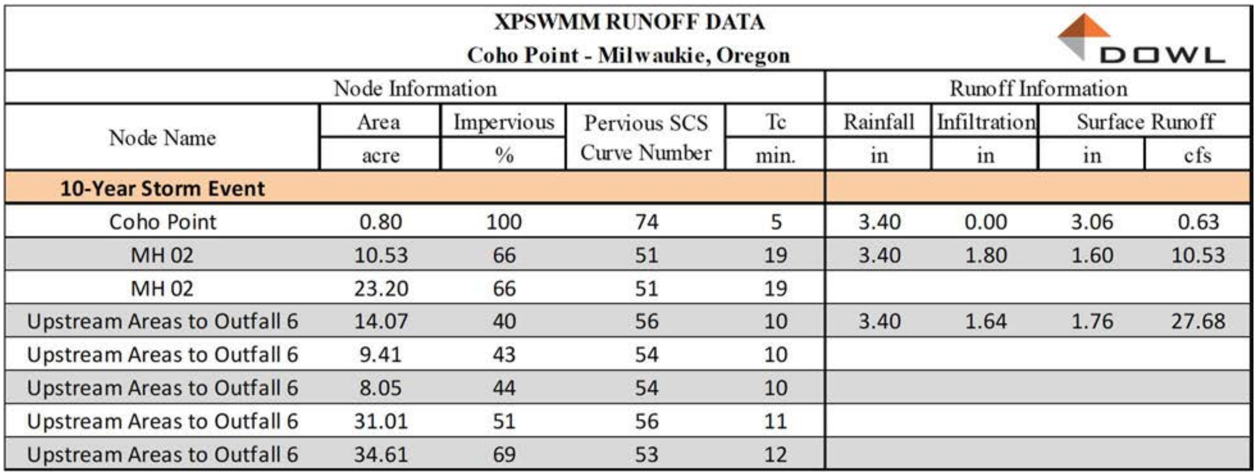
Proposed Conditions – 25yr storm event



	Link8	Link1	Link2	Link3	Link4	Link5	Link6
Storm	25-YR	25-YR	25-YR	25-YR	25-YR	25-YR	25-YR
Diameter (	2.500	2.500	2.500	2.500	2.500	2.500	2.000
Length	67.000	44.000	170.000	56.000	190.000	31.000	31.000
Conduit Slo	1.50000	1.00000	1.00000	1.01786	1.00000	14.41935	13.80645
Design Full	50.24	41.02	41.02	41.38	41.02	155.75	84.06

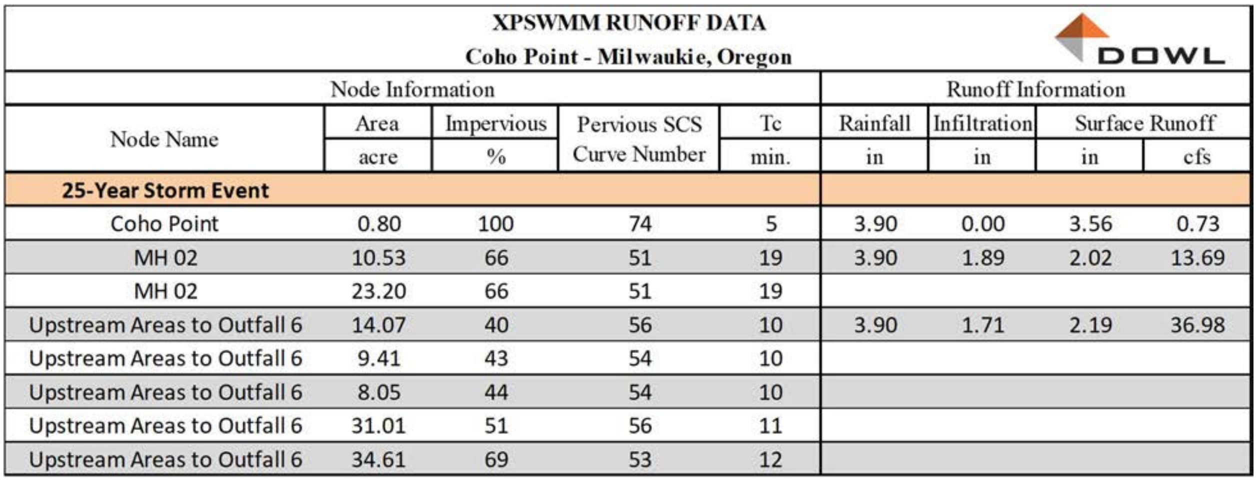
	Upstream Areas to Outfall 6	Upstream Areas to Outfall 6	Upstream Areas to Outfall 6	Upstream Areas to Outfall 6	Upstream Areas to Outfall 6	MH 15	MH 14	MH 13	MH 12	MH 11	MH 10	System 6 Outfall
Storm	25-YR					25-YR	25-YR	25-YR	25-YR	25-YR	25-YR	25-YR
Subcatchm	1	2	3	4	5							
Area	14.070	9.410	8.050	31.010	34.610	0.000	0.000	0.000	0.000	0.000	0.000	0.000
Impervious	40.1	42.7	44.0	51.0	69.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Pervious Ar	56.000	54.000	54.000	56.000	53.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

Runoff Tables



XPSWMM RUNOFF DATA
Coho Point - Milwaukie, Oregon

Node Information					Runoff Information			
Node Name	Area	Impervious	Pervious SCS Curve Number	Tc	Rainfall	Infiltration	Surface Runoff	
	acre	%		min.	in	in	in	cfs
10-Year Storm Event								
Coho Point	0.80	100	74	5	3.40	0.00	3.06	0.63
MH 02	10.53	66	51	19	3.40	1.80	1.60	10.53
MH 02	23.20	66	51	19				
Upstream Areas to Outfall 6	14.07	40	56	10	3.40	1.64	1.76	27.68
Upstream Areas to Outfall 6	9.41	43	54	10				
Upstream Areas to Outfall 6	8.05	44	54	10				
Upstream Areas to Outfall 6	31.01	51	56	11				
Upstream Areas to Outfall 6	34.61	69	53	12				



XPSWMM RUNOFF DATA
Coho Point - Milwaukie, Oregon

Node Information					Runoff Information			
Node Name	Area	Impervious	Pervious SCS Curve Number	Tc	Rainfall	Infiltration	Surface Runoff	
	acre	%		min.	in	in	in	cfs
25-Year Storm Event								
Coho Point	0.80	100	74	5	3.90	0.00	3.56	0.73
MH 02	10.53	66	51	19	3.90	1.89	2.02	13.69
MH 02	23.20	66	51	19				
Upstream Areas to Outfall 6	14.07	40	56	10	3.90	1.71	2.19	36.98
Upstream Areas to Outfall 6	9.41	43	54	10				
Upstream Areas to Outfall 6	8.05	44	54	10				
Upstream Areas to Outfall 6	31.01	51	56	11				
Upstream Areas to Outfall 6	34.61	69	53	12				

Coho Point Conveyance Data

XPSWMM CONVEYANCE DATA - 10 YEAR STORM EVENT
Coho Point - Milwaukie, Oregon

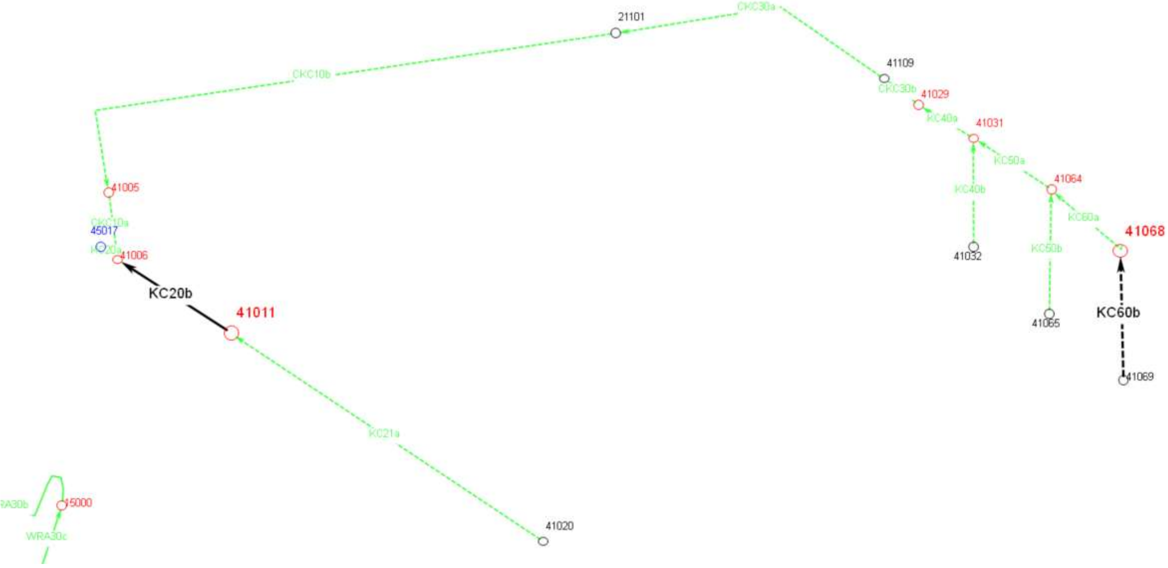
Location		Conduit Properties			Conduit Results						Node Information							
Station		Diameter	Length	Slope	Design Capacity	Qmax / Qdesign	Max Flow	Max Velocity	Max Flow Depth	y/d0	US Ground Elev.	DS Ground Elev.	US IE	DS IE	US Freeboard	DS Freeboard	US HGL	DS HGL
From	To	ft	ft	%	cfs		cfs	ft/s	ft		ft	ft	ft	ft	ft	ft	ft	ft
EXISTING CONDITIONS																		
Upstream Areas to Outfall 6	MH 15	2.50	67.00	1.50	50.24	0.55	27.67	8.39	1.71	0.69	50.90	51.05	44.40	43.40	4.79	6.20	46.11	44.85
MH 15	MH 14	2.50	44.00	1.00	41.02	0.68	27.67	7.55	1.95	0.78	51.05	50.79	42.90	42.46	6.20	6.81	44.85	43.99
MH 14	MH 13	2.50	170.00	1.00	41.02	0.68	27.67	7.68	1.73	0.69	50.79	48.98	42.26	40.56	6.81	6.70	43.99	42.28
MH 13	MH 12	2.50	56.00	1.02	41.38	0.67	27.66	7.56	1.92	0.77	48.98	48.30	40.36	39.79	6.70	6.96	42.28	41.34
MH 12	MH 11	2.50	190.00	1.00	41.02	0.67	27.65	8.13	1.75	0.70	48.30	46.35	39.59	37.69	6.96	8.15	41.34	38.20
MH 11	MH 10	2.50	31.00	14.42	155.75	0.18	27.65	16.18	1.31	0.52	46.35	46.30	37.49	33.02	8.15	11.97	38.20	34.33
MH 10	System 6 Outfall	2.00	31.00	13.81	84.06	0.45	38.16	17.14	1.71	0.85	46.30	41.00	32.62	28.34	11.97	30.01	34.33	10.99
MH 02	MH 10	2.00	53.00	1.53	27.97	0.38	10.52	5.80	1.51	0.75	43.63	46.30	33.63	32.82	8.96	11.97	34.67	34.33
PROPOSED CONDITIONS																		
Upstream Areas to Outfall 6	MH 15	2.50	67.00	1.50	50.24	0.55	27.67	8.39	1.71	0.69	50.90	51.05	44.40	43.40	4.79	6.20	46.11	44.85
MH 15	MH 14	2.50	44.00	1.00	41.02	0.68	27.67	7.55	1.95	0.78	51.05	50.79	42.90	42.46	6.20	6.81	44.85	43.98
MH 14	MH 13	2.50	170.00	1.00	41.02	0.68	27.67	7.62	1.75	0.70	50.79	48.98	42.26	40.56	6.81	6.67	43.98	42.31
MH 13	MH 12	2.50	56.00	1.02	41.38	0.68	28.26	7.58	1.95	0.78	48.98	48.30	40.36	39.79	6.67	6.93	42.31	41.37
MH 12	MH 11	2.50	190.00	1.00	41.02	0.69	28.25	8.16	1.78	0.71	48.30	46.35	39.59	37.69	6.93	8.14	41.37	38.21
MH 11	MH 10	2.50	31.00	14.42	155.75	0.18	28.25	16.21	1.34	0.53	46.35	46.30	37.49	33.02	8.14	11.94	38.21	34.36
MH 10	System 6 Outfall	2.00	31.00	13.81	84.06	0.46	38.76	17.17	1.74	0.87	46.30	41.00	32.62	28.34	11.94	30.01	34.36	10.99
MH 02	MH 10	2.00	53.00	1.53	27.97	0.38	10.52	5.77	1.54	0.77	43.63	46.30	33.63	32.82	8.95	11.94	34.68	34.36
Coho Point	MH 13	1.00	100.00	1.00	3.56	0.17	0.62	2.11	1.75	1.75	47.00	48.98	41.56	40.56	4.66	6.67	42.34	42.31

XPSWMM CONVEYANCE DATA - 25 YEAR STORM EVENT
Coho Point - Milwaukie, Oregon

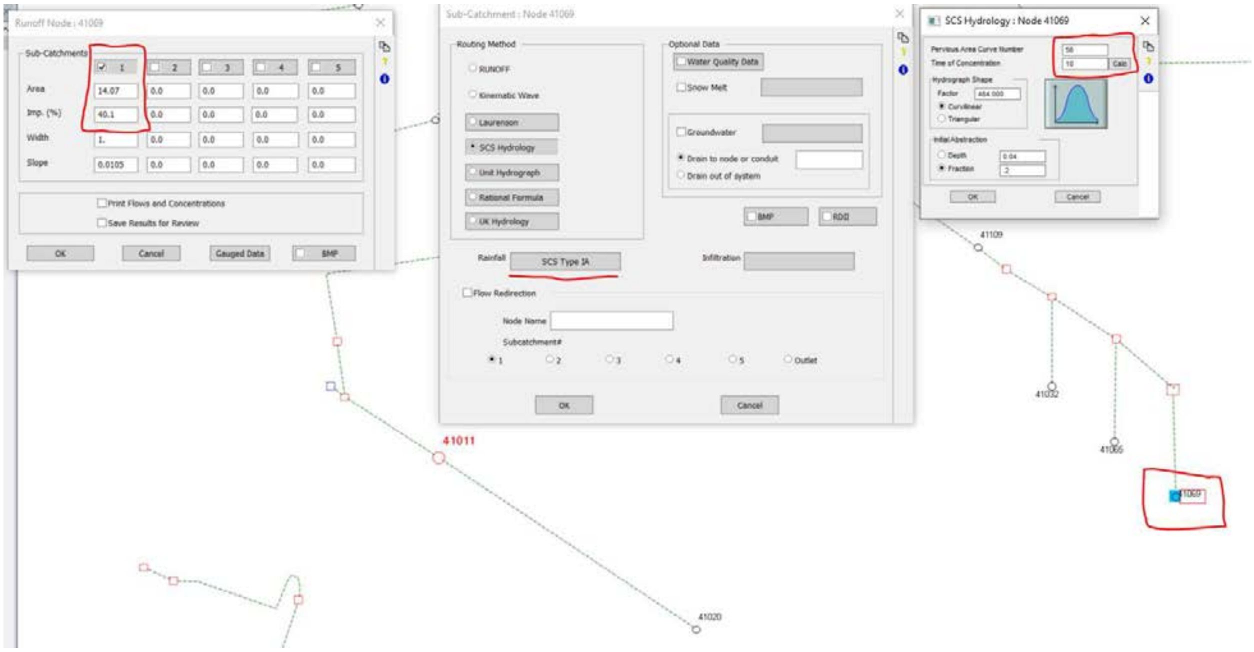
Location		Conduit Properties			Conduit Results						Node Information							
Station		Diameter	Length	Slope	Design Capacity	Qmax / Qdesign	Max Flow	Max Velocity	Max Flow Depth	y/d0	US Ground Elev.	DS Ground Elev.	US IE	DS IE	US Freeboard	DS Freeboard	US HGL	DS HGL
From	To	ft	ft	%	cfs		cfs	ft/s	ft		ft	ft	ft	ft	ft	ft	ft	ft
EXISTING CONDITIONS																		
Upstream Areas to Outfall 6	MH 15	2.50	67.00	1.50	50.24	0.74	36.93	8.41	2.27	0.91	50.90	51.05	44.40	43.40	4.23	5.47	46.67	45.58
MH 15	MH 14	2.50	44.00	1.00	41.02	0.90	36.90	7.73	2.68	1.07	51.05	50.79	42.90	42.46	5.47	6.13	45.58	44.66
MH 14	MH 13	2.50	170.00	1.00	41.02	0.90	36.83	7.79	2.40	0.96	50.79	48.98	42.26	40.56	6.13	6.07	44.66	42.91
MH 13	MH 12	2.50	56.00	1.02	41.38	0.89	36.81	7.91	2.55	1.02	48.98	48.30	40.36	39.79	6.07	6.47	42.91	41.83
MH 12	MH 11	2.50	190.00	1.00	41.02	0.90	36.80	8.56	2.24	0.90	48.30	46.35	39.59	37.69	6.47	7.97	41.83	38.38
MH 11	MH 10	2.50	31.00	14.42	155.75	0.24	36.81	16.20	3.19	1.27	46.35	46.30	37.49	33.02	7.97	10.09	38.38	36.21
MH 10	System 6 Outfall	2.00	31.00	13.81	84.06	0.60	50.45	19.79	3.59	1.79	46.30	41.00	32.62	28.34	10.09	30.01	36.21	10.99
MH 02	MH 10	2.00	53.00	1.53	27.97	0.49	13.68	5.69	3.39	1.69	43.63	46.30	33.63	32.82	7.06	10.09	36.57	36.21
PROPOSED CONDITIONS																		
Upstream Areas to Outfall 6	MH 15	2.50	67.00	1.50	50.24	0.74	36.92	8.41	2.29	0.92	50.90	51.05	44.40	43.40	4.21	5.41	46.69	45.64
MH 15	MH 14	2.50	44.00	1.00	41.02	0.90	36.84	7.76	2.74	1.10	51.05	50.79	42.90	42.46	5.41	6.04	45.64	44.75
MH 14	MH 13	2.50	170.00	1.00	41.02	0.89	36.63	7.72	2.49	1.00	50.79	48.98	42.26	40.56	6.04	6.03	44.75	42.95
MH 13	MH 12	2.50	56.00	1.02	41.38	0.90	37.31	7.93	2.59	1.04	48.98	48.30	40.36	39.79	6.03	6.43	42.95	41.87
MH 12	MH 11	2.50	190.00	1.00	41.02	0.91	37.32	8.58	2.28	0.91	48.30	46.35	39.59	37.69	6.43	7.93	41.87	38.42
MH 11	MH 10	2.50	31.00	14.42	155.75	0.24	37.32	16.23	3.28	1.31	46.35	46.30	37.49	33.02	7.93	10.00	38.42	36.30
MH 10	System 6 Outfall	2.00	31.00	13.81	84.06	0.61	50.99	19.95	3.68	1.84	46.30	41.00	32.62	28.34	10.00	30.01	36.30	10.99
MH 02	MH 10	2.00	53.00	1.53	27.97	0.49	13.68	5.66	3.48	1.74	43.63	46.30	33.63	32.82	6.96	10.00	36.67	36.30
Coho Point	MH 13	1.00	100.00	1.00	3.56	0.21	0.73	2.14	2.39	2.39	47.00	48.98	41.56	40.56	4.00	6.03	43.00	42.95

City of Milwaukie XPSWMM model:

The XPSWMM model provided by the City of Milwaukie was created in 2013. The below schematic shows the pipe network draining to the System 6 outfall, which is where the Coho Point Development will discharge to. The relevant portions of this model (upstream node areas) were copied to the DOWL / Coho Point xpswmm model to accurately represent the existing upstream areas.



