Preliminary Stormwater Drainage Report

City of Milwaukie Parks

Prepared for: City of Milwaukie
Prepared by: Cara Kniphuisen
Project Engineer: Jessica Zink

June 2023 | KPFF Project #2100556

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“I hereby certify that this Stormwater Drainage Report for the Milwaukie Parks has been prepared by me or under my supervision and meets minimum standards of the City of Milwaukie 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.”

Jessica Zink, PE

REGISTERED PROFESSIONAL ENGINEER
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DIGITALLY SIGNED 2023.06.07 17:38:12 -07'00'
OREGON
JAN. 10, 2017
JESSICA ZINK
EXPIRATION DATE: 06/30/24

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**Will be included in a future submittal of this report*

Introduction

The Scott, Balfour, and Bowman Brae Park renovations will take place on the existing park sites located in the City of Milwaukie. Scott Park is located on SE 21st Ave on a 0.58-acre lot adjacent to the Ledding Library. Renovations will take place within the existing lot, with the exception of sidewalk and utility connections on the library's property. Balfour Park is located on a 0.8-acre site off SE Balfour Street and is surrounded by residential properties on three sides. The improvements will take place within the park's property and along the frontage of SE Balfour Street. Bowman-Brae Park is on a 0.69-acre lot located off SE Bowman Street. SE Bowman Street borders the south edge of the property and residential properties border the other 3 sides. The site improvements will be within the park boundaries and along the frontage of SE Bowman Street. See Exhibit 1 – Vicinity Maps.

Existing Conditions

Scott Park

The site is at the dead-end of SE 21st Ave and just north of the existing library. The site slopes to the north-east and drains to Spring Creek located along the east side of the site. There are numerous trees across the site and an existing concrete pathway that meanders around them. There is also a small concrete amphitheater located on the library plot, along with gardens and benches. A small parking lot along SE 21st Ave serves both the library and the park. See Exhibit 2 – Scott Park: Existing Stormwater Basin Map.

A review of NRCS mapping suggests the site is primarily underlain by Woodburn silt loam. A geotechnical investigation was performed by GeoDesign, Inc, and their findings and recommendations are summarized in the report dated August 25, 2017 (see Appendix A). Based on their results, the site has little to no infiltration capacity, see Table 1A.

Table 1A: Scott Park Infiltration Test Results

Boring	Depth	Material	Observed Infiltration Rate (inches per hour)	Percent Fines
B-5	6.0	Sand with Silt	0.3	27

There are two primary drainage outfalls from the property. The first is overland sheet flow to Spring Creek that is located east of the property. This receives most of the drainage from the site. The second is the existing vegetated storm planter that is in the southern portion of the property. The planter was constructed with the library expansion project and receives roughly half of the roof drainage and a small portion of the parking lot drainage (see Appendix B – MLL Preliminary Stormwater Report). The planter outfalls to the City of Milwaukie storm sewer system. Both conveyance systems ultimately discharge to the same tributary of the Johnson Creek basin.

Balfour Park

This site is located on SE Balfour Street and is made up of two plots, with the southern edge of the property fronting the street. The other three sides are adjacent to residential properties. The site slopes south-west to the corner of the property and there is about 20 feet of drop across the site. There is no existing stormwater infrastructure in SE Balfour Street. The site is within the Johnson Creek tributary that eventually feeds the Willamette River. There is a small asphalt and gravel path existing on site, along with many trees and bushes along the perimeter and north. The frontage on SE Balfour Street has no sidewalk or other improvements. See Exhibit 4 – Balfour Park: Existing Stormwater Basin Map.

A review of NRCS mapping suggests the site is primarily underlain by Woodburn silt loam and Latourell loam. A geotechnical investigation was performed by NV5, and their findings and recommendations are summarized in the report dated June 1, 2022 (see Appendix A). The tests found there is adequate infiltration about 10-20 feet below the ground surface, see Table 1B.

Table 1B: Balfour Park Infiltration Test Results

Boring	Depth	Material	Observed Infiltration Rate (inches per hour)	Percent Fines
B-2	20	Clayey Gravel with Sand (GC)	4.4	17

Bowman Brae Park

This park is located at the corner of SE Bowman Street and SE Brae Street. It is bound by SE Bowman Street to the south, a private drive to the east, a residential property to the north, and a maintenance shop to the west. The current conditions consist of grass and a few trees and is generally flat. There is an existing 24-inch storm line located along the east side of the site and down SE Brae Street. The stormwater drains to an outfall in Kellogg Creek, just south of the site. See Exhibit 6 – Bowman Brae Park: Existing Stormwater Basin Map.

A review of NRCS mapping suggests the site is primarily underlain by Woodburn silt loam. A geotechnical investigation was performed by NV5, and their findings and recommendations are summarized in the report dated June 1, 2022 (see Appendix A). Based on their observations, the soils are not conducive to infiltration, and stormwater management designs will have to consider alternative strategies to provide water quality treatment and flow control prior to release to the city-owned system.

Stormwater Requirements

Onsite treatment, detention, and discharge at each site will be required for stormwater management. Storm detention facilities must provide storage for up to the 25-year storm event and have overflow conveyance for the 100-year storm event. The 2-, 5-, 10-, 25-, and 100-year storm events will all be evaluated using the Unit Hydrograph Method.

According to the City of Milwaukie Public Works Standards, all water quality facilities shall meet the design requirements of the City of Portland, Stormwater Management Manual (SWMM). The SWMM states that the water quality design storm is 1.61 inches of rainfall over 24 hours. Allowable release rates under the developed conditions will not exceed peak flow rates under existing conditions for the 2-, 5-, 10-, and 25-year modeled storm events and meet minimum 1-inch orifice sizing.

Proposed Storm Drainage

Scott Park

The existing and proposed conditions for this site have two drainage basins. The first basin makes up most of the site and flows northeast to Spring Creek, and the second is captured in the existing storm basin. The storm water design for this site has been configured to use this existing planter for the new impervious area. The details of this planter can be found in Appendix B – MLL Preliminary Stormwater Report. The Presumptive Approach Calculator (PAC) was used to estimate the size and composition of the existing planter, see Appendix C: Scott Park Existing Planter Calculation.

The proposed site will add 1,563 square feet of impervious area, or 4%. The additional area consists of the playground surface which will have underdrains to direct runoff to the existing storm planter. See Exhibit 4 for proposed drainage basins and impervious areas. The planter expansion area was sized using the PAC, see Appendix B: Scott Park Proposed Planter Calculation. The existing planter will need to be expanded by 70 square feet to be able to meet runoff and water quality standards. The expansion will have the same composition as the existing planter. The proposed conditions will also use the same overflow inlet that is located in the existing planter. The overflow drainage is piped to the stormwater system within SE 21st Ave.

Table 2A summarizes basin areas under existing conditions and Table 2B summarizes basin areas under the developed conditions. Table 2C summarizes the total impervious areas in both conditions.