The below screenshot shows the information within one of the City of Milwaukie xpswmm model nodes. The rest of the upstream area node information can be found in the runoff data table above. Please note the curve number shown is for the pervious areas only. The composite curve numbers for the upstream areas are in the 75-80 range depending on site use.



Runoff Node : 41069

Sub-Catchments

	☑ 1	☐ 2	☐ 3	☐ 4	☐ 5
Area	14.07	0.0	0.0	0.0	0.0
Imp. (%)	40.1	0.0	0.0	0.0	0.0
Width	1.	0.0	0.0	0.0	0.0
Slope	0.0105	0.0	0.0	0.0	0.0

☐ Print Flows and Concentrations
☐ Save Results for Review

Sub-Catchment : Node 41069

Routing Method: ● SCS Hydrology

Optional Data: ☐ Water Quality Data; ☐ Snow Melt; ☐ Groundwater; ● Drain to node or conduit

Rainfall: SCS Type IA

☐ Flow Redirection

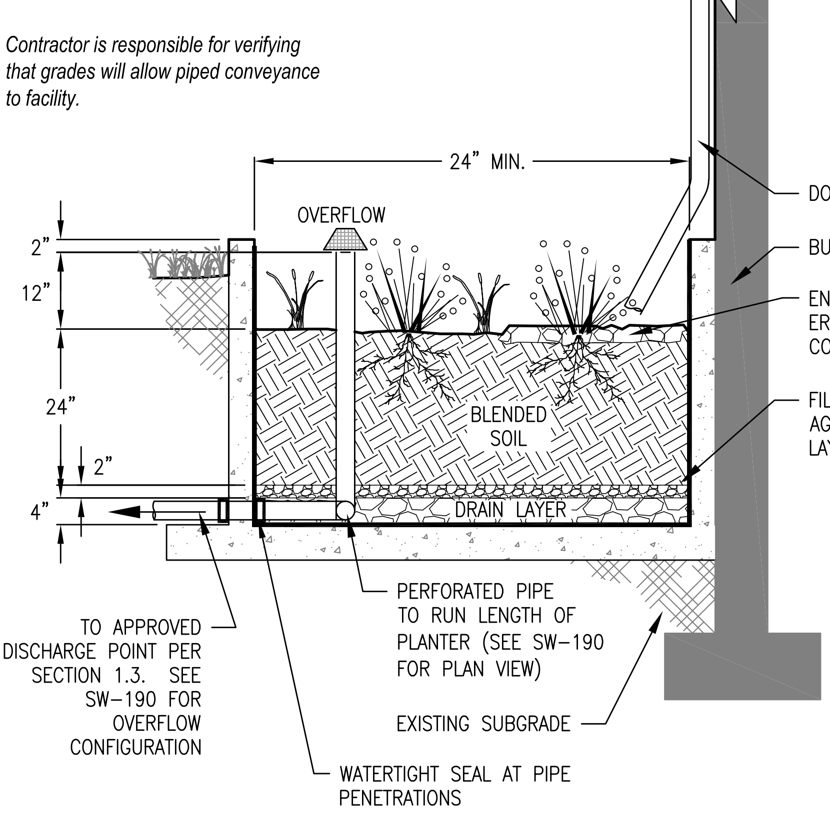
Subcatchment#: ● 1

SCS Hydrology : Node 41069

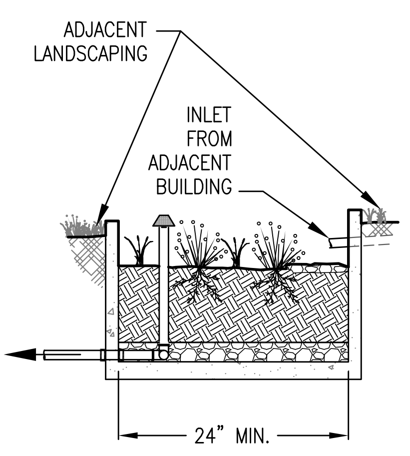
Pervious Area Curve Number: 58
Time of Concentration: 10
Hydrograph Shape
Factor: 484.000
● Curvilinear
○ Triangular
Initial Abstraction
○ Depth: 0.04
● Fraction: .2

ADJACENT TO BUILDING

Contractor is responsible for verifying that grades will allow piped conveyance to facility.



FREESTANDING PLANTER



1. Setbacks: No setback is required for lined planters. Walls can't exceed 30" height above grade if within 5' of property line including right-of-way.

2. Facility Slope (planted floor): Maximum of 0.5% in all directions.

3. Planter Structure: A single-pour monolithic concrete shell, without cold joints, is required to avoid the requirement for liner. Include walls on foundation plans. Check state structural standards for foundations.

4. Waterproofing: No additional waterproofing is needed if structure is monolithically poured.

5. Piping: Conform with Oregon Plumbing Specialty Code (OPSC) requirements.

6. Drain Layer: 4" of $\frac{3}{4}$"–1 $\frac{1}{2}$" washed drain rock. Filter aggregate layer: 2–3" of $\frac{1}{4}$"–No.10 washed angular aggregate.

7. Overflow: Overflow elevation must allow for 2" of freeboard, minimum. Protect from debris and sediment with strainer or grate.

8. Blended Soil: Use BES' standard soil blend for stormwater facilities (SWMM Section 6.3) unless otherwise approved. Install minimum of 24" of blended soil.

9. Vegetation: Refer to plant list in SWMM Section 3.5. Minimum container size is 1 gal. Number of plantings per 100sf of facility area: 80 herbaceous plants OR 72 herbaceous plants and 4 small shrubs.

10. Entrance Erosion Control: Install river rock, flagstone, or similar to dissipate the energy of incoming water at entrances and ends of downspout extensions.

11. Inspections: Call BDS IVR Inspection Line, (503) 823-7000, request 487. 3 inspections required.

CONSTRUCTION REQUIREMENTS

Do not allow temporary storage of construction waste or materials in the facilities. Do not allow entry of runoff or sediment during construction.

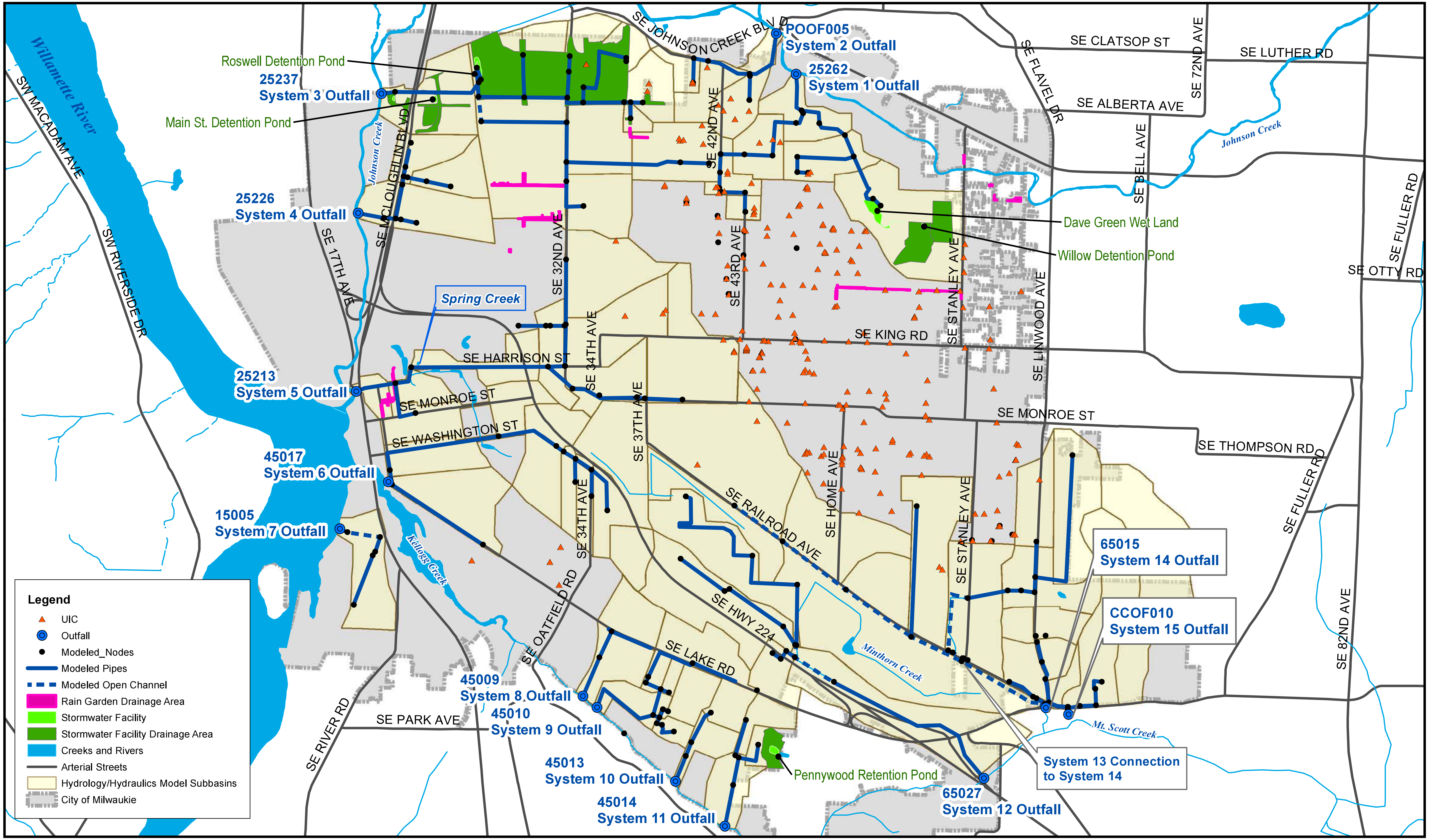
– DRAWINGS NOT TO SCALE –

Bureau of Environmental Services

STORMWATER MANAGEMENT
TYPICAL DETAILS FOR
PRIVATE PROPERTY

LINED PLANTER SW-141

9-2-20



Brown AND Caldwell

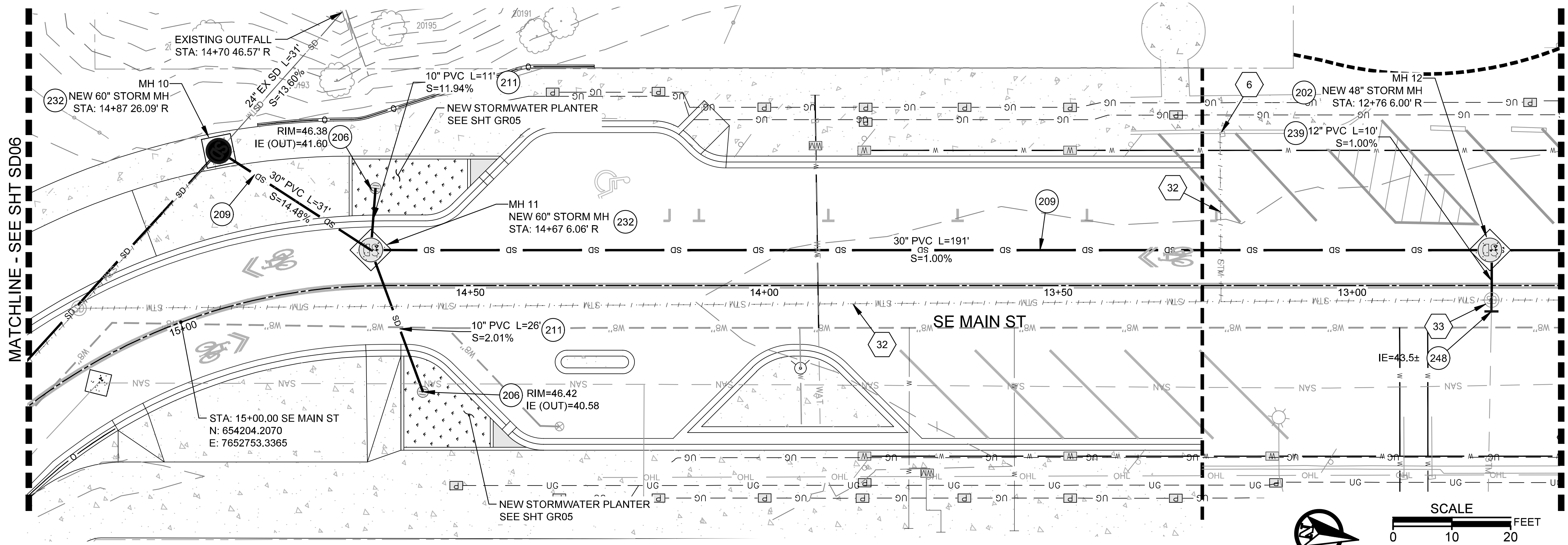
March 19th, 2013

0 1,500 3,000 Feet

N

CITY OF MILWAUKIE
STORMWATER MASTER PLAN UPDATE

BMP COVERAGE
FIGURE 2-5



1 PLAN - MAIN ST - STA 12+70 TO 15+00

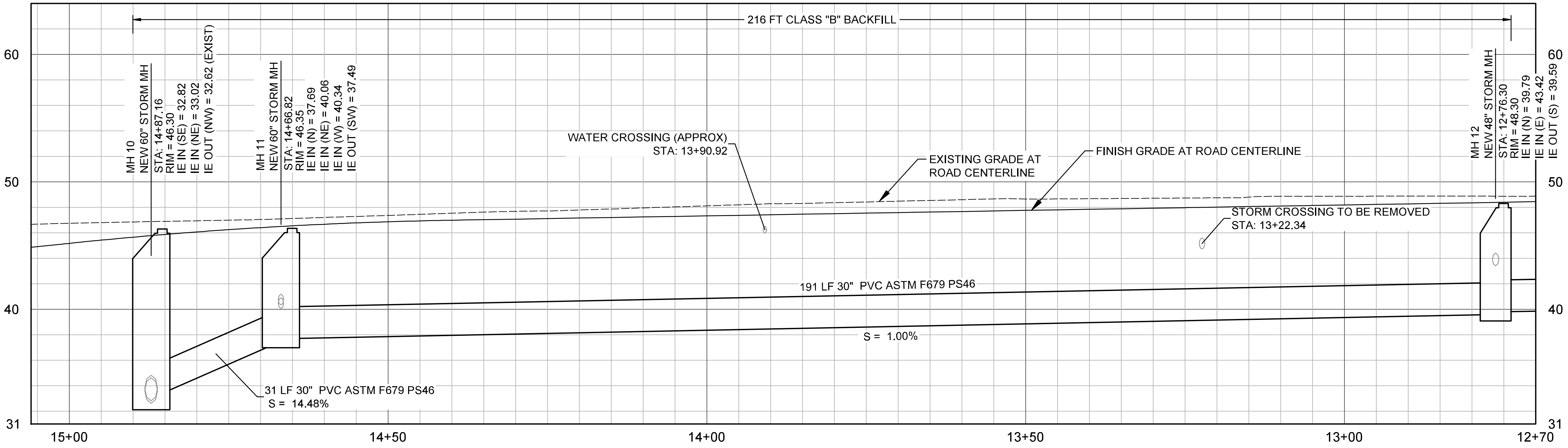
PLAN

UTILITY CONSTRUCTION NOTES:

202	INSTALL 48" MANHOLE. SEE ODOT STANDARD DRAWING RD335.
206	CONSTRUCT BEEHIVE INLET. SEE DTL 5 ON SHT DT11.
209	INSTALL 30" STORM PIPE
210	INSTALL 24" STORM PIPE.
211	INSTALL 10" STORM PIPE.
231	CONNECT EXISTING PIPE TO NEW MANHOLE.
232	INSTALL 60" MANHOLE.
239	INSTALL 12" STORM PIPE
248	CONNECT TO EXISTING LATERAL WITH WATERTIGHT COUPLING.

DEMOLITION NOTES:

6	REMOVE EXISTING CATCH BASIN.
9	REMOVE STORM PIPE.
32	FILL AND ABANDON EXISTING STORM PIPE IN PLACE. SEE SHEET SD01-SD07 FOR MORE DETAILS.
33	ABANDON EXISTING MANHOLE. REMOVE STRUCTURE TO DEPTH OF EXISTING LATERAL. SEE SHEET SD01-SD07 FOR MORE DETAILS.



2 PROFILE - MAIN ST - STA 12+70 TO STA 15+00

H: 1" = 10' V: 1" = 5'

PROFILE

NO.	DATE	BY	REVISIONS
	12/07/18	NMR	CONFORMED SET

NMR/CTF DESIGNED	03/23/2018 DATE
CTF DRAFTED	03/23/2018 DATE
NMR/JGR CHECKED	03/23/2018 DATE
JGR APPROVED	03/23/2018 DATE

REGISTERED PROFESSIONAL ENGINEER
88690PE
OREGON
MAY 21, 2014
NATHAN M. ROBINSON
RENEWAL DATE 12-31-18

CITY OF MILWAUKIE

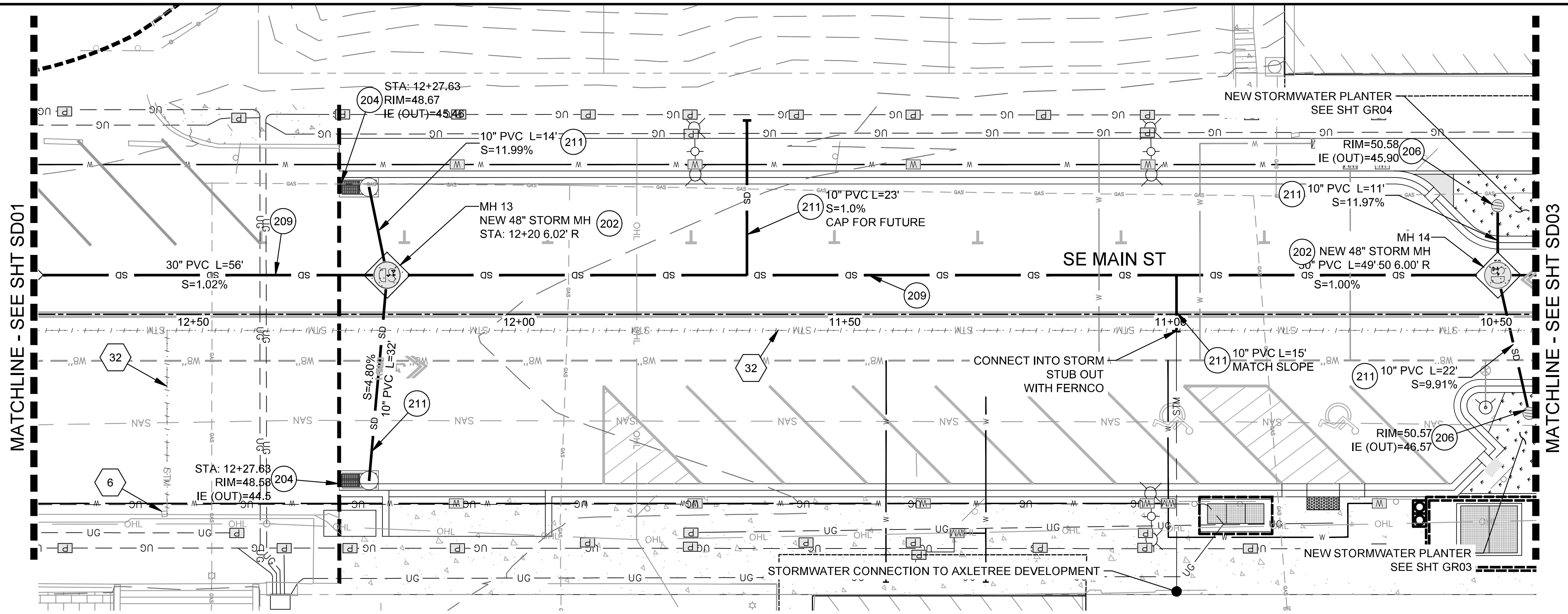
6101 SE JOHNSON CREEK BLVD.
MILWAUKIE, OR 97206

PHONE: 503-786-7600
FAX: 503-774-8236

MAIN STREET RECONSTRUCTION

STORM PIPE PLAN AND PROFILE
MAIN ST STA: 12+70 TO 15+00

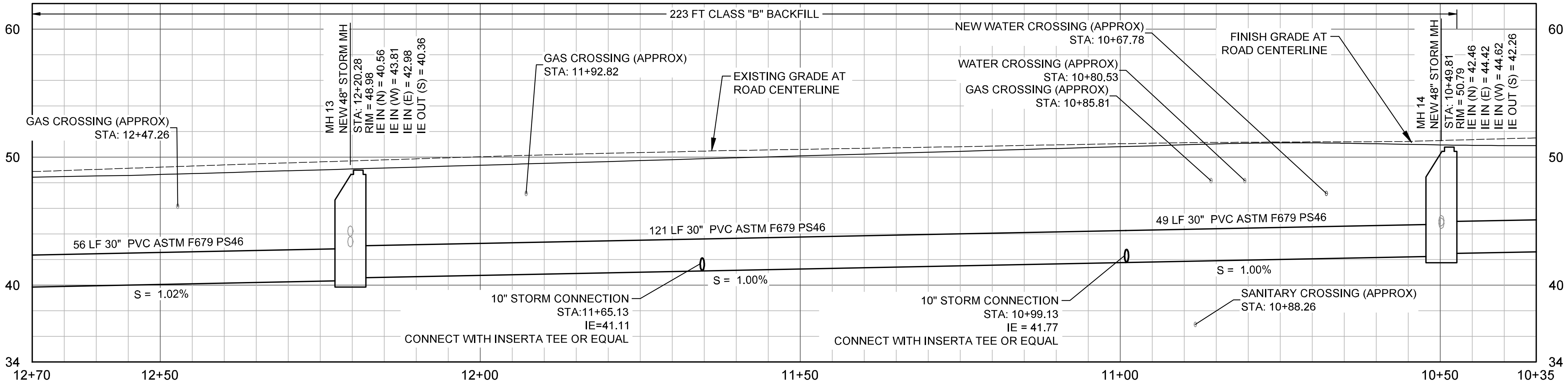
PROJECT NO.:	CONTRACT NO.:	DATE:	SHEET NO.:
CIP2018-Y18	C2018-051	08/14/2018	SD01



SCALE
0 10 20 FEET
1 INCH = 10 FEET

1 PLAN - MAIN ST - STA 10+50 TO 12+70

PLAN



2 PROFILE - MAIN ST - STA 10+50 TO 12+70

H: 1" = 10'
V: 1" = 5'

PROFILE

UTILITY CONSTRUCTION NOTES:

202	INSTALL 48" MANHOLE. SEE ODOT STANDARD DRAWING RD335.
204	INSTALL CONTECH STORMWATER CATCH BASIN. SEE SHEET DT08.
206	CONSTRUCT BEEHIVE INLET. SEE DTL 5 ON SHT DT11.
209	INSTALL 30" STORM PIPE
210	INSTALL 24" STORM PIPE.
211	INSTALL 10" STORM PIPE.
231	CONNECT EXISTING PIPE TO NEW MANHOLE.

DEMOLITION NOTES:

6	REMOVE EXISTING CATCH BASIN.
32	FILL AND ABANDON EXISTING STORM PIPE IN PLACE. SEE SHEET SD01-SD07 FOR MORE DETAILS.

NO.	DATE	BY	REVISIONS
	12/07/18	NMR	CONFORMED SET

NMR/CTF DESIGNED	03/23/2018 DATE
CTF DRAFTED	03/23/2018 DATE
NMR/JGR CHECKED	03/23/2018 DATE
JGR APPROVED	03/23/2018 DATE

REGISTERED PROFESSIONAL ENGINEER
88690PE
OREGON
MAY 21, 2014
NATHAN M. ROBINSON
RENEWAL DATE 12-31-18

CITY OF MILWAUKIE

6101 SE JOHNSON CREEK BLVD.
MILWAUKIE, OR 97206

PHONE: 503-786-7600
FAX: 503-774-8236

MAIN STREET RECONSTRUCTION

STORM PIPE PLAN AND PROFILE
MAIN ST STA: 10+50 TO 12+70

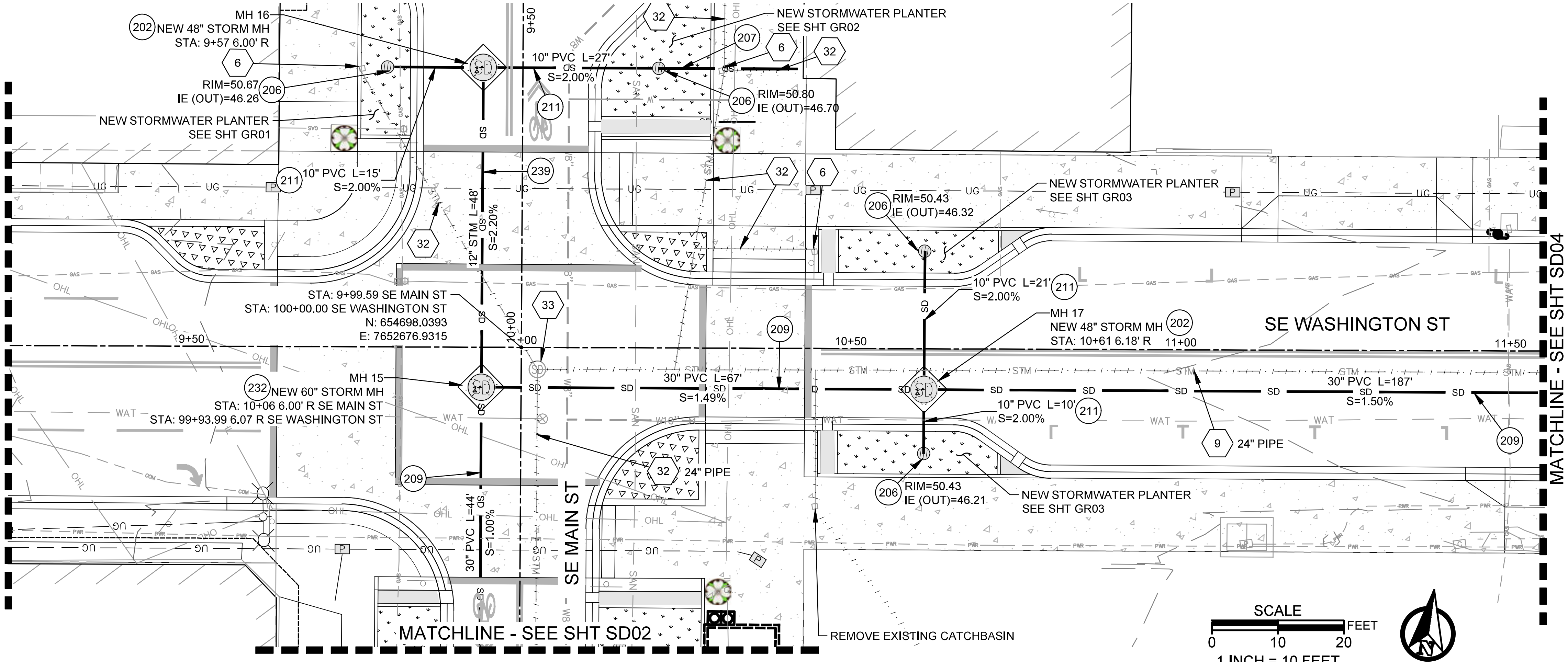
PROJECT NO.: CIP2018-Y18 | CONTRACT NO.: C2018-051 | DATE: 08/14/2018 | SHEET NO.: SD02

UTILITY CONSTRUCTION NOTES:

202	INSTALL 48" MANHOLE. SEE ODOT STANDARD DRAWING RD335.
206	CONSTRUCT BEEHIVE INLET. SEE DTL 5 ON SHT DT11.
207	CONNECT EXISTING ROOF DRAIN TO NEW CATCH BASIN WITH 6-INCH PVC PIPE.
209	INSTALL 30" STORM PIPE
210	INSTALL 24" STORM PIPE
211	INSTALL 10" STORM PIPE
231	CONNECT EXISTING PIPE TO NEW MANHOLE.
232	INSTALL 60" MANHOLE.
239	INSTALL 12" STORM PIPE

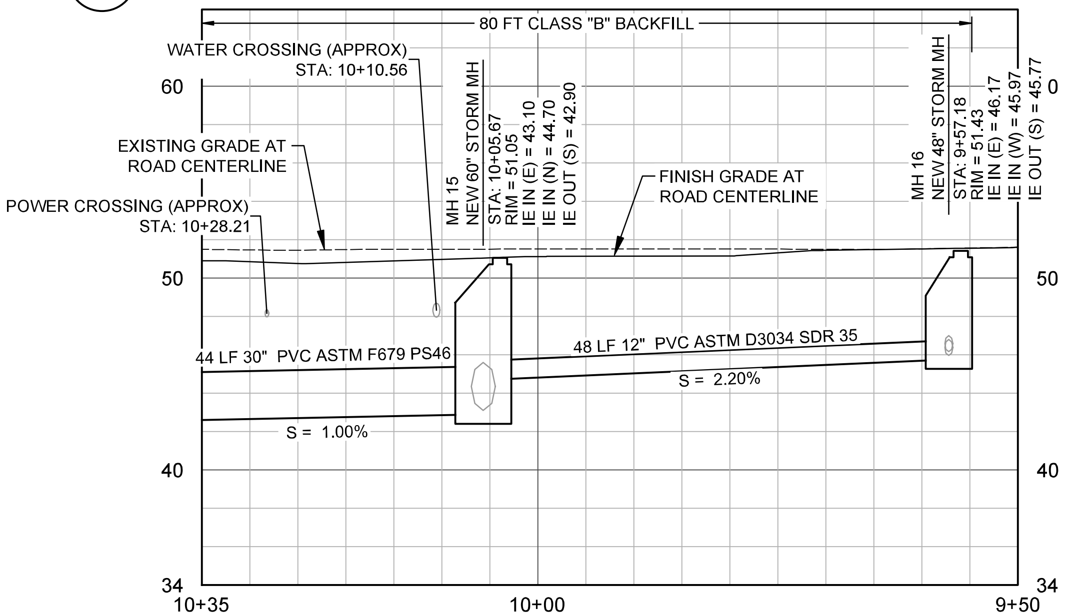
DEMOLITION NOTES:

6	REMOVE EXISTING CATCH BASIN.
9	REMOVE STORM PIPE.
32	FILL AND ABANDON EXISTING STORM PIPE IN PLACE. SEE SHEET SD01-SD07 FOR MORE DETAILS.
33	ABANDON EXISTING MANHOLE. REMOVE STRUCTURE TO DEPTH OF EXISTING LATERAL. SEE SHEET SD01-SD07 FOR MORE DETAILS.