Table 2A: Scott Park Existing Condition Basin Areas

Catchment/Area	Source	Area		CN	Storm Facility Type
		SF	AC		
Basin A	Pavement	4,900	0.11	98	Planter
	Roof	5,305	0.12	98	
	Landscaping	1,336	0.03	74	
	Total	11,541	0.26	95.2	
Basin X	Pavement	4,152	0.10	98	Not treated
	Landscaping	22,224	0.51	74	
	Total	26,376	0.61	77.8	
	Total Site Area	37,917	0.87		
	Total Percent Impervious	38%			

Table 2B: Scott Park Developed Condition Basin Areas

Catchment/Area	Source	Area		CN	Storm Facility Type
		SF	AC		
Basin A	Pavement	4,900	0.11	98	Planter
	Roof	5,305	0.12	98	
	Landscaping	1,336	0.03	74	
	Play Surface	1,563	0.04	98	
	Total	13,104	0.30	95.6	
Basin X	Pavement	4,152	0.10	98	Not treated
	Landscaping	16,924	0.39	74	
	Pervious Pavement	3,283	0.08	80	
	Dirt Path	454	0.01	87	
	Total	24,813	0.57	79.0	
Total Site Area		37,917	0.87		
Total Percent Impervious		42%			

Table 2C: Scott Park Total Areas

Description	Impervious Surface Area		Pervious Surface Area		Total Site Area		Total Percentage Impervious
	(sf)	(ac)	(sf)	(ac)	(sf)	(ac)	
Pre-Development	14,357	0.33	23,560	0.54	37,917	0.87	38%
Post-Development	15,920	0.37	21,997	0.50	37,917	0.87	42%

Balfour Park

The development of the site stormwater management system has been preliminarily configured to provide a combination of water quality and detention for most of the site. The proposed development will increase the impervious area from 1,807 square feet to 1,907 square feet.

There are three drainage basins for the proposed conditions (see Exhibit 5). The west portion of the site will continue to drain southwest off the site and will not be treated or detained, see “Basin X” in Exhibit 5. Basin A includes the proposed playground area, landscaping, and some sidewalk. Runoff from this area will sheet flow to a swale located along the proposed path. This drainage will then be conveyed to the lower portion of the stormwater facility via a culvert. The remaining area onsite will drain directly to the lower planter.

The stormwater planter will provide water quality treatment only. From the planter stormwater, will be directed to a new drywell. Based on the infiltration testing for this site, the runoff collected can be detained and infiltrated on-site. There is no overflow drain required for the facility.

Table 3A summarizes basin areas under existing conditions and Table 3B summarizes basin areas under the developed conditions. Table 3C summarizes the total impervious areas in both conditions.

Table 3A: Balfour Park Existing Condition Basin Areas

Catchment/Area	Source	Area		CN	Storm Facility Type
		SF	AC		
Basin Ex	Pavement	1,807	0.04	98	Not treated
	Landscaping	32,815	0.75	76	
	Total Site Area	34,522	0.79	77	
	Total Percent Impervious	5%			

Table 3B: Balfour Park Developed Condition Basin Areas

Catchment/Area	Source	Area		CN	Storm Facility Type
		SF	AC		
Basin A	Pervious Pavement	884	0.02	80	Planter
	Mulch	1,614	0.04	98	
	Landscaping	2,379	0.05	76	
	Total	4,877	0.11	84	
Basin B	Pervious Pavement	3,535	0.08	80	Planter
	Roof	289	0.01	98	
	Dirt	266	0.01	87	
	Landscaping	14,865	0.34	76	
	Total	18,955	0.44	77	
Basin X	Landscaping	10,790	0.25	76	Not treated
	Total	10,790	0.25	76	
	Total Site Area	34,622	0.79		
	Total Percent Impervious	19%			

Table 3C: Balfour Park Total Areas

Description	Impervious Surface Area		Pervious Surface Area		Total Site Area		Total Percentage Impervious
	(sf)	(ac)	(sf)	(ac)	(sf)	(ac)	
Pre-Development	1,807	0.04	32,815	0.75	34,622	0.79	5%
Post-Development	1,903	0.04	32,719	0.75	34,622	0.79	5%

Bowman Brae Park

The development of the site stormwater management system has been preliminarily configured to provide a combination water quality and detention for most of the site. The proposed development will increase the impervious area from 0 square feet to 1,1770 square feet, or about 6% more than under existing conditions.

There are two drainage basins for the proposed conditions (see Exhibit 7). Small portions of the east and west of the site, shown as “Basin X” in Exhibit 7, will continue to drain off the site and into the existing right-of-way. These areas will not be treated or detained. Basin A includes the proposed playground area, landscaping, pervious pavement sidewalk, and a small roof covering. The stormwater runoff for this basin will sheet flow to a proposed stormwater planter at the southern property line.

The proposed stormwater planter was sized using the PAC and is designed to detain flows up to the 25-year design storm. Based on the infiltration testing for this site, the runoff collected cannot be infiltrated on-site. Therefore, an overflow inlet will be constructed within the planter to carry drainage to the existing stormwater line in SE Brae Street.

Table 4A summarizes basin areas under existing conditions and Table 4B summarizes basin areas under the developed conditions. Table 4C summarizes the total impervious areas in both conditions.

Table 4A: Bowman and Brae Park Existing Condition Basin Areas

Catchment/Area	Source	Area		CN	Storm Facility Type
		SF	AC		
Basin A	Landscaping	27,895	0.64	75.5	Not Treated
Basin X	Landscaping	2,060	0.05	75.5	
	Total Site Area	29,955	0.69		
	Total Percent Impervious	0%			

Table 4B: Bowman and Brae Park Developed Condition Basin Areas

Catchment/Area	Source	Area		CN	Storm Facility Type
		SF	AC		
Basin A	Pervious Pavement	4,388	0.10	80	Planter
	Mulch	1,770	0.04	98	
	Roof	289	0.01	98	
	Landscaping	20,308	0.47	74	
	Total	26,755	0.61	76.8	
Basin X	Pervious Pavement	653	0.01	80	Not treated
	Landscaping	2,547	0.06	74	
	Total	3,200	0.07	75.2	
	Total Site Area	29,955	0.69		
	Total Percent Impervious	7%			

Table 4C: Bowman and Brae Park Total Areas

Description	Impervious Surface Area		Pervious Surface Area		Total Site Area		Total Percentage Impervious
	(sf)	(ac)	(sf)	(ac)	(sf)	(ac)	
Pre-Development	0	0.00	29,955	0.69	29,955	0.69	0%
Post-Development	2,059	0.05	27,896	0.64	29,955	0.69	7%

Detention

Scott Park

The existing and proposed landscape areas were given a CN of 74 based on the soil type at the site. Impervious areas such as the proposed playground areas, existing sidewalk, etc. shall have a CN of 98. The pervious pavement for the proposed sidewalks was given a CN of 80. A time of concentration of 5 minutes was used for both existing and developed conditions.

The storm planter for Ledding Library was modeled using the PAC for both existing and proposed conditions. The existing planter needs to be modified to add 70-sf more to the treatment area. This will be accomplished and not change the overflow inlet. See Appendix C for existing and proposed PAC calculations.

Balfour Park

A curve number (CN) value of 76 was used to represent the lawn and landscape areas for both existing and developed conditions. New impervious areas including roofs, playground areas, etc. shall have a CN value of

98. The pervious pavement for the sidewalks was given a CN of 80. A time of concentration of 5 minutes was used for both existing and developed conditions.

The stormwater facility was sized for water quality only. The flow control is provided with a new drywell. As the new proposed site area is less than 10,000 square feet, the simplified approach was used to size the proposed drywell. Table 3-8 in the BES SWMMM was used to size the diameter and depth of the drywell. According to the table, a 10-foot deep by 28-inch diameter concrete drywell can handle up to 2,500-sf of treatment area which is more than Basin A. The proposed design will therefore use a 10-ft tall by 28-inch diameter perforated concrete drywell section with the top approximately 3-feet below grade. It will include a cleanout inspection port in the center of the lid.

Groundwater was not encountered during the geotechnical test borings, which were extended to a depth of 20-feet below the ground surface. The SWMM requires a 5-feet minimum separation between the bottom of the infiltration facilities and the seasonal groundwater level. Due to the short stature of the proposed drywells and the depth of borings, separation is likely beyond the minimum.