1 PLAN - MAIN ST - STA 9+50 TO 10+50 AND WASHINGTON ST - STA 99+90 TO 101+45

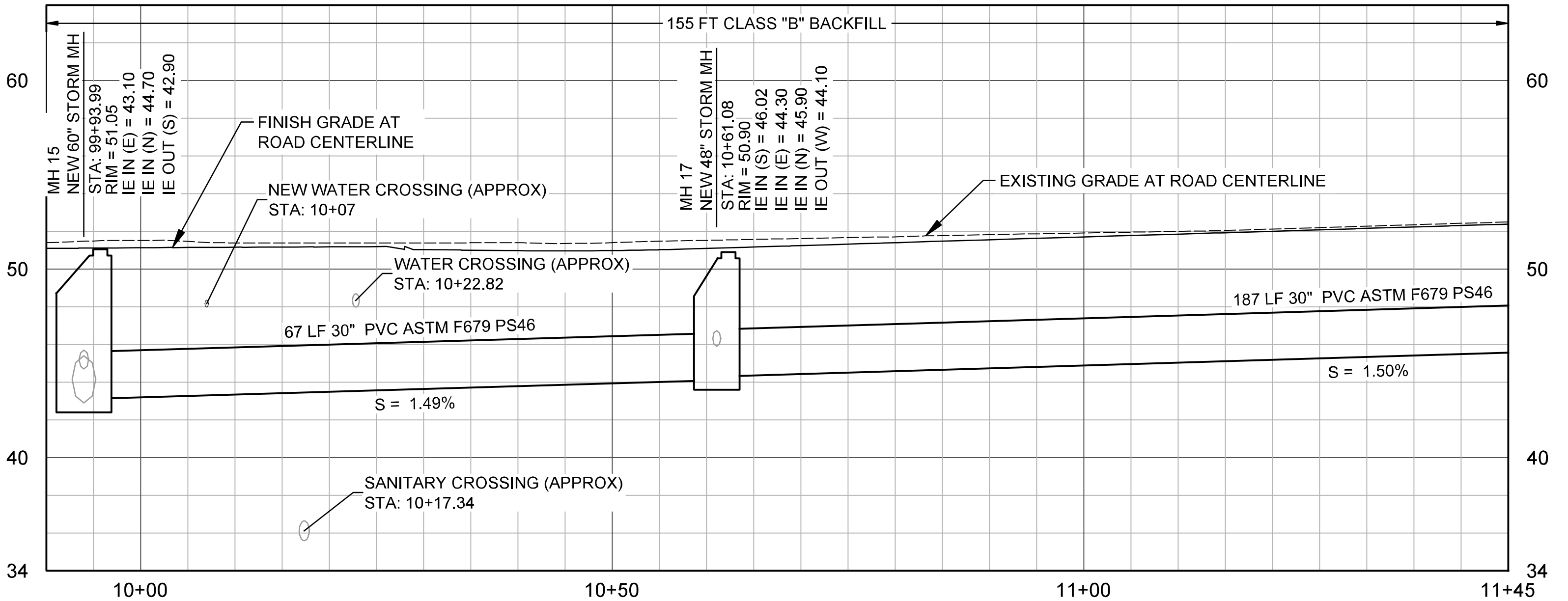
PLAN



2 PROFILE - MAIN ST - STA 9+50 TO 10+35

H: 1" = 10'
V: 1" = 5'

PROFILE



3 PROFILE - WASHINGTON ST - STA 9+90 TO 11+45

H: 1" = 10'
V: 1" = 5'

PROFILE

NO.	DATE	BY	REVISIONS
	12/07/18	NMR	CONFORMED SET

DESIGNED	NMR/CTF	03/23/2018
DRAFTED	CTF	03/23/2018
CHECKED	NMR/JGR	03/23/2018
APPROVED	JGR	03/23/2018

REGISTERED PROFESSIONAL ENGINEER
88690PE
OREGON
MAY 21, 2014
NATHAN M. ROBINSON
RENEWAL DATE 12-31-18

CITY OF MILWAUKIE

6101 SE JOHNSON CREEK BLVD.
MILWAUKIE, OR 97206

PHONE: 503-786-7600
FAX: 503-774-8236

MAIN STREET RECONSTRUCTION

STORM PIPE PLAN AND PROFILE
MAIN ST STA: 9+50 TO 10+50 AND WASHINGTON ST STA: 99+90 TO 101+45

PROJECT NO.: CIP2018-Y18
CONTRACT NO.: C2018-051
DATE: 08/14/2018
SHEET NO.: SD03

Composite Curve Number Calculations

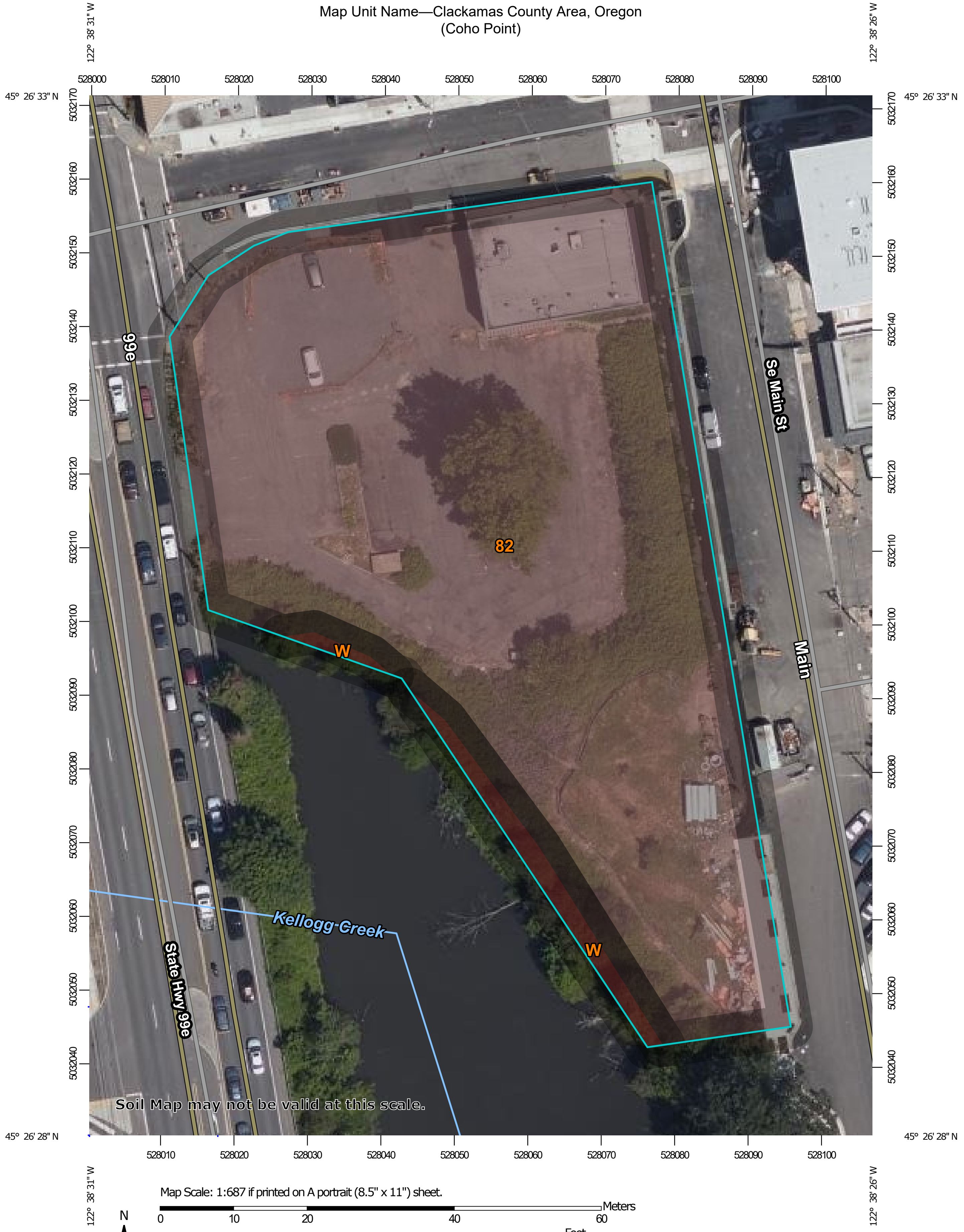
Subject	Coho Point Upstream Areas	By	MSG	Date	3/5/2021
Project	14464				

Composite CN Example Calculation for Upstream Basins

Soil Name and Hydrologic group	Cover Description (cover type, treatment, and hydrologic condition; percent impervious; unconnected/connect impervious area ratio)	Curve Number				Area (ac)	Product of CN **X** area
		HSG A	HSG B	HSG C	HSG D		
C	Open Space Good Condition (Amended Soils			56		15.01	840.56
C	Impervious surfaces-pavement, roofs, etc.			98		16.00	1568
		Totals				31.01	2409
		Use CN				78	

$$CN(Weighted) = \frac{Total\_\Pr oduct}{Total\_Area}$$

Map Unit Name—Clackamas County Area, Oregon
(Coho Point)



Map Scale: 1:687 if printed on A portrait (8.5" x 11") sheet.

Meters
0 10 20 40 60

N

Feet
0 30 60 120 180

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

MAP LEGEND

Area of Interest (AOI)

- ☐ Area of Interest (AOI)

Soils

Soil Rating Polygons

- 🟫 Urban land
- 🟥 Water
- ⬜ Not rated or not available

Soil Rating Lines

- 〰 Urban land
- 〰 Water
- 〰 Not rated or not available

Soil Rating Points

- ■ Urban land
- ■ Water
- □ Not rated or not available

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: Clackamas County Area, Oregon
Survey Area Data: Version 16, 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.

Map Unit Name

Map unit symbol	Map unit name	Rating	Acres in AOI	Percent of AOI
82	Urban land	Urban land	1.4	97.4%
W	Water	Water	0.0	2.6%
Totals for Area of Interest			**1.4**	**100.0%**

Description

A soil map unit is a collection of soil areas or nonsoil areas (miscellaneous areas) delineated in a soil survey. Each map unit is given a name that uniquely identifies the unit in a particular soil survey area.

Rating Options

Aggregation Method: No Aggregation Necessary

Tie-break Rule: Lower

GEODESIGN INC

REPORT OF GEOTECHNICAL ENGINEERING SERVICES

Coho Point at Kellogg Creek
11100 SE McLoughlin Boulevard
Milwaukie, Oregon

For
City of Milwaukie
September 24, 2018

GeoDesign Project: Milwaukie-7-01

GEODESIGN INC

September 24, 2018

City of Milwaukie
6101 SE Johnson Creek Boulevard
Milwaukie, OR 97206

Attention: Leila Aman

Report of Geotechnical Engineering Services
Coho Point at Kellogg Creek
11100 SE McLoughlin Boulevard
Milwaukie, Oregon
GeoDesign Project: Milwaukie-7-01

GeoDesign, Inc. is pleased to submit this report of geotechnical engineering services for the proposed Coho Point at Kellogg Creek development located southwest of the intersection of SE Main Street and SE Washington Street in Milwaukie, Oregon. This report has been prepared in accordance with our proposal dated July 10, 2018.

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

JTW:RSK:BAS:kt
Attachments
One copy submitted (via email only)
Document ID: Milwaukie-7-01-092418-geor.docx

EXECUTIVE SUMMARY

The following is a summary of our findings and recommendations for design and construction of the proposed development. This executive summary is limited to an overview of the project. We recommend that the report be referenced for a more thorough description of the subsurface conditions and geotechnical recommendations for the project.

- The underlying sand and silty sand are susceptible to liquefaction during a seismic event. Our analysis indicates liquefaction settlement of 4 inches is possible with differential settlement estimated equal to the total predicted settlement. If the buildings cannot tolerate this settlement, we recommend the soil beneath the buildings be improved to mitigate liquefaction or the buildings be supported on foundations that limit differential settlement.
- The near-surface soil at the site consists of variable fill and soft native soil that is not suitable to support the proposed building loads. Foundations, at a minimum, will need to be supported on improved soil.
- Floor slabs should be structurally supported by ground improvements or deep foundations to limit damage from seismic settlements. Alternatively, floor slabs can be installed directly on the existing subgrade; however, they will be subject to the liquefaction settlement described above.
- Because liquefiable soil is present at the site, the site class is F. This site class requires all building footings to be structurally tied together.
- Up to 20 feet of fill is present in portions of the site. The presence of undocumented fill can affect the performance of floor slabs and pavements at the site. We recommend that all subgrades be evaluated prior to placing base rock and pavements. If soft, loose, or deleterious material is encountered, we recommend that the material be over-excavated and replaced with crushed rock.
- The on-site soil can be sensitive to small changes in moisture content and difficult, if not impossible, to adequately compact during wet weather or when the moisture content of the soil is more than a couple of percent above the optimum required for compaction. As discussed in the report, the moisture content of the soil currently is above optimum and drying will be required if used as structural fill.
- The on-site soil will provide inadequate support for construction equipment during periods of wet weather or when above optimum moisture. Granular haul roads and working pads should be employed if earthwork will occur during the wet winter months.

TABLE OF CONTENTS PAGE NO.

TABLE OF CONTENTS **PAGE NO.**

ACRONYMS AND ABBREVIATIONS

AC	asphalt concrete
ASTM	American Society for Testing and Materials
BGS	below ground surface
CAPWAP	case pile wave analysis program
g	gravitational acceleration (32.2 feet/second^2)
H:V	horizontal to vertical
IBC	International Building Code
MCE	maximum considered earthquake
NAVD	North American Vertical Datum
OSHA	Occupational Safety and Health Administration
pcf	pounds per cubic foot
pci	pounds per cubic inch
PDA	Pile Driving Analyzer®
PGA	peak ground acceleration
psf	pounds per square foot
SPT	standard penetration test
SOSSC	State of Oregon Structural Specialty Code
UST	underground storage tank
WEAP	wave equation analysis program

1.0 INTRODUCTION

This report provides geotechnical engineering recommendations for the proposed Coho Point at Kellogg Creek development located southwest of the intersection of SE Main Street and SE Washington Street in Milwaukie, Oregon. Figure 1 shows the site relative to existing topographic and physical features. Figure 2 shows the existing conditions and our approximate exploration locations.

The exploration logs and laboratory test results are presented in the Appendix. Acronyms and abbreviations used herein are defined above, immediately following the Table of Contents.

2.0 PROJECT UNDERSTANDING

We understand that the project includes Tax Lots 1100, 1200, 1300, 1301, and 1302. The proposed development will include mixed-use buildings likely constructed at grade. The structures will have four to five stories of wood framing over two stories of concrete. The preliminary building loads provided to us indicate maximum column and wall loads of 530 kips and 15 kips per linear foot, respectively. We anticipate floor loads will be less than 150 psf.

We understand the building on Tax Lot 1200 will be demolished as part of the project. The south side of the site is currently within the flood plain; site grades may be raised to remove it from the flood plain.

3.0 PURPOSE AND SCOPE

The purpose of our services is 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 explorations, including public and private utility locates, access preparation, and scheduling of contractors and GeoDesign staff.
- Conducted a subsurface exploration program that consisted of drilling three borings to depths between 30.5 and 86.3 feet BGS.
- Maintained continuous logs of the explorations and collected soil samples at representative intervals.
- Performed a laboratory testing program that included the following:
    - Seventeen moisture content determinations in general accordance with ASTM D2216
    - Two particle-size analyses in general accordance with ASTM D1140
    - Two Atterberg limits tests in general accordance with ASTM D4318
- Provided recommendations for site preparation and grading, including temporary and permanent slopes, fill placement criteria, suitability of on-site soil for fill, and subgrade preparation.
- Provided recommendations for wet weather construction.

- Provided foundation support options for the proposed buildings. Our recommendations include preferred foundation type, allowable bearing pressure, and lateral resistance parameters.
- Provided recommendations for floor slab support.
- Provided recommendations for use in design of conventional retaining walls, including backfill and drainage requirements and lateral earth pressures.
- Evaluated groundwater conditions at the site and provided general recommendations for dewatering during construction and subsurface drainage (if required).
- Provided seismic design recommendations in accordance with the procedures outlined in the 2012 IBC and 2014 SOSSC.
- Prepared this geotechnical engineering report that presents our findings, conclusions, and recommendations.

4.0 SITE CONDITIONS

4.1 GEOLOGIC SETTING

The site is located in the southernmost part of the Portland Basin physiographic province, which is a smaller basin within the Willamette Valley-Puget Sound Lowland. The lowland is a tectonically active forearc basin located along the convergent Cascadia margin (Orr and Orr, 1999). The Portland Basin is bound by the Tualatin Mountains to the west and south and the Cascade Range to the east and north.

Surface geology at the site is mapped as catastrophic flood deposits resulting from the Missoula Floods, a series of catastrophic floods caused by the repeated failure of a glacial ice dam that impounded glacial Lake Missoula in present day Montana during the Pleistocene. (Beeson et al., 1989). These floods swept across eastern Washington and followed the Columbia River channel out to sea, backfilling the Willamette Valley with flood waters and sediment during each event. Beeson et al. (1989) classify the flood deposits at the site as channel facies (variable silts, sands, and gravels deposited in major flood pathways and re-worked by subsequent events). Beeson et al. (1989) further note that irregular surfaces, abandoned drainages, and scours left by the floods at this location have been filled in by bog and pond sediment, as well as sediment transported by local creeks, including Kellogg Creek.

Bedrock at the site is mapped as the Basalt of Waverly Heights, a sequence of subaerial basaltic lava flows and oceanic sediments, deposited as part of an island arc during the Eocene (approximately 40 million years ago) and subsequently accreted to western Oregon (Madin, 2004).

4.2 SURFACE CONDITIONS

The site consists of approximately 0.81 acre located in downtown Milwaukie. It is bound by SE Washington Street to the north, SE Main Street to the east, and SE McLoughlin Boulevard to the west. Kellogg Creek and an adjacent public park (Dogwood Park) border the site to the south. A two-story structure with a daylight basement occupies the northeast corner of the parcel; we understand this structure will be demolished as part of the project. Most of the site consists of an AC parking lot. Two large deciduous trees occupy a planter area in the center of the site, and small shrubs, deciduous trees, and brush are present along the bank of Kellogg Lake. The site generally slopes gently towards Kellogg Lake to the south with elevations between 35 and 42 feet (NAVD88). There is an approximately 12-foot-high 1.5H:1V to 1.7H:1V slope along the east site boundary and a 1.3H:1V to 2H:1V bank down to Kellogg Lake.

4.3 SUBSURFACE CONDITIONS

4.3.1 General

We completed three borings (B-1 through B-3) at the site to depths between 30.5 and 86.3 feet BGS. The approximate locations of our explorations are shown on Figure 2. Descriptions of the field exploration and laboratory testing programs, logs of the explorations, and results of laboratory testing presented in the Appendix.

Subsurface conditions generally consist of fill underlain by alluvial and flood deposits overlying weathered basalt. A more detailed description of the subsurface conditions at the site is presented below.

4.3.2 Fill

Undocumented fill was observed in all borings completed at the site. The fill generally consists of silt with gravel, gravelly silt, silty gravel, and gravel with silt. Trace amounts of concrete and metal debris were observed in boring B-1 within the silty gravel. The fill extends to depths between approximately 4.5 and 20.2 feet BGS. Moisture contents varied from 12 to 30 percent at the time of our explorations.

4.3.3 Alluvial Silt and Sand

Underlying the fill are layers of gray to dark gray silt, sand, and organic silt. The silt is generally sandy to with sand, and the sand is generally silty. Within this section clayey sand and organic silt were encountered. Based on SPT blow counts the silt is generally very soft to medium stiff and the sand is very loose to loose. The natural moisture content varied from 40 to 82 percent at the time of our explorations.

4.3.4 Weathered Basalt

Underlying the alluvial soil is medium dense to very dense, silty gravel to gravel with silt that we interpret to be weathered basalt of the Waverly Heights Formation. The depth to weathered basalt appears to very drastically across the site with decomposed basalt encountered at 83.0 and 21.0 feet BGS in borings B-1 and B-2, respectively.

4.3.5 Groundwater

We did not observe groundwater in our borings due to the mud rotary drilling techniques used. Samples were generally observed to be wet at depths of approximately 15 feet BGS and below. Groundwater is anticipated to correspond to the level of Kellogg Lake to the south. The existing conditions survey completed by Statewide Land Surveying Inc. indicates that the ordinary high water line for Kellogg Lake is at an elevation of 26 feet (NAVD88).

The depth to groundwater may fluctuate in response to seasonal changes, prolonged rainfall, changes in surface topography, and other factors not observed in this study. We anticipate that perched water may be present within a few feet of the ground surface during the wet season or during extended periods of precipitation.

4.4 GEOLOGIC HAZARDS

4.4.1 Liquefaction

Liquefaction is caused by a rapid increase in pore water pressure that reduces the effective stress between soil particles to near zero. 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.

We performed a liquefaction analysis for the site using the data collected from the field explorations and our laboratory testing program. We considered both subduction zone and crustal earthquake scenarios. For our analysis, we modeled a subduction zone earthquake as a magnitude 9.0 event with a PGA of 0.20 g. We modeled a crustal earthquake as a magnitude 6.8 event with a PGA of 0.42 g. We assumed groundwater was present at a depth of 12 feet BGS. We evaluated the liquefaction potential using the method proposed by Boulanger and Idriss (2014) employing the depth weighting methods from Cetin (2009).

Based on our analysis, the silty sand and sand layers are susceptible to liquefaction during a design-level earthquake. Our analysis indicates that total liquefaction settlement of approximately 4 inches is possible with differential settlement estimated equal to the total predicted settlement.

If these seismic settlements cannot be tolerated, we recommend the soil beneath the buildings be improved to mitigate liquefaction or the buildings be supported on foundations that limit differential settlement.

4.4.2 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 a river or creek bank. Liquefied soil adjacent to an open face can flow toward the open face, resulting in lateral ground displacement. The primary difference between a conventional slope stability failure and lateral spreading is that no distinct failure plane is formed during a lateral spreading event. Liquefied soil flows downslope or to an exposed bank similar to the behavior of a viscous fluid. We expect the risk of lateral spreading to be low as gravel was encountered in boring B-3 to 20 feet BGS. This assumes the depth of Kellogg Creek is shallower than this.

4.4.3 Fault Surface Rupture

The Portland Hills fault is mapped approximately 0.72 mile northeast of the site (Beeson et al., 1991; Madin, 1990). Consequently, it is our opinion that the probability of surface fault rupture beneath the site is low.

5.0 CONCLUSIONS AND RECOMMENDATIONS

5.1 GENERAL

Based on our geotechnical evaluation, we conclude that the site is feasible for development provided the site is prepared as recommended in this report. The “Executive Summary” provides a brief overview of the primary geotechnical considerations for the project. Our specific recommendations are presented in the following sections.

5.2 SITE PREPARATION

5.2.1 Stripping and Grubbing

Stripping and grubbing will be required at this site to remove the trees and shrubs in the landscaped areas adjacent to the parking areas. The existing root zone material should be removed from all proposed structure and pavement areas. The actual stripping and grubbing depth should be based on field observations at the time of construction. Stripping and grubbing should extend at least 5 feet beyond the limits of proposed structural areas. Organic material should be transported off site for disposal or used as fill in landscaped areas.

5.2.2 Demolition

Demolition includes complete removal of the existing buildings, concrete pavement, sidewalks, utilities, USTs, and any other underlying structural elements. The slab, walls, and footings of the existing building should be completely removed. Any monitoring wells or USTs should be abandoned in accordance with state and local regulations prior to site redevelopment. Abandoned utility lines under new structural components should be completely removed. Excavations resulting from the demolition of existing improvements should be backfilled with compacted structural fill as recommended in this report. The base of the excavations should be excavated to expose firm subgrade. The sides of the temporary excavations should be cut into firm material and sloped no steeper than 1½H:1V.

5.2.3 Undocumented Fill

Undocumented fill was observed in all borings completed at the site to depths ranging approximately 4.5 to 20.2 feet BGS. The fill is of variable composition and some old construction debris, including concrete and metal fragments, were observed within it. We recommend that the undocumented fill be evaluated during construction where it exists beneath slabs, pavements, and other structures to determine if over-excavation will be necessary.

5.2.4 Subgrade Evaluation

A member of our geotechnical staff should observe the exposed footing, slab, and pavement subgrade after stripping, excavation, and placement of structural fill have been completed to confirm that there are no areas of unsuitable or unstable soil. The subgrade should be evaluated using moisture-density testing, a hand probe, and/or proof rolling with a fully loaded dump truck (or similar heavy, rubber tire construction equipment). If soft, loose, or otherwise unsuitable soil is found at the subgrade level, we recommend that the soil be over-excavated and replaced with structural fill.

5.3 EXCAVATION

Excavations will be required for the installation of foundation elements, utilities, and other earthwork. Conventional earthmoving equipment in proper working condition should be capable of making the necessary excavations. It is possible that buried obstructions may be encountered, which could result in difficult excavation conditions and trenches being wider than anticipated. Excavations in the silt and sand may be prone to raveling. Excavations deeper than 4 feet BGS will require shoring or should be sloped. Sloped excavations may be used to vertical depths of 10 feet BGS and should have side slopes no steeper than 1½H:1V, provided groundwater seepage does not occur. We recommend a minimum horizontal distance of 5 feet from the edge of the existing improvements to the top of any temporary slope. All cut slopes should be protected from erosion by covering them during wet weather. If seepage, sloughing, or instability is observed, the slope should be flattened or shored. Shoring will be required where slopes are not possible. The contractor should be responsible for selecting the appropriate shoring system.

Excavations should not be allowed to undermine adjacent improvements. If existing roads or structures are located near a proposed excavation, unsupported excavations can be maintained outside of a 1H:1V downward projection that starts 5 feet from the base of the existing footings. Excavations that must be inside of this zone should be supported by temporary or permanent shoring designed for moment resistance for the full height of the excavation, including kick-out for the full buried depth of the retaining system.

We anticipate that excavations for this project will not extend below the groundwater level. We anticipate that significant dewatering will not be required for this project. Perched water or rainwater can likely be removed by pumping from sumps located within the excavation.

While we have described certain approaches to performing excavations, it is the contractor's responsibility to select the excavation and dewatering methods, monitor the excavations for safety, and provide any shoring required to protect personnel and adjacent improvements. All excavations should be in accordance with applicable OSHA and state regulations.

5.4 PERMANENT SLOPES

Permanent cut or fill slopes should not exceed a gradient of 2H:1V, unless specifically evaluated for stability. Upslope buildings, access roads, and pavements should be set back a minimum of 5 feet from the crest of such slopes. Slopes should be planted with appropriate vegetation to provide protection against erosion as soon as possible after grading. Surface water runoff should be collected and directed away from slopes to prevent water from running down the face of the slope.

5.5 STRUCTURAL FILL

Structural fill includes fill beneath foundations, slabs, pavements, any other areas intended to support structures, or within the influence zones of structures. Structural fill should be free of organic matter and other deleterious material and, in general, should consist of particles no larger than 3 inches in diameter. Recommendations for suitable fill material are provided in the following sections.

5.5.1 On-Site Soil

The on-site silt, sand, and silty gravel soil will be suitable for use as structural fill only if it can be moisture conditioned. Based on our experience, the soil is very sensitive to small changes in moisture content and may be difficult, if not impossible, to compact. Laboratory testing indicates that the moisture content of the on-site soil is significantly greater than the anticipated optimum moisture content required for satisfactory compaction. Therefore, this soil may require extensive drying if it is used as structural fill. We recommend using imported granular material for structural fill if the moisture content of the on-site soil cannot be reduced.

5.5.2 Imported Granular Material

Imported granular material should be pit- or quarry-run rock, crushed rock, or crushed gravel and sand that is fairly well graded between coarse and fine and has less than 5 percent by dry weight passing the U.S. Standard No. 200 sieve. All granular material must be durable such that there is no degradation of the material during and after installation as structural fill. The percentage of fines can be increased to 12 percent if the fill is placed during dry weather and provided the fill material is moisture conditioned, as necessary, for proper compaction. The material should be placed in lifts with a maximum uncompacted thickness of 8 inches and compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D 1557. During the wet season or when wet subgrade conditions exist, the initial lift should have a maximum thickness of 12 inches and compacted by rolling with a smooth-drum, non-vibratory roller.

5.5.3 Recycled Concrete

Recycled concrete from the existing building foundations can be used for structural fill provided the concrete is broken to a maximum particle size of 6 inches. This material can be used as trench backfill and pavement base rock if it meets the requirements for imported granular material, which would require a smaller maximum particle size. The material should be placed in lifts with a maximum uncompacted thickness of 12 inches and compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D 1557.

5.5.4 Trench Backfill Material

Trench backfill for the utility pipe base and pipe zone should consist of durable, well-graded granular material that has a maximum particle size of 1 inch, has less than 5 percent by dry weight passing the U.S. Standard No. 200 sieve, and does not contain organic or other deleterious material. Backfill above the pipe zone should meet the requirements above, except that the maximum particle size may be increased to 1½ inches.

Backfill for the pipe base and within the pipe zone should be placed in maximum 12-inch-thick lifts and compacted to not less than 90 percent of the maximum dry density, as determined by ASTM D 1557, or as recommended by the pipe manufacturer. Backfill above the pipe zone should be placed in maximum 12-inch-thick lifts and compacted to not less than 92 percent of the maximum dry density, as determined by ASTM D 1557. Trench backfill located within 2 feet of finish subgrade elevation should be placed in maximum 12-inch-thick lifts and compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D 1557. Outside of structural areas, trench backfill material should be compacted to at least 90 percent of the maximum dry density, as determined by ASTM D 1557.

5.6 DRAINAGE

5.6.1 Surface

Pavement surfaces and open space areas should be sloped such that surface water runoff is collected and routed to suitable discharge points. The finished ground surface around the buildings should be sloped away from foundations at a minimum 2 percent gradient for a distance of at least 5 feet. Runoff water should not be directed to the top of the slope.

5.6.2 Temporary

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

6.0 FOUNDATION SUPPORT RECOMMENDATIONS

6.1 GENERAL

As described in the "Geologic Hazards" section, the soil at the site is potentially liquefiable during a seismic event. Our analysis indicates liquefaction settlement at the ground surface on the order of 4 inches is possible with differential settlement estimated equal to the total predicted settlement.

These seismic settlements may be acceptable for the proposed structures to be founded on conventional spread footings underlain by ground improvement such as short rammed aggregate piers. If liquefaction settlement exceeds building tolerances for conventional spread footings or a mat foundation, the soil will need to be improved.

We note that pavement and landscaped areas will also experience liquefaction; however, they are not typically mitigated for liquefaction due to the high costs of ground improvement and lower cost for pavement (and potentially utility) repair.

The near-surface soil at the site consists of variable fill and very soft to soft native soil that is not suitable to support the proposed building loads. Foundations, at a minimum, regardless of liquefaction settlement, will need to be founded on improved soil.

Options for supporting the proposed buildings are discussed below. Based on the Site Class F designation, all building footings must be structurally tied together.

1. Improve the soil beneath the foundations and floor slabs and support the structures on a mat. Soil improvement would most likely include stone columns or deep soil mix columns that extend through the liquefiable zone and extend to a depth of approximately 50 feet.

2. Support the structures on deep foundations consisting of driven piles or drilled shafts that extend through the compressible and liquefiable soil.

6.2 FOUNDATIONS ON GROUND IMPROVEMENT

6.2.1 Stone Columns

Stone columns can be used to mitigate liquefaction and provide support for the proposed structures on a mat underlain by improved ground, provided that resulting settlement from the structure loads are within suitable tolerances. Design of stone columns should be performed by a specialty contractor. Typically, stone columns beneath structural elements are installed on 7-

to 9-foot centers, with diameters that vary from 36 to 42 inches. Additional explorations may be completed by design-build contractors to assist in design of soil improvement. GeoDesign can also provide these services.

6.2.2 Deep Soil Mix Columns

Soil mixing consists of drilling into the soil using a specialty drill rig that injects cement slurry into the ground. Paddles along the shaft blend the soil and cement slurry together until a relatively uniform column of soil and cement is formed. A mat foundation can be constructed directly on top of the columns similar to stone columns. The allowable bearing pressure for shallow foundations supported on deep soil mix columns is typically 4,000 to 6,000 psf. Soil mix columns are typically between 36 and 60 inches in diameter and installed on a regular or semi-regular layout under the spread footings and floor slabs. Spoils generated during installation can be used as on-site fill or hauled off site following approval and environmental profiling, which should be identified in the project Contaminated Media Management Plan. Soil mix columns are more rigid than stone columns, can support larger loads, and more efficiently mitigate liquefaction in fine-grained soil.

6.2.3 Rammed Aggregate Piers

Rammed aggregate pier foundation systems consist of compacted aggregate piers that reinforce and improve the soil. These systems are proprietary and designed and constructed by a specialty contractor. Conventional spread foundations are placed over the completed rammed aggregate piers. The allowable bearing pressure for shallow foundations supported on rammed aggregate piers is typically 4,000 to 6,000 psf.

We anticipate that static foundation settlement of the rammed aggregate pier foundation system will be less than 1 inch with differential settlement of 0.5 inch. Seismic settlement will be as described in the "Geologic Hazards" section. The design-build contractor should be provided with this report to complete settlement analysis for the aggregate piers.

6.2.4 Spread Footings on Soil Improvements or Rammed Aggregate Piers

6.2.4.1 Dimensions and Capacities

Footings established on improved soil as described above can be used to support structures at the site. Footings should be proportioned on an allowable bearing pressure provided by the soil improvement contractor. For preliminary purposes, we estimate that the allowable bearing pressure for footings installed on stone columns, deep soil mix columns, or rammed aggregate piers will be as discussed above. The specialty contractor will specify the allowable bearing pressure.

Continuous wall and isolated spread footings should be at least 16 and 20 inches wide, respectively. The bottom of exterior footings should be at least 18 inches below the lowest adjacent exterior grade. The bottom of interior footings should be established at least 12 inches below the base of the slab.

6.2.4.2 Resistance to Sliding

Lateral loads on footings can be resisted by passive earth pressure on the sides of the structures and by friction on the base of the footings. Our analysis indicates that the available passive earth pressure for footings confined by on-site soil and structural fill is 300 pcf, modeled as an equivalent fluid pressure. Adjacent floor slabs, pavements, or the upper 12-inch depth of adjacent, unpaved areas should not be considered when calculating passive resistance. A coefficient of friction equal to 0.4 can be used for the resistance to sliding for footings in contact with the improved soil.

6.3 DRIVEN PILE FOUNDATIONS

As an alternative to ground improvement, driven steel or grout piles can be used to support the structures. The piles will obtain the majority of their capacity through end bearing in the underlying weathered basalt. Pile lengths will vary across the site as the weathered basalt unit was encountered at depths between approximately 21.0 and 83.0 feet BGS. Due to the variation in depth to the gravel across the site, we recommend the piling contractor install indicator piles to help define the required length of piles during early production driving. The following sections provide specific design recommendations for deep foundations

6.3.1 Downward Axial Capacity

Table 1 presents the calculated allowable compressive capacity of steel pipe and driven grout piles driven to basalt bedrock. The allowable capacities in Table 1 assume the piles are spaced at least 3 pile diameters on-center.

Table 1. Axial Allowable Compressive Capacity

Pile Type	Allowable Compressive Capacity (tons)
12-inch-diameter steel pipe pile (closed-end)	220
18-inch-diameter steel pipe pile (closed-end)	220
16-inch-diameter driven grout pile	160
18-inch-diameter driven grout pile	220

1. Lower and higher value assumes weathered basalt at 21.0 and 83.0 feet BGS, respectively.

The capacity of driven steel piling will be limited by the structural capacity of the pile section. High-strength, 12-inch-diameter steel pipes have been used in the Portland Metropolitan area and have achieved an allowable capacity of 220 tons. These types of piles are not readily available. Eighteen-inch-diameter pipe piles are more easily available and can achieve an allowable capacity of 220 tons.

A factor of safety of 2 was used in our analysis; therefore, verification of capacity will be required in the field using a PDA and full-time observation during pile driving for both steel pipe and driven grout piles. PDA testing on a driven grout pile will require installation and re-driving of a sacrificial pile to conduct the PDA testing.

All piles should be driven to refusal on the gravel or the terminal driving criteria as determined by PDA and CAPWAP analysis, whichever is less.