Bowman Brae Park

The existing and proposed landscape areas were given a CN of 74 based on the soil type at the site. New impervious areas including roofs, playground areas, etc. shall have a CN of 98. The pervious pavement for the proposed sidewalks was given a CN of 80. A time of concentration of 5 minutes was used for both existing and developed conditions.

The Presumptive Approach Calculator (PAC) was used to size the proposed stormwater planter to fully detain runoff from the proposed impervious areas for the 10-year design storm. The planter is designed as configuration type D, which includes sub-grade rock storage within a lined facility, with an overflow inlet that discharges to the city’s stormwater system. The PAC report for the planter is included in Appendix C.

Flow control will be managed via an outlet pipe of the storm planter. The outlet pipe will contain an orifice to restrict the flow of stormwater leaving the site. In order to meet the city of Milwaukie code, the orifice will be 1-inch in diameter. With this size, the planter cannot meet the flow control. In order to meet flow control, the orifice would need to be reduced to 3/8-inch diameter, see Appendix C for alternative PAC calculations.

Table 5: Storm Planter & Catchments Table

Catchment /Area	Sub-Area	Impervious Area (Roof, Pavement or other)	Area (sf)	Storm Facility				
				ID	Type	Area (sf)	Total Storage Depth / 10-yr Event Water Depth (in)	Sizing Ratio*
A	A1 A2	Playground Roof	1,770 289	SP-1	Storm Planter (Lined)	216.5	8” / 9.8” **	10.5%
		Total:	**2,059**					

*Ratio of the facility area to the catchment area.
**Calculated using the CoP PAC

Water Quality

Scott Park

The storm planter for Ledding Library was modeled using the PAC for both existing and proposed conditions. The existing planter needs to be modified to add 70-sf more treatment area. This will be accomplished and not change the overflow inlet. See Appendix C for existing and proposed PAC calculations.

Balfour Park

The stormwater planter was sized for water quality only, using the PAC calculator. See Appendix C for the PAC report.

Bowman Brae Park

Treatment for stormwater at Bowman-Brae will be provided by a lined stormwater planter that was sized using the PACR. See Appendix C for the PAC report.

Conveyance

Pipe capacity will be calculated using Manning's "n" value of 0.013 for PVC pipe to convey the 10-year design storm. Conveyance calculations will be provided in the future with the final design and permit application.

Operations and Maintenance

The water quality and conveyance systems shall be operated and maintained in accordance with the standards of the city of Milwaukie. This section will be included with the final design permit application.

Appendix A

Geotechnical Reports

Appendix A
Scott

GEODESIGN INC

REPORT OF GEOTECHNICAL ENGINEERING SERVICES

Ledding Library of Milwaukie Renovation and Expansion
10660 SE 21st Avenue
Milwaukie, Oregon

For
City of Milwaukie
August 25, 2017

GeoDesign Project: CMilwaukie-2-01

Appendix A
Scott

GEODESIGN INC

August 25, 2017

PlanB Consultancy
696 McVey Avenue
Lake Oswego, OR 97034

Attention: Amy Winterowd

Report of Geotechnical Engineering Services
Ledding Library of Milwaukie Renovation and Expansion
10660 SE 21st Avenue
Milwaukie, Oregon
GeoDesign Project: CMilwaukie-2-01

GeoDesign, Inc. is pleased to submit THIS report of geotechnical engineering services for the proposed renovation and expansion of the Ledding Library of Milwaukie located at 10660 SE 21st Avenue in Milwaukie, Oregon. Our services for this project were conducted in accordance with our proposal dated March 24, 2017.

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

Sincerely,

GeoDesign, Inc.

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

cc: Jordan Henderson, PlanB Consultancy (via email only)

JTW:BAS:kt
Attachments
One copy submitted (via email only)
Document ID: CMilwaukie-2-01-082517-geor.docx
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EXECUTIVE SUMMARY

The following is a summary of our findings and recommendations for design and construction of the proposed library renovation and expansion. 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.

- Based on the assumed foundation loads, the proposed structures can be supported on shallow foundations bearing on granular pads constructed on firm native soil or soil compacted as structural fill as presented in the "Shallow Foundations" section.

- The on-site soils 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 soils currently is above optimum and drying will be required if used as structural fill.

- The on-site soils will provide inadequate support for construction equipment during periods 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.

- Based on our explorations, the near-surface soils at the site generally consist of fine-grained silt and clay. Based on our infiltration testing, the site has little to no infiltration capacity.

- The soils encountered during our subsurface explorations are not susceptible to liquefaction under design levels of ground shaking

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1.0 INTRODUCTION

GeoDesign, Inc. is pleased to submit this geotechnical engineering report for the proposed renovation and expansion of the Ledding Library of Milwaukie located at 10660 SE 21st Avenue in Milwaukie, Oregon. Figure 1 shows the site relative to existing topographic and physical features. Figure 2 shows the approximate site boundaries and our approximate exploration locations.

The exploration logs and laboratory testing results are presented in Appendix A. Our site-specific seismic evaluation is presented in Appendix B. Acronyms and abbreviations used herein are defined at the end of this document.

1.1 PROJECT UNDERSTANDING

The site encompasses Tax Lot 11E36BB011800, Parcel Number 00026803. The parcel is currently developed with the existing Ledding Library building and includes an AC-paved parking area and landscaped areas with walkways. We understand that plans are preliminary and currently being developed; however, they may consist of expansion of the library into the existing parking areas and/or landscaped areas. In addition, development plans will also include renovations to the existing library building.

Based on preliminary information provided by ABHT Structural Engineers, isolated column loads are anticipated to be between 150 and 200 kips and continuous wall loads are anticipated to be between 3 and 6 kips per linear foot. We anticipate maximum floor loads will be 100 psf. The building addition will be classified as a special occupancy structure and will require a site-specific seismic evaluation per the current SOSSC.

2.0 SCOPE OF SERVICES

The purpose of our geotechnical engineering services was to characterize site subsurface conditions and provide geotechnical engineering recommendations for use in design and construction of the proposed development. Our scope of work is presented as follows:

- Reviewed readily available published geologic data and our in-house files for existing information on subsurface conditions in the site vicinity.
- Explored subsurface conditions by drilling five borings to depths ranging between 8.0 and 16.5 feet BGS.
- Classified the materials encountered in the explorations, and maintained a detailed log of each exploration.
- Completed laboratory testing on disturbed soil samples collected from the explorations as follows:
  - Twenty-one moisture content determinations in general accordance with ASTM D 2216
  - Four particle-size determinations in general accordance with ASTM C 117 and ASTM D 1140
  - One Atterberg limits tests in general accordance with ASTM D 4318

- Provided recommendations for site preparation and grading, including clearing and grubbing, demolition, temporary and permanent slopes, fill placement criteria, suitability of on-site soil for fill, subgrade preparation, and recommendations for wet weather construction.
- Provided foundation support recommendations for the proposed building addition. Our recommendations include preferred foundation type, allowable bearing capacity, and lateral resistance parameters.
- 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.
- Provided pavement design recommendations for AC paving, including subbase, base course, and AC paving thickness.
- Provided recommendations for seismic design factors in accordance with the procedures outlined in the 2012 IBC and 2014 SOSSC.
- Conducted a site-specific seismic hazard evaluation as required for the public "occupied structure" in accordance with procedures in the 2014 SOSSC.
- Prepared this geotechnical engineering report that presents our findings, conclusions, and recommendations.