Depth of penetration of the piles will vary depending on the depth and consistency of the weathered basalt unit; however, we estimate that 5 to 10 feet of penetration into the weathered basalt unit will achieve the allowable compressive capacities presented in Table 1. Pipe piles should be driven closed-ended with steel plates designed to withstand the force caused by hard driving into the weathered basalt unit.

6.3.2 Uplift Resistance

Uplift capacity of the piles will be mobilized through skin friction between the pile and the surrounding soil for the length of the pile installed into the underlying weathered basalt unit. We compute the following allowable uplift capacity for each pile type.

Table 2. Allowable Uplift Capacity

Pile Type	Allowable Uplift Capacity[1] (tons)
12-inch-diameter steel pipe pile (closed-end)	40
18-inch-diameter steel pipe pile (closed-end)	60
16-inch-diameter driven grout pile	65
18-inch-diameter driven grout pile	75

1. Assumes minimum 5 feet embedment in weathered basalt

The computed uplift capacity should assume the pile will penetrate no more than 5 feet into the weathered basalt and the piles are spaced at least 3 pile diameters on-center. These uplift capacities may not be achievable due to drilling refusal. Supplemental anchors may be necessary to resist uplift.

6.3.3 Lateral Resistance

Resistance to lateral loads can be developed by passive pressure on the face of pile caps, grade beams, tie beams, and other buried foundation elements. Sliding friction on the base of pile-supported foundation elements should be ignored. Assuming a minimum translation of 1.0 inch, the allowable passive resistance on the face of buried foundation elements may be computed using an equivalent fluid pressure of 300 pcf for foundation elements cast neat against the existing soil or backfilled with structural fill. Adjacent floor slabs, pavements, or the upper 12-inch depth of adjacent, unpaved areas should not be considered when calculating passive resistance. We will provide the design team with lateral pile response curves when the pile size has been selected.

6.3.4 Other Considerations

The terminal blow counts will depend on the pile type and driving equipment. The structural integrity of the steel pipe pile or the mandrel should be evaluated to confirm that they will withstand the stresses induced by pile driving. GeoDesign should be consulted to select the appropriate hammer energy to develop the required capacity while avoiding excessive driving stresses. Terminal blow criteria should be based on WEAP analysis considering the pile type, required capacity, and the selected driving equipment. Our analysis should be verified in the field using a PDA.

The piling should be installed with suitable alignment tolerances. Vertical alignment should be within 3 percent of plumb or as determined by the structural engineer. Lateral alignment should be within tolerances determined by the structural engineer, considering the pile cap design. Settlement of piles driven to refusal in the lower gravel will be negligible beyond the elastic compression of the pile.

If buried obstructions are encountered during driving, the pile should be extracted and the obstruction removed. If the buried obstruction cannot be removed, the structural engineer should be consulted to select a new pile location. Each pile should be carefully inspected for damage caused by impacting buried obstructions during driving.

We recommend full-time monitoring of pile installation to confirm that the piles are driven in accordance with the recommendations in this report and with the project specifications.

7.0 SLABS ON GRADE

If slabs on grade will be constructed for this project, satisfactory subgrade support for slabs supporting floor loads of up to 150 psf can be obtained on the near-surface soil or on structural fill. If fill is present at the slab subgrade level, we recommend that the fill be evaluated during construction to determine if scarifying and re-compaction or over-excavation will be required.

A minimum 6-inch-thick layer of imported granular material should be placed and compacted over the prepared subgrade to assist as a capillary break. The imported granular material should have a maximum particle size of 1½ inches, less than 5 percent by dry weight passing the U.S. Standard No. 200 sieve, and at least two mechanically fractured faces. The imported granular material should be placed in one lift and compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D1557.

A soil subgrade modulus of 120 pci should be used to design floor slabs supported on near-surface soil or structural fill. Settlement of the slab supporting the anticipated design loads and constructed as recommended is not expected to exceed approximately 1 inch of total and differential settlement.

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

8.0 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. This value is based on the assumption that (1) the retained soil is level, (2) the retained soil is drained, and (3) the wall is less than 15 feet in height. If retaining walls with more than one level of bracing will be constructed, GeoDesign should be contacted to provide additional recommendations. If surcharges (i.e., retained slopes, foundations, vehicles, etc.) are located within a horizontal distance of twice the height of the wall from the back of the wall, additional pressures will need to be account for in the wall design. Our office should be contacted for the appropriate wall surcharges based on the actual magnitude and configuration of the applied loads. Seismic lateral forces can be calculated using a dynamic force equal to $7.5H^2$ 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.

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.

9.0 SEISMIC DESIGN CRITERIA

Seismic design is prescribed by the 2014 SOSSC and 2012 IBC. Table 3 presents the site design parameters prescribed by the 2012 IBC for the site. The building code require that seismic design parameters associated with a percent probability of being exceeded in a 50-year period be used in design.

Due to the potential for liquefaction, the site is considered a Site Class F. When using the code-based seismic design parameters and provided the buildings have a fundamental period of less than 0.5 second, a Site Class E can be used when completing a site-specific analysis. Table 3 provides the IBC seismic design parameters for the site.

Table 3. 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 = 0.983 g	S_1 = 0.421 g
Site Class	F	
Site Coefficient, F	F_a = 0.920	F_v = 2.400
MCE Spectral Acceleration Parameters, S_M	S_{MS} = 0.905 g	S_{M1} = 1.010 g
Design Spectral Response Acceleration Parameters	S_{DS} = 0.603 g	S_{D1} = 0.673 g

10.0 CONSTRUCTION CONSIDERATIONS

10.1 WET WEATHER CONSTRUCTION

Trafficability of soil at the ground surface may be difficult during extended wet periods or when the moisture content of the surface soil is more than a few percentage points above optimum. At the time of our explorations, the moisture contents were significantly higher than optimum. If not carefully executed, the earthwork activities can create extensive soft areas, resulting in significant repair costs. When the subgrade is wet, site preparation may need to be accomplished using track-mounted equipment and loading material into trucks supported on granular haul roads.

Haul roads and working blankets will be required to support construction equipment when the subgrade is wet of optimum. Based on our experience, at least 12 inches of granular material is typically required for light staging areas and at least 18 inches of granular material for haul roads subject to repeated equipment traffic. We typically recommend that imported granular material for haul roads and working blankets consist of durable crushed rock that is well graded and has less than 8 percent by dry weight passing the U.S. Standard No. 200 sieve. Where silt is exposed at the ground surface, a geotextile should be placed on the subgrade before placing the granular material. The granular material should be placed in a single lift and the surface compacted until well keyed. Although we have presented typical recommendations for haul road and working blankets, the actual thickness and material should be determined by the contractor based on their sequencing of the project and the type and frequency of construction equipment.
The base rock thickness for pavement and structural slab areas is intended to support post-construction design loads and will not support construction traffic or pavement construction when the subgrade soil is wet. If construction is planned for periods when the subgrade soil is wet, an increased thickness of base rock will be required.

10.2 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 permanent 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.

11.0 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.

12.0 LIMITATIONS

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

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

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

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

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

◆ ◆ ◆

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

Sincerely,

GeoDesign, Inc.

Joe Westergreen

Joe Westergreen, P.E. (Washington)
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

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FIGURES

Printed By: aday | Print Date: 8/30/2018 9:46:39 AM
File Name: J:\M-R\Milwaukie\Milwaukie-7\Milwaukie-7-01\Figures\CAD\Milwaukie-7-01-VM01.dwg | Layout: FIGURE 1

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

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MILWAUKIE-7-01

SEPTEMBER 2018

VICINITY MAP

COHO POINT AT KELLOGG CREEK
MILWAUKIE, OR

FIGURE 1

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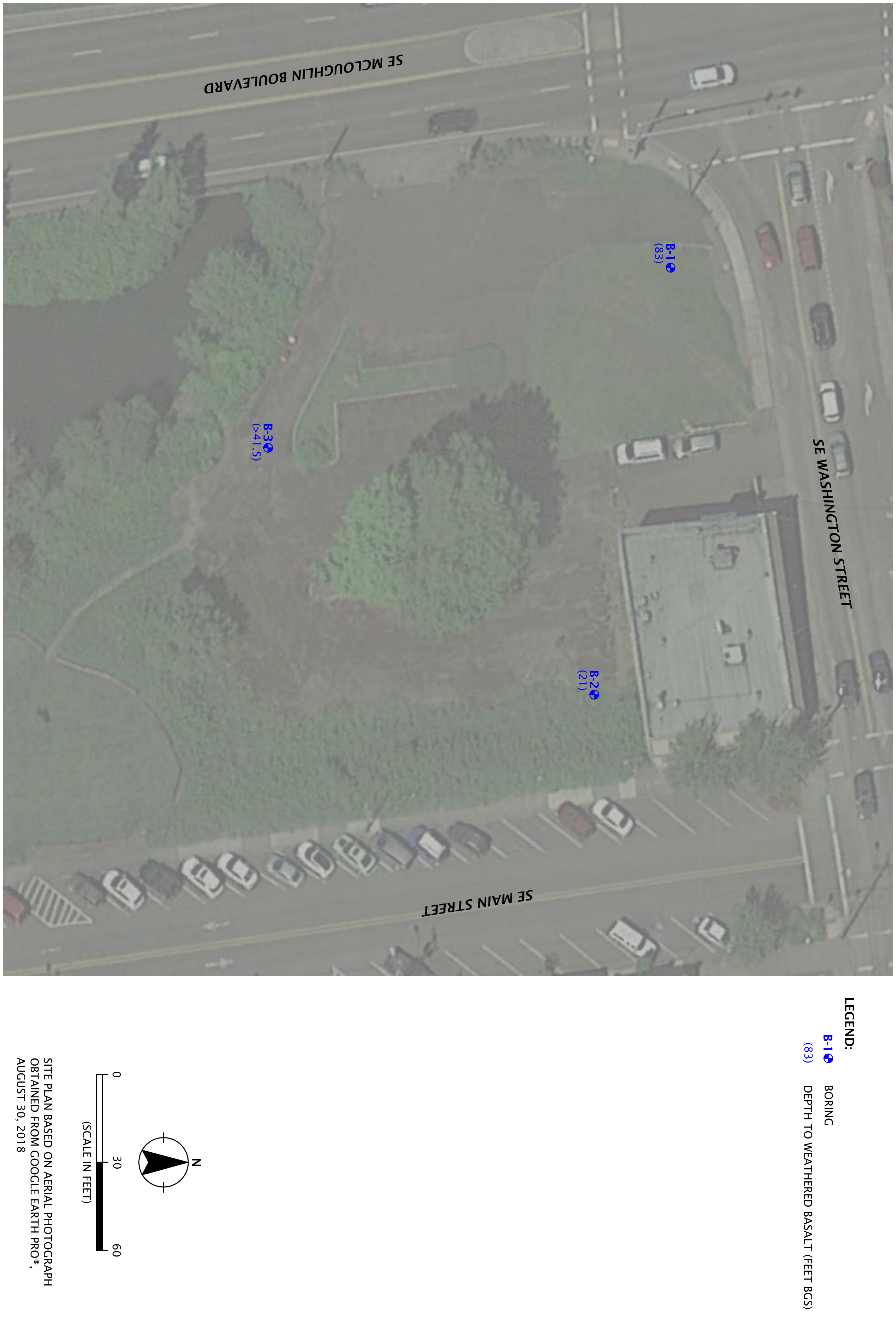
File Name: J:\M-R\Milwaukie\Milwaukie-7\Milwaukie-7-01\Figures\CAD\Milwaukie-7-01-SP01.dwg | Layout: FIGURE 2



LEGEND:

B-1 ● BORING
(83) DEPTH TO WEATHERED BASALT (FEET BGS)

SITE PLAN BASED ON AERIAL PHOTOGRAPH
OBTAINED FROM GOOGLE EARTH PRO®,
AUGUST 30, 2018

GEODESIGN INC 9450 SW Commerce Circle - Suite 300 Wilsonville OR 97070 503.968.8787 www.geodesigninc.com	MILWAUKIE-7-01	**SITE PLAN**	
	SEPTEMBER 2018	COHO POINT AT KELLOGG CREEK MILWAUKIE, OR	**FIGURE 2**

APPENDIX

APPENDIX

FIELD EXPLORATIONS

GENERAL

Subsurface conditions at the site were explored by drilling three borings (B-1 through B-3). The borings were drilled by Western States Soil Conservation of Hubbard, Oregon, on August 15, 2018 using a truck-mounted drill rig and mud rotary drilling methods. The exploration logs are presented in this appendix.

Elevations shown on the logs were determined based on an existing conditions survey dated January 27, 2016 prepared by Statewide Land Survey, Inc.

SOIL SAMPLING

A member of our geotechnical staff observed the explorations. We collected representative samples of the various soils encountered in the explorations for geotechnical laboratory testing. Soil samples were collected by conducting SPTs in general conformance with ASTM D1586. The sampler was driven with a 140-pound hammer free-falling 30 inches. The number of blows required to drive the sampler 1 foot, or as otherwise indicated, into the soil is shown adjacent to the sample symbols on the exploration logs. Disturbed soil samples were collected from the split barrel for subsequent classification and index testing. Higher quality, relatively undisturbed samples were collected using a standard Shelby tube in general accordance with ASTM D1587, the Standard Practice for Thin-walled Tube Sampling of Soils. Sampling methods and intervals are shown on the exploration logs.

The average efficiency of the automatic SPT hammer used by Western States Soil Conservation was 75.1 percent. The calibration testing results are presented at the end of this appendix.

SOIL CLASSIFICATION

The soil samples were classified in the field 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 soil 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

We visually examined soil samples collected from the explorations to confirm field classifications. We also performed to following laboratory testing to evaluate the engineering properties of the soil.

MOISTURE CONTENT

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

ATTERBERG LIMITS

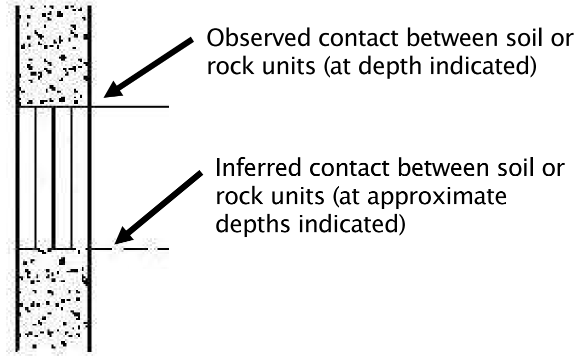
The plastic limit and liquid limit (Atterberg limits) of select soil samples were determined in accordance with ASTM D4318. The Atterberg limits and the plasticity index were completed to aid in the classification of the soil. The test results are presented in this appendix.

PARTICLE-SIZE TESTING

Particle-size testing was performed on select soil samples to determine the distribution of soil particle sizes. The testing consisted of percent fines determination (percent passing the U.S. Standard No. 200 sieve) analyses completed in general accordance with ASTM D1140. The test results are presented in this appendix.

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

Graphic Log of Soil and Rock Types



GEOTECHNICAL TESTING EXPLANATIONS

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

ENVIRONMENTAL TESTING EXPLANATIONS

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

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EXPLORATION KEY

TABLE A-1

RELATIVE DENSITY - COARSE-GRAINED SOIL

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

CONSISTENCY - FINE-GRAINED SOIL

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

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

MOISTURE CLASSIFICATION

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

ADDITIONAL CONSTITUENTS

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

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

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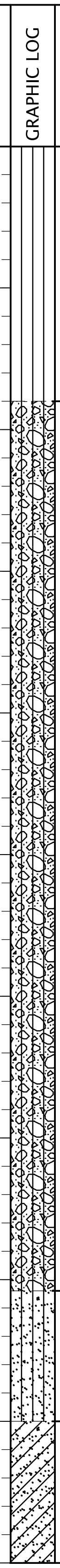
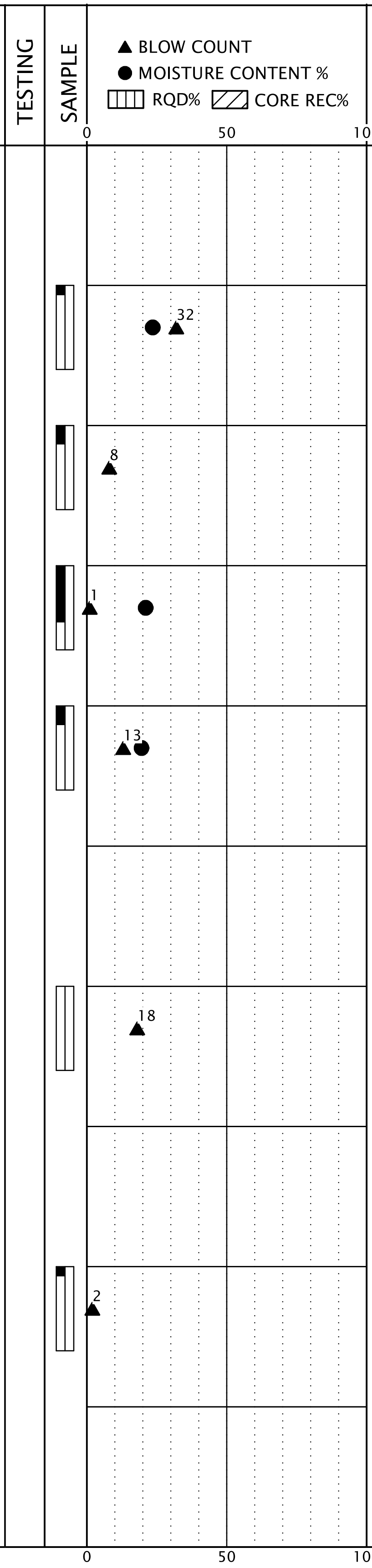
SOIL CLASSIFICATION SYSTEM

TABLE A-2

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	INSTALLATION AND COMMENTS
0.0	Hard, brown, gravelly SILT (ML), minor sand; moist - **FILL**.	38.0	
4.5	Loose, light brown, silty GRAVEL with sand and concrete debris (GM); moist to wet - **FILL**.	33.5 4.5	
6.5			Driller comment: drilling became smoother at 6.5 feet.
7.5	very loose at 7.5 feet		
10.0	medium dense, minor sand at 10.0 feet		Driller comment: alternating layers of silt from 10.0 to 20.0 feet.
15.0			No recovery at 15.0 feet.
20.0	dark gray, with sand, trace metal debris at 20.0 feet		
20.2	Very loose, gray, silty SAND (SM); wet, sand is fine.	17.8 20.2	
22.5	Very loose, light gray, clayey SAND (SC); wet, sand is fine.	15.5 22.5	





DRILLED BY: Western States Soil Conservation, Inc.