3.0 SITE CONDITIONS

3.1 SURFACE CONDITIONS

The approximately 1.8-acre property is currently developed with the existing Ledding Library building and includes an AC-paved parked area and landscaped areas with walkways. The building expansion will likely extend to the south of the existing structure into the landscape area or north into the existing parking lot. The site is relatively level with grade changes between approximately 42 and 47 feet MSL.

3.2 SUBSURFACE CONDITIONS

3.2.1 General

Our subsurface exploration program consisted of drilling five borings (B-1 through B-5) to depths ranging between 8.0 and 16.5 feet BGS. Borings B-1 through B-3 were drilled in the AC parking lot and B-4 and B-5 were drilled in existing landscape areas. Drilling refusal was encountered in all borings on the underlying gravel and silty gravel. We conducted infiltration testing in B-5 at a depth of 6.0 feet BGS. The approximate locations of the explorations are shown on Figure 2. A more detailed description of the exploration and laboratory testing programs, the exploration logs, and results of our laboratory testing are presented in Appendix A.

Subsurface conditions generally consist of silt and clay, over silty sand and sand with interbeds of silt, overlying medium dense to dense gravel. The following sections provide a more detailed description of the units encountered.

3.2.2 Pavement Section

Borings B-1 through B-3 were completed in the existing AC-paved parking lot. The AC varied from 3.0 to 6.0 inches thick and the aggregate base was observed to be 7.0 to 11.0 inches thick. Table 1 presents the thickness of the AC and aggregate base encountered at the boring locations.

Table 1. Existing Pavement Thicknesses

Boring	AC Thickness (inches)	Base Thickness (inches)
B-1	3.0	11.0
B-2	6.0	7.0
B-3	3.0	9.0

3.2.3 Silt and Clay

Below the AC and aggregate base and from the surface in B-4 we encountered brown to gray medium stiff to stiff silt and clay with trace to minor amounts of sand to depths ranging between 8.0 and 9.5 feet BGS in B-1 through B-4. A layer of very stiff silt was also observed between depths of 11.0 and 14.0 feet BGS in B-4. Laboratory analysis of the silt and clay indicates the moisture content ranged between 19 and 39 percent at the time of testing.

3.2.4 Sand

Loose to medium dense, brown silty sand and sand with silt was observed at depths ranging between 8.0 and 13.0 feet BGS below the silt and from the ground surface to a depth of 6.5 feet BGS in B-5. Interbedded layers of silt were observed throughout the silty sand and sand with silt. Laboratory analysis of the silty sand and sand with silt indicates the moisture content ranged from 14 to 39 percent at the time of testing.

3.2.5 Gravel

We encountered medium dense, brown to gray, silty gravel to gravel with sand starting at depths ranging between 6.5 and 14.0 feet BGS and extending to the maximum depth explored of 16.5 feet BGS. Laboratory testing indicates the moisture content ranged from 12 to 19 percent at the time of testing.

3.3 GROUNDWATER

Groundwater was observed in the three deeper borings during drilling. The depths to the observed groundwater are summarized in Table 2.

Table 2. Groundwater Measurements

Boring	Depth (feet BGS)
B-1	13.0
B-3	14.3
B-4	13.3

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. In addition, we expect the depth to groundwater may be associated with the water level of the pond and Spring Creek located along the east side of the property.

3.4 INFILTRATION TESTING

Infiltration testing was completed to assist in the evaluation of potential stormwater infiltration facilities for the project. We conducted one infiltration test in B-5 at a depth of 6.0 feet BGS. The infiltration test was performed using the encased falling head method using a 6-inch-inside diameter casing and approximately 12 inches of water head. Laboratory testing was performed to determine the percent fines content at the infiltration test depth. Table 3 summarizes the unfactored infiltration test results and the amount of fines present at the depth of the infiltration test.

Table 3. Infiltration Test Results

Boring	Depth (feet BGS)	Material	Observed Infiltration Rate[1] (inches per hour)	Percent Fines[2]
B-5	6.0	Sand with Silt	0.3	27

1. Infiltration rates are measured rates with no factor of safety.
2. Fines content: material passing the U.S. Standard No. 200 sieve

Given the infiltration test results, fine-grained soils present across the site, relatively shallow groundwater, and without additional testing, it is our opinion that the site has little to no infiltration capacity.

4.0 CONCLUSIONS

Based on the results of our subsurface explorations and engineering analyses, it is our opinion that the site can be developed as proposed. The primary geotechnical considerations for the project are summarized in the "Executive Summary." Our specific recommendations are provided in the following sections.

5.0 DESIGN

5.1 GENERAL

The following sections provide our design recommendations for the project. All site preparation and structural fill should be prepared as recommended in the "Construction" section.

5.2 SHALLOW FOUNDATIONS

5.2.1 General

Based on the results of our explorations and analysis, the proposed library addition can be supported by conventional spread footings resting on granular pads underlain by undisturbed native soil or structural fill overlying firm native soil. Foundations should not be established on undocumented fill, soft soil, or soil containing deleterious material. If present, this material should be removed and replaced with granular pads.

The granular pads should be a minimum of 4 inches thick, increasing to a minimum of 6 inches thick during the wet winter months, and extend 6 inches beyond the margins of the footings for every foot excavated below the base grade of the footing. The granular pads should consist of imported granular material, as defined in the "Structural Fill" section. The imported granular material should be compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D 1557, or, as determined by one of our geotechnical staff, until well-keyed. We recommend that a member of our geotechnical staff observe the prepared footing subgrade and the prepared granular pad.

5.2.2 Dimensions and Capacities

Continuous wall and isolated spread footings should be at least 18 and 24 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.

Footings bearing on subgrade prepared as recommended above should be sized based on an allowable bearing pressure of 2,500 psf. This is a net bearing pressure; the weight of the footing and overlying backfill can be ignored in calculating footing sizes. The recommended allowable bearing pressure applies to the total of dead plus long-term live loads and may be doubled for short-term loads such as those resulting from wind or seismic forces.

5.2.3 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 native soil and structural fill is 250 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. The passive resistance should be reduced to 120 pcf below groundwater.

For footings in contact with native soil, a coefficient of friction equal to 0.30 may be used when calculating resistance to sliding. For footings in contact with granular fill, a coefficient of friction equal to 0.40 may be used when calculating resistance to sliding.

5.2.4 Settlement

Based on the anticipated foundation loads, post-construction settlement of footings and floor slabs founded as recommended is anticipated to be less than 1 inch. Differential settlements between similarly loaded, newly constructed foundation elements should be approximately one-half of the total settlement. Differential settlement between new and existing foundation elements that are structurally tied together will likely be negligible and approaching the total settlement if structurally isolated.

5.2.5 Subgrade Observation

All footing and floor subgrades should be evaluated by a representative of GeoDesign to evaluate the bearing conditions. Observations should also confirm that all loose or soft material, organics, unsuitable fill, prior topsoil zones, and softened subgrades (if present) have been removed. Localized deepening of footing excavations may be required to penetrate deleterious material.

5.3 FLOOR SLABS

Satisfactory subgrade support for building floor slabs supporting up to 100 psf areal loading can be obtained on the existing undisturbed native silt and clay or on structural fill. To help reduce moisture transmission and slab shifting, we recommend a minimum 6-inch-thick layer of floor slab base rock be placed and compacted over a subgrade that has been prepared in conformance with the "Site Preparation" section. The floor slab base rock should meet the requirements in the "Materials" section and be compacted to at least 95 percent of ASTM D 1557.

While groundwater is unlikely to be encountered within the slab subgrade material, the native soil is fine grained and will tend to maintain a high moisture content. In areas where moisture-sensitive floor slab and flooring will be installed, the installation of a vapor barrier is warranted in order to reduce the potential for moisture transmission through and efflorescence growth on the slab and flooring. In addition, flooring manufacturers often require vapor barriers to protect flooring and flooring adhesives and they will warrant their product only if a vapor barrier is installed according to their recommendations.

Slabs should be reinforced according to their proposed use and per the structural engineer's recommendations. Load-bearing concrete slabs may be designed assuming a modulus of subgrade reaction, k, of 150 psi per inch.

5.4 RETAINING STRUCTURES

5.4.1 Assumptions

Retaining walls may be needed to address grade changes. Our retaining wall design recommendations are based on the following assumptions: (1) the walls consist of conventional, cantilevered retaining walls, (2) the walls are less than 8 feet in height, (3) the backfill is drained, and (4) the backfill has a slope flatter than 4H:1V. Re-evaluation of our recommendations will be required if the retaining wall design criteria for the project varies from these assumptions.

5.4.2 Wall Design Parameters

For unrestrained retaining walls, an active pressure of 35 pcf equivalent fluid pressure should be used for design. For embedded building walls, a superimposed seismic lateral force should be calculated based on a dynamic force of $7.0H^2$ pounds per lineal foot of wall, where H is the height of the wall in feet, and applied a distance of 0.6H from the base of the wall. Where retaining walls are restrained from rotation prior to being backfilled, a pressure of 55 pcf equivalent fluid pressure should be used for design.

If surcharges (e.g., retained slopes, building foundations, vehicles, steep slopes, terraced walls, etc.) are located within a horizontal distance from the back of a wall equal to twice the height of the wall, additional pressures will need to be accounted for in the wall design. Our office should be contacted for appropriate wall surcharges based on the actual magnitude and configuration of the applied loads.

The base of the wall footing excavations should extend a minimum of 18 inches below lowest adjacent grade. The footing excavations should then be lined with a minimum 4-inch-thick layer of compacted imported granular material, as described in the "Materials" section.

The wall footings should be designed in accordance with the guidelines provided in the appropriate portion of the "Shallow Foundations" section.

5.4.3 Wall Drainage and Backfill

The above design parameters have been provided assuming that back-of-wall drains will be installed to prevent buildup of hydrostatic pressures behind all walls. If a drainage system is not installed, our office should be contacted for revised design forces.

The backfill material placed behind the walls and extending a horizontal distance of ½H, where H is the height of the retaining wall, should consist of retaining wall select backfill placed and compacted in conformance with the "Structural Fill" section.

A minimum 6-inch-diameter, perforated collector pipe should be placed at the base of the walls. The pipe should be embedded in a minimum 2-foot-wide zone of angular drain rock that is wrapped in a drainage geotextile fabric and extends up the back of the wall to within 1 foot of the finished grade. The drain rock and drainage geotextile fabric should meet specifications provided in the "Materials" section. The perforated collector pipes should discharge at an appropriate location away from the base of the wall. The discharge pipe(s) should not be tied directly into stormwater drain systems, unless measures are taken to prevent backflow into the drainage system of the wall.

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

5.5 SEISMIC DESIGN CONSIDERATIONS

5.5.1 IBC Parameters

Based on our explorations, the following design parameters can be applied if the building is designed using the applicable provisions of the 2012 IBC and 2014 SOSSC. The parameters in Table 4 are appropriate for code-level seismic design obtained from USGS seismic design maps (USGS, 2014). We performed a site-specific seismic evaluation study, the results of this study are presented in Appendix B.

Table 4. IBC Seismic Design Parameters

Seismic Design Parameter	Short Period (T_s = 0.2 second)	1 Second Period (T_1 = 1.0 second)
MCE Spectral Acceleration, S	S_s = 0.984 g	S_1 = 0.421 g
Site Class	D	
Site Coefficient, F	F_a = 1.11	F_v = 1.58
Adjusted Spectral Acceleration, S_M	S_{MS} = 1.088 g	S_{M1} = 0.665 g
Design Spectral Response Acceleration Parameters, S_D	0.726 g	0.443 g

5.6 PAVEMENTS

5.6.1 Design Assumptions and Parameters

We anticipate some re-grading and re-paving may be needed to accommodate the building addition and site improvements. Pavements should be installed on undisturbed native subgrade, scarified and re-compacted soil, or new engineered fills described in the “Site Preparation” and “Structural Fill” sections.

Our pavement recommendations are based on the following assumptions:

- The top 12 inches of soil subgrade is compacted to at least 92 percent of its maximum dry density, as determined by ASTM D 1557, or until proof rolling with heavy equipment indicates that is it firm and unyielding.
- Resilient moduli of 3,700 psi and 20,000 psi were assumed for the subgrade and base rock, respectively.
- No traffic growth.
- A pavement design life of 20 years.
- Initial and terminal serviceability indices of 4.2 and 2.5, respectively.
- Reliability of 75 percent and standard deviation of 0.49.

We do not have specific information on the frequency of vehicles expected at the site. Consequently, we have provided pavement sections for automobile parking and heavy-duty areas with high automobile traffic and occasional heavy vehicles (i.e., garbage trucks, delivery trucks, semi-trucks, etc.). The breakdown of the type and frequency of the trucks used in our analysis are presented in Table 5. If any of these assumptions vary from project design values, our office should be contacted with the appropriate information so that the pavement designs can be revised.

Table 5. Truck Traffic Breakdown

FHWA Class Group	Description	Percent
5	2-axle, single unit	60
6	3-axle, single unit	30
7	4-axle, single unit	0
8	tractor/trailer 3- to 4-axle	10
9	tractor/trailer 3- to 4-axle	0
10	tractor/trailer 3- to 4-axle	0
11	5-axle, multi-trailer	0
12	6-axle, multi-trailer	0

Our pavement design recommendations assuming a maximum of five trucks per day are presented in Table 6.

Table 6. Recommended Standard Pavement Sections

Pavement Use	Trucks per Day[1]	ESALs	AC (inches)	Base Rock (inches)
Automobile Parking	0	10,000	2.5	8.0
Heavy Duty[1]	5	30,000	3.0	9.0

1. See Table 5 for the assumed breakdown of the trucks.

All thicknesses are intended to be the minimum acceptable. The design of the recommended pavement section is based on the assumption that construction will be completed during an extended period of dry weather. Wet weather construction could require an increased thickness of aggregate base. The AC and aggregate base should meet the requirements outlined in the "Materials" section.

Construction traffic should be limited to non-building, unpaved portions of the site or haul roads. Construction traffic should not be allowed on new pavements. If construction traffic is to be allowed on newly constructed road sections, an allowance for this additional traffic will need to be made in the design pavement section. The aggregate base does not account for construction traffic, and haul roads and staging areas should be used as described in the "Construction" section.

If any of these assumptions are incorrect, our office should be contacted with the appropriate information so that the pavement designs can be revised.

5.7 DRAINAGE

5.7.1 Surface Water Control

The ground surface around the structure should be sloped away from its foundations at a minimum 2 percent gradient for a distance of at least 5 feet. Downspouts should discharge into solid, smooth-walled drainage pipes that carry the collected water away from the building foundations. Trapped planter areas should not be created adjacent to buildings without providing means for positive drainage (e.g., swales or catch basins).

5.7.2 Foundation Drainage

We recommend installing footing drains around the perimeter of the proposed building addition. The footing drains should consist of a filter fabric-wrapped, drain rock-filled trench that extends at least 2 feet below the lowest adjacent grade (i.e., slab subgrade elevation). A minimum 4-inch-diameter, perforated pipe should be placed at the base to collect water that gathers in the drain rock. The drain rock and drainage geotextile fabric should meet the specifications outlined in the "Materials" section.