LOGGED BY: J. Guenther

COMPLETED: 08/15/18

BORING METHOD: mud rotary (see document text)

BORING BIT DIAMETER: 4 inches

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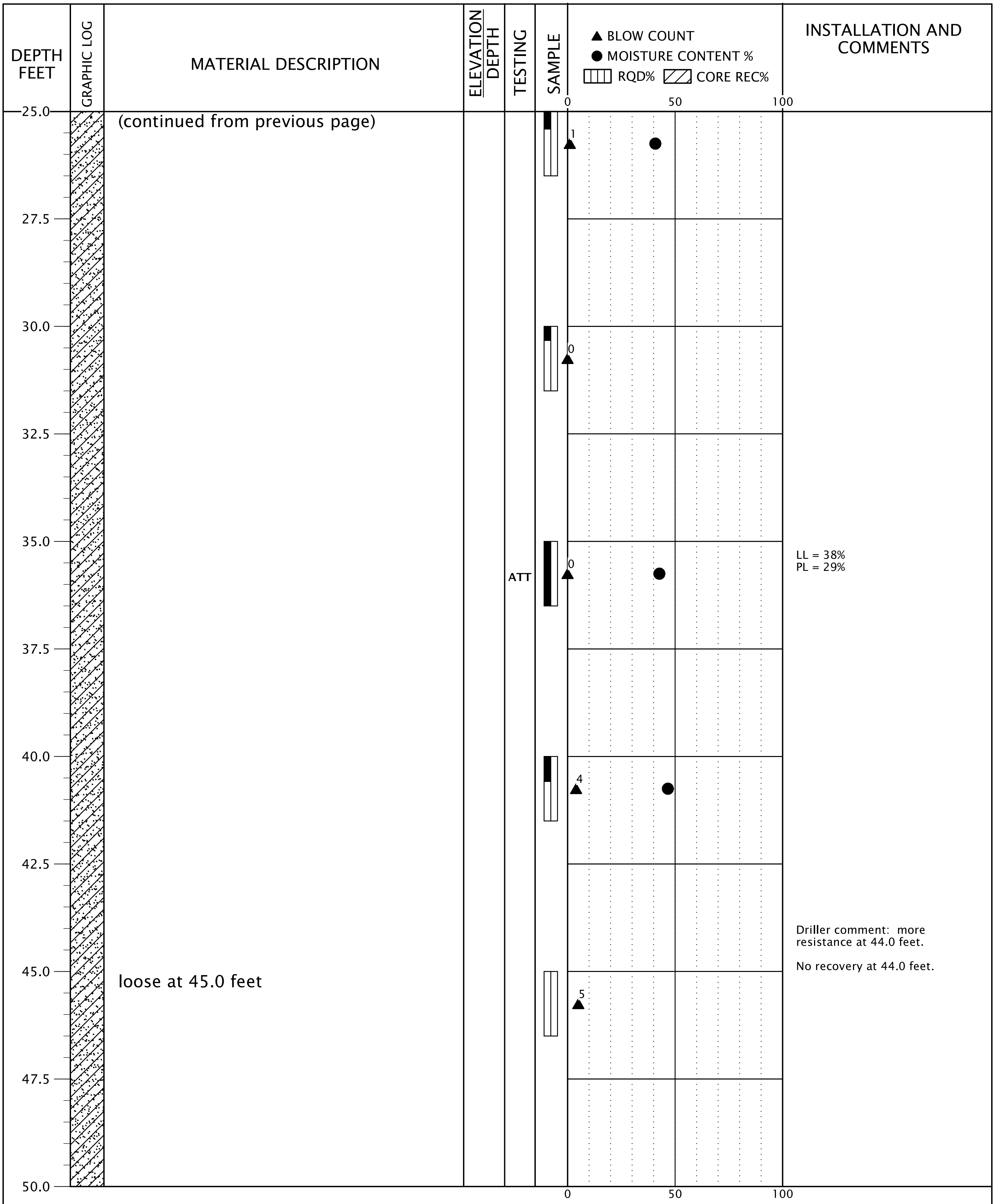
MILWAUKIE-7-01

SEPTEMBER 2018

BORING B-1

COHO POINT AT KELLOGG CREEK
MILWAUKIE, OR

FIGURE A-1



(continued from previous page)

loose at 45.0 feet

LL = 38%
PL = 29%

Driller comment: more resistance at 44.0 feet.

No recovery at 44.0 feet.

DRILLED BY: Western States Soil Conservation, Inc.

LOGGED BY: J. Guenther

COMPLETED: 08/15/18

BORING METHOD: mud rotary (see document text)

BORING BIT DIAMETER: 4 inches

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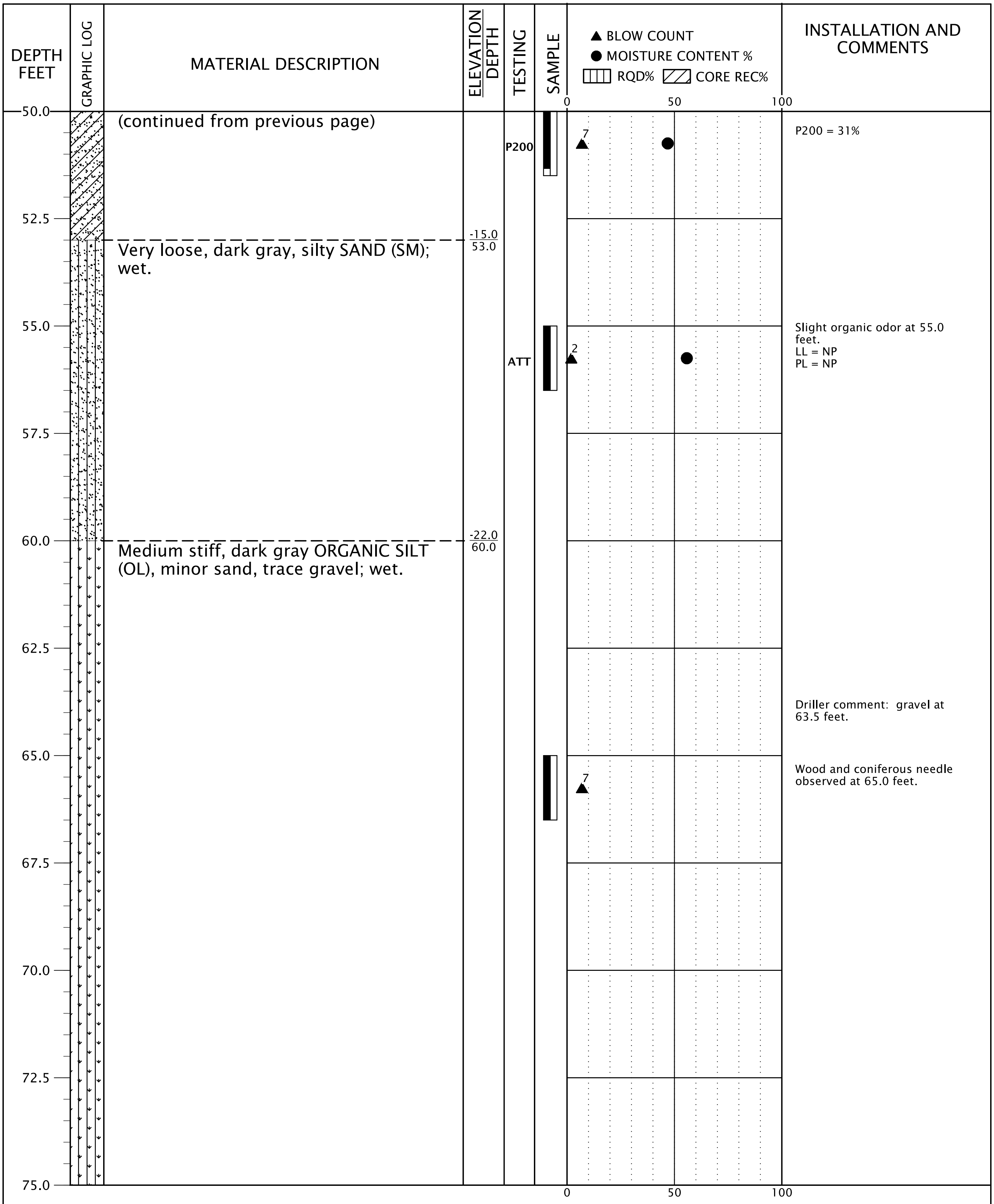
BORING B-1
(continued)

COHO POINT AT KELLOGG CREEK
MILWAUKIE, OR

FIGURE A-1



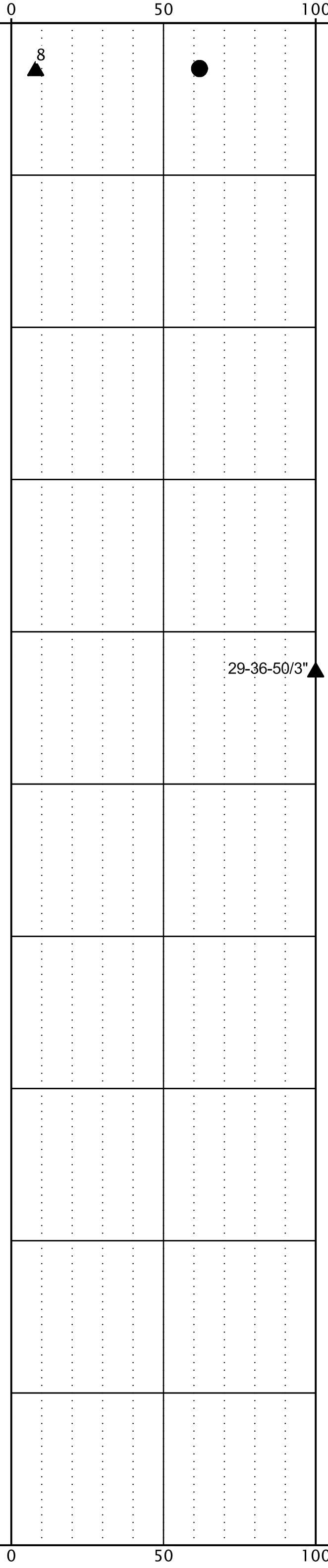
DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	BLOW COUNT	INSTALLATION AND COMMENTS
50.0	(continued from previous page)		P200	7	P200 = 31%
53.0	Very loose, dark gray, silty SAND (SM); wet.	-15.0 53.0			
55.0			ATT	2	Slight organic odor at 55.0 feet. LL = NP PL = NP
60.0	Medium stiff, dark gray ORGANIC SILT (OL), minor sand, trace gravel; wet.	-22.0 60.0			
63.5					Driller comment: gravel at 63.5 feet.
65.0				7	Wood and coniferous needle observed at 65.0 feet.

DRILLED BY: Western States Soil Conservation, Inc.
LOGGED BY: J. Guenther
COMPLETED: 08/15/18

BORING METHOD: mud rotary (see document text)
BORING BIT DIAMETER: 4 inches

BORING LOG MILWAUKIE-7-01-B1_3.GPJ GEODESIGN.GDT PRINT DATE: 9/22/18:KM:KT

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	▲ BLOW COUNT ● MOISTURE CONTENT % ▥ RQD% ▨ CORE REC%	INSTALLATION AND COMMENTS
75.0		medium stiff at 75.0 feet					
83.0		Very dense, dark gray GRAVEL with silt and sand (GP-GM); wet (weathered basalt).	-45.0 83.0				Driller comment: gravel at 83.0 feet.
86.3		Exploration terminated at a depth of 86.3 feet due to refusal. Hammer efficiency factor is 75.1 percent.	-48.3 86.3				



DEPTH FEET: 75.0, 77.5, 80.0, 82.5, 85.0, 87.5, 90.0, 92.5, 95.0, 97.5, 100.0

DRILLED BY: Western States Soil Conservation, Inc.

LOGGED BY: J. Guenther

COMPLETED: 08/15/18

BORING METHOD: mud rotary (see document text)

BORING BIT DIAMETER: 4 inches

BORING LOG MILWAUKIE-7-01-B1_3.GPJ GEODESIGN.GDT PRINT DATE: 9/22/18:KM:KT

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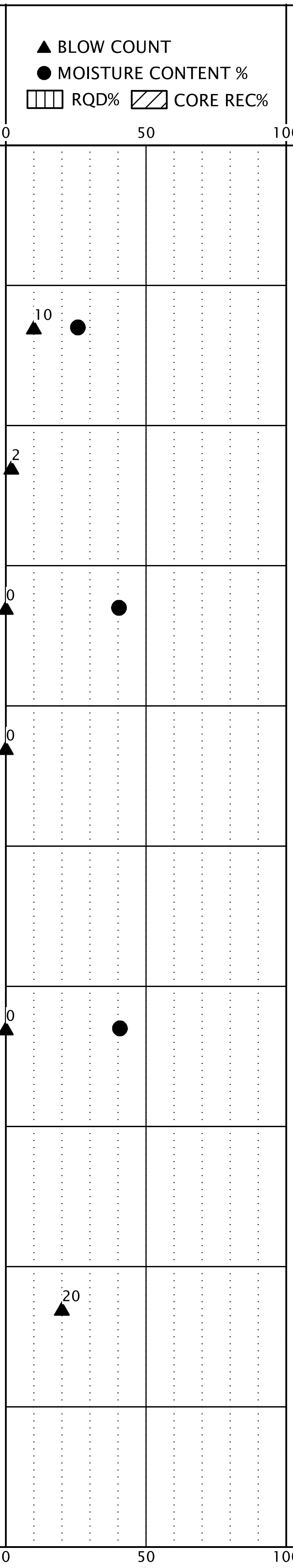
SEPTEMBER 2018

BORING B-1
(continued)

COHO POINT AT KELLOGG CREEK
MILWAUKIE, OR

FIGURE A-1

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	BLOW COUNT	INSTALLATION AND COMMENTS
0.0	ASPHALT CONCRETE (2.0 inches).	38.0 37.8 / 0.2			
	AGGREGATE BASE (3.0 inches).	37.6 / 0.4			
	Loose, brown, silty GRAVEL with sand (GM), trace organics; moist - **FILL**.				
2.5				10	
	Soft, brown, sandy SILT (ML); moist to wet, sand is fine.	33.5 / 4.5			
5.0				2	No recovery at 5.0 feet.
7.5	very soft; wet at 7.5 feet		**P200**	0	Sampler penetrated 1.5 feet under weight of drill rod at 7.5 feet. P200 = 59%
10.0				0	
12.5					
15.0				0	
17.5					
	Medium dense, brown-orange, silty SAND with gravel (SM); wet.	19.0 / 19.0			
20.0				20	
	Medium dense, gray, silty GRAVEL with sand (GM); wet (weathered basalt).	17.0 / 21.0			
22.5					
25.0					



DRILLED BY: Western States Soil Conservation, Inc.

LOGGED BY: J. Guenther

COMPLETED: 08/15/18

BORING METHOD: mud rotary (see document text)

BORING BIT DIAMETER: 4 inches

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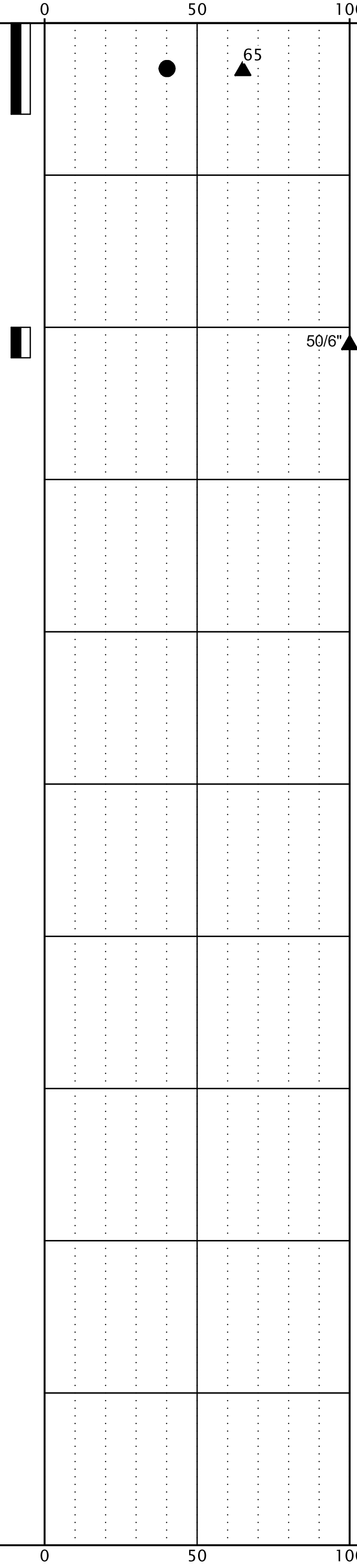
BORING B-2

COHO POINT AT KELLOGG CREEK
MILWAUKIE, OR

FIGURE A-2

BORING LOG MILWAUKIE-7-01-B1_3.GPJ GEODESIGN.GDT PRINT DATE: 9/22/18:KM:KT

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE	▲ BLOW COUNT ● MOISTURE CONTENT % ▥ RQD% ▨ CORE REC%	INSTALLATION AND COMMENTS
25.0		very dense, gray with orange and brown mottles at 25.0 feet					
27.5		dark gray-brown, minor gravel at 28.5 feet					
30.0		Exploration completed at a depth of 30.5 feet. Hammer efficiency factor is 75.1 percent.	7.5 30.5				
32.5							
35.0							
37.5							
40.0							
42.5							
45.0							
47.5							
50.0							



DRILLED BY: Western States Soil Conservation, Inc.