5.8 PERMANENT SLOPES

Permanent cut and fill slopes should not exceed 2H:1V. Slopes within stormwater facilities should not exceed 3H:1V. Access roads and pavements should be located at least 5 feet from the top of cut and fill slopes. The setback should be increased to 10 feet for buildings. The 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.

6.0 CONSTRUCTION

6.1 SITE PREPARATION

6.1.1 Demolition

Demolition should include removal of existing structures, pavements, and utilities that are present underneath areas to be improved. Demolished material should be transported off site for disposal or recycled and used on site if the material is acceptable for use as structural fill. Excavations remaining from site preparation activities should be backfilled with structural fill where below planned site grades. The base of excavations should be excavated to expose firm subgrade before filling. Utility lines abandoned under new structural elements should be completely removed and backfilled with structural fill in accordance with the recommendations provided in the "Structural Fill" section.

6.1.2 Stripping and Grubbing

The existing topsoil and vegetation should be stripped and removed from all proposed building and pavement areas and for a 5-foot margin around such areas. The actual stripping depth should be based on field observations at the time of construction. Stripped material should be transported off site for disposal or used in landscaped areas. Greater depths may be necessary to remove localized zones of organic material or deeper root zones.

Trees should also be removed from improved areas. Root balls should be grubbed out to the depth of the roots. Based on our experience, the grubbing depth required to remove tree root balls will be approximately 2.5 to 3 feet BGS and the grubbing depth to remove brush roots will be approximately 1 foot to 2 feet BGS. Depending on the methods used to remove the root balls, considerable disturbance and loosening of the subgrade could occur during site grubbing. We recommend that soil disturbed during grubbing operations be removed to expose firm subgrade. The resulting excavations should be backfilled with structural fill.

6.1.3 Subgrade Evaluation

Upon completion of stripping and subgrade stabilization, and prior to the placement of fill or pavement improvements, the exposed subgrade should be evaluated by proof rolling. The subgrade should be proof rolled with a fully loaded dump truck or similarly heavy, rubber-tired construction equipment to identify soft, loose, or unsuitable areas. A member of our geotechnical staff should observe the proof rolling to evaluate yielding of the ground surface. During wet weather, subgrade evaluation should be performed by probing with a foundation probe rather than proof rolling. Areas that appear soft or loose should be improved in accordance with subsequent sections of this report.

6.2 CONSTRUCTION CONSIDERATIONS

The fine-grained soils present on this site are easily disturbed. If not carefully executed, site preparation, utility trench work, and excavations can create extensive soft areas and significant repair costs can result. Earthwork planning, regardless of the time of year, should include considerations for minimizing subgrade disturbance.

If construction occurs during or extends into the wet season, or if the moisture content of the surficial soil is more than a couple percentage points above optimum, site stripping and cutting may need to be accomplished using track-mounted equipment. Likewise, the use of granular haul roads and staging areas will be necessary for support of construction traffic during the rainy season or when the moisture content of the surficial soil is more than a few percentage points above optimum. The base rock thickness for pavement areas is intended to support post-construction design traffic loads. This design base rock thickness will likely 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, staging and haul roads with increased thicknesses of base rock will be required.

The amount of staging and haul road areas, as well as the required thickness of granular material, will vary with the contractor's sequencing of a project and type/frequency of construction equipment. Based on our experience, between 12 and 18 inches of imported granular material is generally required in staging areas and between 18 and 24 inches in haul roads areas. A geotextile fabric is commonly placed below the imported granular material. The actual thickness will depend on the contractor's means and methods and should be the contractor's responsibility. The imported granular material, stabilization material, and geotextile are described in the "Materials" section.

6.3 EXCAVATION

6.3.1 Excavation and Shoring

Temporary excavation sidewalls should stand vertical to a depth of approximately 4 feet, provided groundwater seepage is not observed in the sidewalls. Open excavation techniques may be used to excavate trenches with depths between 4 and 8 feet, provided the walls of the excavation are cut at a slope of 1.5H:1V and groundwater seepage is not present. At this inclination, the slopes with loose sand may ravel and require some ongoing repair. Excavations should be flattened if excessive sloughing or raveling occurs. In lieu of large and open cuts, approved temporary shoring may be used for excavation support. A wide variety of shoring and dewatering systems are available. Consequently, we recommend that the contractor be responsible for selecting the appropriate shoring and dewatering systems.

If box shoring is used, it should be understood that box shoring is a safety feature used to protect workers and does not prevent caving. If the excavations are left open for extended periods of time, caving of the sidewalls may occur. The presence of caved material will limit the ability to properly backfill and compact the trenches. The contractor should be prepared to fill voids between the box shoring and the sidewalls of the trenches with sand or gravel before caving occurs.

If shoring is used, we recommend that the type and design of the shoring system be the responsibility of the contractor, who is in the best position to choose a system that fits the overall plan of operation. All excavations should be made in accordance with applicable OSHA and state regulations.

6.3.2 Trench Dewatering

Shallow excavations (less than 5 feet) will not likely encounter groundwater. However, perched groundwater may be encountered after prolonged wet periods. Dewatering systems are best designed by the contractor. It may be possible to remove groundwater encountered by pumping from a sump in the trenches. More intense use of pumps may be required at certain times of the year and where more intense seepage occurs. Removed water should be routed to a suitable discharge point.

If groundwater is present at the base of utility trench excavations, we recommend placing up to 12 inches of stabilization material at the base of the excavations. Trench stabilization material should meet the requirements provided in the “Structural Fill” section.

We note that these recommendations are for guidance only. The dewatering of excavations is the sole responsibility of the contractor, as the contractor is in the best position to select these systems based on their means and methods.

6.3.3 Safety

All excavations should be made in accordance with applicable OSHA requirements and regulations of the state, county, and local jurisdiction. While this report describes certain approaches to excavation and dewatering, the contract documents should specify that the contractor is responsible for selecting excavation and dewatering methods, monitoring the excavations for safety, and providing shoring (as required) to protect personnel and adjacent structural elements.

6.4 MATERIALS

6.4.1 Structural Fill

6.4.1.1 General

Fill should be placed on subgrade that has been prepared in conformance with the “Site Preparation” section. A variety of material may be used as structural fill at the site. However, all material used as structural fill should be free of organic matter or other unsuitable material and should meet the specifications provided in OSSC 00330 (Earthwork), OSSC 00400 (Drainage and

Sewers), and OSSC 02600 (Aggregates), depending on the application. A brief characterization of some of the acceptable materials and our recommendations for their use as structural fill is provided below.

6.4.1.2 On-Site Soil

The material at the site should be suitable for use as general structural fill provided it is properly moisture conditioned; free of debris, organic material, and particles over 4 inches in diameter; and meets the specifications provided in OSSC 00330.12 (Borrow Material).

Based on laboratory test results, the moisture content of the on-site soil will be significantly above the optimum required for compaction. Therefore, moisture conditioning (drying) will be required to use the on-site fine-grained soil for structural fill. Extended dry weather and sufficient area to dry the soil will be required to adequately condition the soil for use as structural fill. The on-site fine-grained soil should not be used as structural fill during the wet season. We note that during summer the near-surface (within 2 to 3 BGS) soils can become dry and require the addition of water to moisture condition for compaction.

When used as structural fill, the on-site fine-grained soils should be placed in lifts with a maximum uncompacted thickness of 8 inches and compacted to not less than 92 percent of the maximum dry density, as determined by ASTM D 1557.

6.4.1.3 Imported Granular Material

Imported granular material used as structural fill should be pit- or quarry-run rock, crushed rock, or crushed gravel and sand and should meet the specifications provided in OSSC 00330.14 (Selected Granular Backfill) or OSSC 00330.15 (Selected Stone Backfill). The imported granular material should also be angular, fairly well graded between coarse and fine material, have less than 5 percent by dry weight passing the U.S. Standard No. 200 sieve, and have at least two fractured faces.

Imported granular material should be placed in lifts with a maximum uncompacted thickness of 12 inches and compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D 1557. During the wet season or when wet subgrade conditions exists, the initial lift should be approximately 18 inches in uncompacted thickness and should be compacted by rolling with a smooth-drum roller without using vibratory action.

6.4.1.4 Stabilization Material

Stabilization material should consist of pit- or quarry-run rock, crushed rock, or crushed gravel and should meet the specifications provided in OSSC 00330.16 (Stone Embankment Material). In addition, the material should have a maximum particle size of 6 inches, less than 5 percent by dry weight passing the U.S. Standard No. 4 sieve, and at least two mechanically fractured faces. The material should be free of organic matter and other deleterious material. Stabilization material should be placed in lifts between 12 and 18 inches thick and compacted to a firm condition.

Where the stabilization material is used for staging or construction haul roads, a geotextile should be placed as a barrier between the soil subgrade and the imported granular material. The placement of the imported granular fill should be done in conformance with the specifications provided in OSSC 00331 (Subgrade Stabilization). The geotextile fabric should meet the specifications provided below for subgrade geotextiles. Geotextile is not required where stabilization material is used at the base of utility trenches.

6.4.1.5 Trench Backfill

Trench backfill placed beneath, adjacent to, and for at least 12 inches above utility lines (i.e., the pipe zone) should consist of well-graded granular material with a maximum particle size of 1½ inches and less than 7 percent by dry weight passing the U.S. Standard No. 200 sieve and should meet the specifications provided in OSSC 00405.13 (Pipe Zone Material). The pipe zone backfill should be compacted to at least 90 percent of the maximum dry density, as determined by ASTM D 1557, or as required by the pipe manufacturer or local building department.

Within roadway alignments, the remainder of the trench backfill up to the subgrade elevation should consist of well-graded granular material with a maximum particle size of 2½ inches and less than 7 percent by dry weight passing the U.S. Standard No. 200 sieve and should meet the specifications provided in OSSC 00405.14 (Trench Backfill; Class B, C, or D). This material should be compacted to at least 92 percent of the maximum dry density, as determined by ASTM D 1557, or as required by the pipe manufacturer or local building department. The upper 3 feet of the trench backfill should be compacted to at least 95 percent of the maximum dry density, as determined by ASTM D 1557.

Outside of structural improvement areas (e.g., roadway alignments or building pads) trench backfill placed above the pipe zone may consist of general fill material that is free of organics and material over 6 inches in diameter and meets the specifications provided in OSSC 00405.14 (Trench Backfill; Class A, B, C, or D). This general trench backfill should be compacted to at least 90 percent of the maximum dry density, as determined by ASTM D 1557, or as required by the pipe manufacturer or local building department.

6.4.1.6 Floor Slab Aggregate Base

Imported granular material used as base rock for building floor slabs should consist of ¾- or 1½-inch-minus material (depending on the application) and meet the requirements in OSSC 00641 (Aggregate Subbase, Base, and Shoulders). In addition, the aggregate should have less than 5 percent by dry weight passing the U.S. Standard No. 200 sieve. The aggregate base should be compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D 1557.

6.4.1.7 Pavement Aggregate Base

Imported granular material used as base rock for building floor slabs should consist of ¾- or 1½-inch-minus material (depending on the application) and meet the requirements in OSSC 00641 (Aggregate Subbase, Base, and Shoulders). In addition, the aggregate should have less than 5 percent by dry weight passing the U.S. Standard No. 200 sieve. The aggregate base should be compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D 1557.

6.4.1.8 Retaining Wall Select Backfill

Backfill material placed behind retaining walls and extending a horizontal distance of ½H, where H is the height of the retaining wall, should consist of select granular material that meets the requirements provided in OSSC 00510.12 (Granular Wall Backfill). We recommend the select granular wall backfill be separated from general fill, native soil, and/or topsoil using a geotextile fabric that meets the specifications provided below for drainage geotextiles.

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

6.4.1.9 Drain Rock Material

Drain rock should consist of angular, granular material that meets the specifications provided in OSSC 00430.11 (Granular Drain Backfill Material) and the aggregate should have at least two fractured faces. The drain rock should be wrapped in a drainage geotextile that meets the specifications provided below for drainage geotextiles.

6.4.1.10 Retaining Wall Leveling Pad

Imported granular material placed at the base of retaining wall footings should consist of select granular material that meets the specifications provided in OSSC 00510.13 (Granular Structure Backfill). The granular material should meet either the 1”-0 or ¾”-0 aggregate size listed in OSSC Table 02630-1 – Grading Requirements for Dense-Graded Aggregate and have at least two mechanically fractured faces. The leveling pad material should be placed in a 6- to 12-inch lift and compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D 1557.

6.4.2 AC

6.4.2.1 ACP

The AC should be Level 2, ½-inch, dense ACP according to OSSC 00744 (Asphalt Concrete Pavement) and compacted to 91 percent of the theoretical maximum density of the mix, as determined by AASHTO T 209. The minimum and maximum lift thickness is 2.0 and 3.0 inches, respectively, for ½-inch ACP. Lift thicknesses desired outside these limits should be discussed with the design team prior to design or construction. Asphalt binder should be performance graded and conform to PG 64-22 or better.

6.4.2.2 Cold Weather Paving Considerations

In general, AC paving is not recommended during cold weather (temperatures less than 40 degrees Fahrenheit). Compacting under these conditions can result in low compaction and premature pavement distress.

Each AC mix design has a recommended compaction temperature range that is specific for the particular AC binder used. In colder temperatures, it is more difficult to maintain the temperature of the AC mix as it can lose heat while stored in the delivery truck, as it is placed, and in the time between placement and compaction. In Oregon, the AC surface temperature during paving should be at least 40 degrees Fahrenheit for lift thickness greater than 2.5 inches and at least 50 degrees Fahrenheit for lift thickness between 2.0 and 2.5 inches.

If paving activities must take place during cold-weather construction as defined above, the project team should be consulted and a site meeting should be held to discuss ways to lessen low compaction risks.

6.4.3 Geotextile Fabric

6.4.3.1 Subgrade Geotextile

The subgrade geotextile should meet the specifications provided in OSSC Table 02320-4 - Geotextile Property Values for Subgrade Geotextile (Separation). The geotextile should be installed in conformance with OSSC 00350 (Geosynthetic Installation). A minimum initial aggregate base lift of 6 inches is required over geotextiles. All drainage aggregate and stabilization material should be underlain by a subgrade geotextile. Geotextile is not required where stabilization material is used at the base of utility trenches.

6.4.3.2 Drainage Geotextile

Drainage geotextile should meet the specifications provided in OSSC Table 02320-1 - Geotextile Property Values for Drainage Geotextile. The geotextile should be installed in conformance with OSSC 00350 (Geosynthetic Installation). A minimum initial aggregate base lift of 6 inches is required over geotextiles.

6.5 EROSION CONTROL

The site soil is susceptible to erosion; therefore, erosion control measures should be carefully planned and in place before construction begins. 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.