LOGGED BY: J. Guenther

COMPLETED: 08/15/18

BORING METHOD: mud rotary (see document text)

BORING BIT DIAMETER: 4 inches

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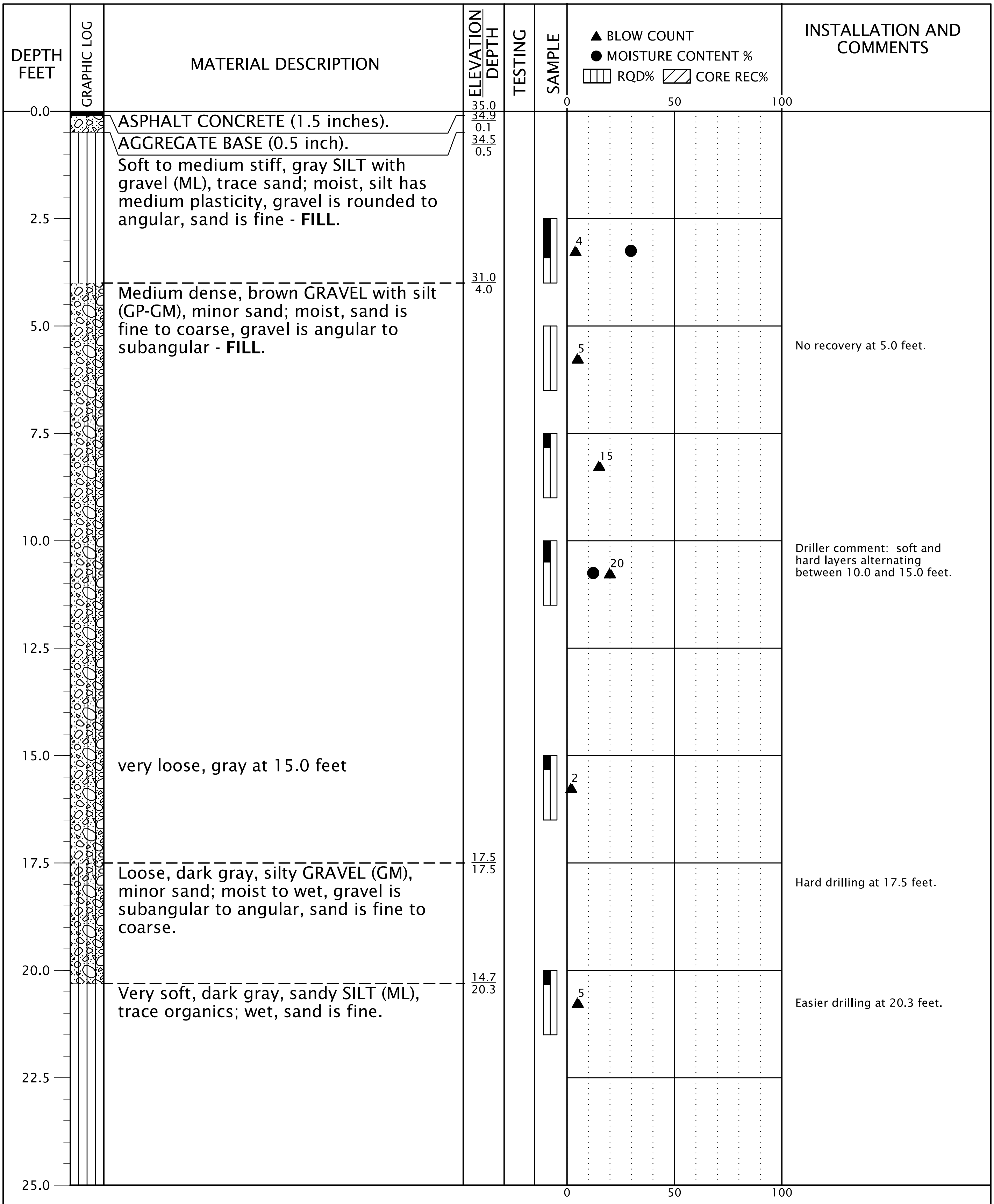
BORING B-2
(continued)

COHO POINT AT KELLOGG CREEK
MILWAUKIE, OR

FIGURE A-2



DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	BLOW COUNT	INSTALLATION AND COMMENTS
0.0	ASPHALT CONCRETE (1.5 inches).	35.0 34.9 0.1		
	AGGREGATE BASE (0.5 inch).	34.5 0.5		
	Soft to medium stiff, gray SILT with gravel (ML), trace sand; moist, silt has medium plasticity, gravel is rounded to angular, sand is fine - **FILL**.		4	
4.0	Medium dense, brown GRAVEL with silt (GP-GM), minor sand; moist, sand is fine to coarse, gravel is angular to subangular - **FILL**.	31.0 4.0	5	No recovery at 5.0 feet.
			15	
			20	Driller comment: soft and hard layers alternating between 10.0 and 15.0 feet.
15.0	very loose, gray at 15.0 feet		2	
17.5	Loose, dark gray, silty GRAVEL (GM), minor sand; moist to wet, gravel is subangular to angular, sand is fine to coarse.	17.5 17.5		Hard drilling at 17.5 feet.
20.3	Very soft, dark gray, sandy SILT (ML), trace organics; wet, sand is fine.	14.7 20.3	5	Easier drilling at 20.3 feet.

DRILLED BY: Western States Soil Conservation, Inc.   LOGGED BY: J. Guenther   COMPLETED: 08/15/18

BORING METHOD: mud rotary (see document text)   BORING BIT DIAMETER: 4 inches

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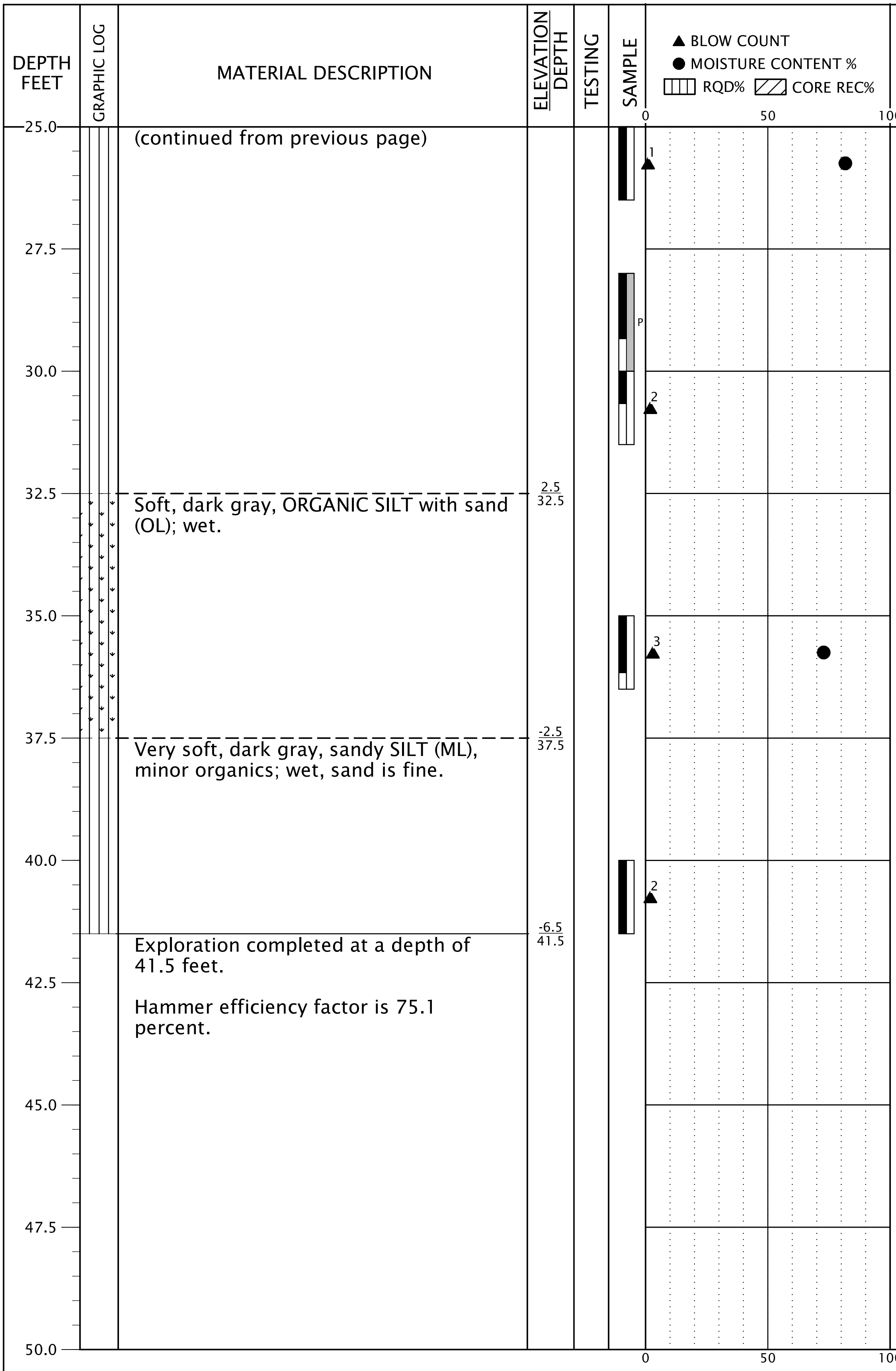
BORING B-3

COHO POINT AT KELLOGG CREEK
MILWAUKIE, OR

FIGURE A-3

BORING LOG MILWAUKIE-7-01-B1_3.GPJ GEODESIGN.GDT PRINT DATE: 9/22/18:KM:KT

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	INSTALLATION AND COMMENTS
25.0	(continued from previous page)		Sampler driven 1 foot by weight of hammer at 25.0 feet.
32.5	Soft, dark gray, ORGANIC SILT with sand (OL); wet.	2.5 32.5	
37.5	Very soft, dark gray, sandy SILT (ML), minor organics; wet, sand is fine.	-2.5 37.5	Sampler driven 6 inches by weight of hammer at 39.0 feet.
41.5	Exploration completed at a depth of 41.5 feet. Hammer efficiency factor is 75.1 percent.	-6.5 41.5	



DRILLED BY: Western States Soil Conservation, Inc.

LOGGED BY: J. Guenther

COMPLETED: 08/15/18

BORING METHOD: mud rotary (see document text)

BORING BIT DIAMETER: 4 inches

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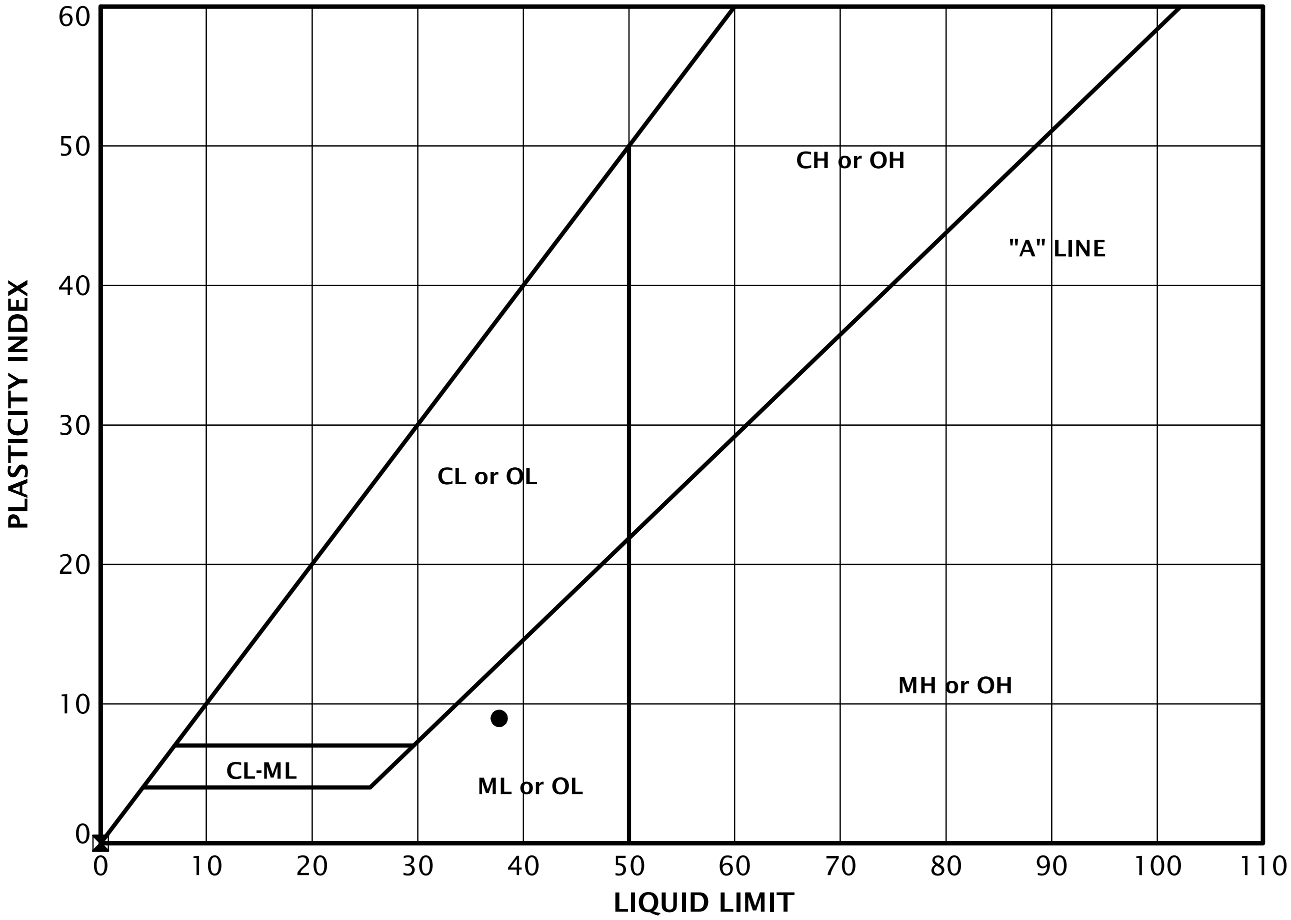
BORING B-3
(continued)

COHO POINT AT KELLOGG CREEK
MILWAUKIE, OR

FIGURE A-3

BORING LOG MILWAUKIE-7-01-B1_3.GPJ GEODESIGN.GDT PRINT DATE: 9/22/18:KM:KT



KEY	EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	MOISTURE CONTENT (PERCENT)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
●	B-1	35.0	43	38	29	9
☒	B-1	55.0	56	NP	NP	NP

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ATTERBERG LIMITS TEST RESULTS

COHO POINT AT KELLOGG CREEK
MILWAUKIE, OR

FIGURE A-4

ATTERBERG_LIMITS 7 MILWAUKIE-7-01-B1_3.GPJ GEODESIGN.GDT PRINT DATE: 9/10/18:KM

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		24							
B-1	7.5		21							
B-1	10.0		20							
B-1	25.0		41							
B-1	35.0		43					38	29	9
B-1	40.0		47							
B-1	50.0		47				31			
B-1	55.0		56					NP	NP	NP
B-1	75.0		62							
B-2	2.5		26							
B-2	7.5		40				59			
B-2	15.0		41							
B-2	25.0		40							
B-3	2.5		30							
B-3	10.0		12							
B-3	25.0		82							
B-3	35.0		73							

LAB SUMMARY MILWAUKIE-7-01-B1_3.GPJ GEODESIGN.GDT PRINT DATE: 9/10/18:KM

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MILWAUKIE-7-01

SEPTEMBER 2018

SUMMARY OF LABORATORY DATA

COHO POINT AT KELLOGG CREEK
MILWAUKIE, OR

FIGURE A-5

Summary of SPT Test Results

Project: WSSC-8-02, Test Date: 12/29/2017

EMX: Maximum Energy

ETR: Energy Transfer Ratio - Rated

Instr. Length ft	Start Depth ft	Final Depth ft	**N Value**	**N60 Value**	Average EMX ft-lb	Average ETR %
15.00	0.00	0.00	**0**	**0**	263	75.1
				Overall Average Values:	263	75.1
				Standard Deviation:	9	2.7
				Overall Maximum Value:	281	80.2
				Overall Minimum Value:	224	63.9

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