7.0 OBSERVATION OF CONSTRUCTION

Satisfactory foundation and earthwork performance depends to a large degree on quality of construction. 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. Subsurface conditions observed during construction should be compared with those encountered during the subsurface exploration. Recognition of changed conditions often requires experience; therefore, qualified personnel should visit the site with sufficient frequency to detect if subsurface conditions change significantly from those anticipated.

We recommend that GeoDesign be retained to observe earthwork activities, including stripping, proof rolling of the subgrade and repair of soft areas, footing subgrade preparation, performing laboratory compaction and field moisture-density tests, observing final proof rolling of the pavement subgrade and base rock, and asphalt placement and compaction.

8.0 LIMITATIONS

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

Exploration observations 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 preliminary 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 for the buildings, and walls, the conclusions and recommendations presented may not be applicable. If design changes are made, we request that we be retained to review our conclusions and recommendations and to provide a written modification or verification.

The scope 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 generally accepted practices in this area at the time the report was prepared. No warranty, 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 T. 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: 6/30/18

REFERENCES

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Appendix A
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FIGURES

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

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CMILWAUKIE-2-01	**VICINITY MAP**
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FIGURE 1

Printed By: aday | Print Date: 7/26/2017 9:19:39 AM
File Name: J:\A-D\CMilwaukie\CMilwaukie-2\CMilwaukie-2-01\Figures\CAD\CMilwaukie-2-01-VM01.dwg | Layout: FIGURE 1

Appendix A
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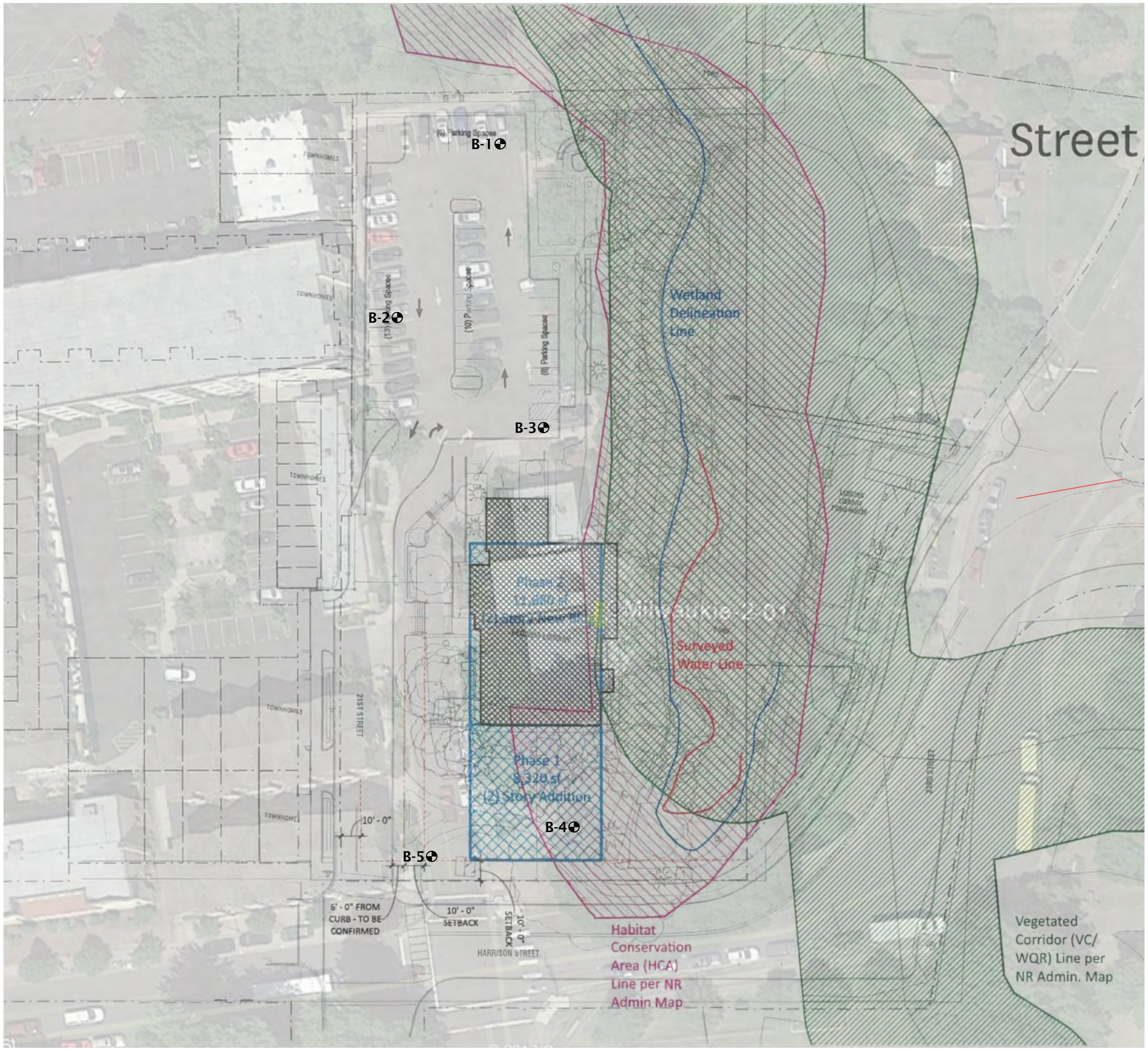
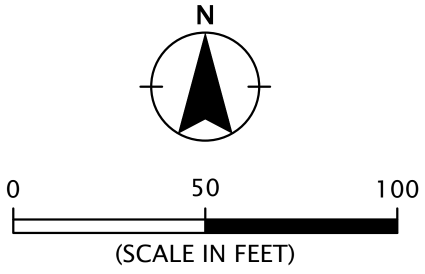


LEGEND:

B-1● BORING



SITE PLAN BASED ON IMAGE OBTAINED FROM
PLAN B CONSULTANCY

FIGURE 2

SITE PLAN

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File Name: J:\A-D\CMilwaukie\CMilwaukie-2\CMilwaukie-2-01\Figures\CAD\CMilwaukie-2-01-SP01.dwg | Layout: FIGURE 2

APPENDIX A

APPENDIX A

FIELD EXPLORATIONS

GENERAL

We explored the site by drilling five borings (B-1 through B-5) to depths ranging between 8.0 and 16.5 feet BGS. Drilling services were provided by Dan J. Fischer Excavating Inc. of Forest Grove, Oregon, using a trailer-mounted drill rig with solid-stem auger drilling methods. The exploration logs are presented in this appendix.

Approximate locations of our explorations are shown on Figure 2. The exploration locations were determined by pacing from existing site features and should be accurate implied by the methods used.

SOIL SAMPLING

A member of our geology 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 D 1586. 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. Sampling methods and intervals are shown on the exploration logs.

We understand that calibration of the SPT hammer used by Dan J. Fischer Excavating, Inc. has not been completed. The SPT blows completed by Dan J. Fischer Excavating, Inc. were conducted using two wraps around a cathead.

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

CLASSIFICATION

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

ATTERBERG LIMITS

The plastic limit and liquid limit (Atterberg limits) of a selected soil sample were determined in accordance with ASTM D 4318. 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.

MOISTURE CONTENT

We tested the natural moisture content of selected soil samples in general accordance with ASTM D 2216. The natural moisture content is a ratio of the weight of the water to soil in a test sample and is expressed as a percentage. The test results are presented in this appendix.

PARTICLE-SIZE ANALYSES

Particle-size analyses were completed on selected soil samples in general accordance with ASTM C 117 and ASTM D 1140. The test results are presented in this appendix.

Appendix A
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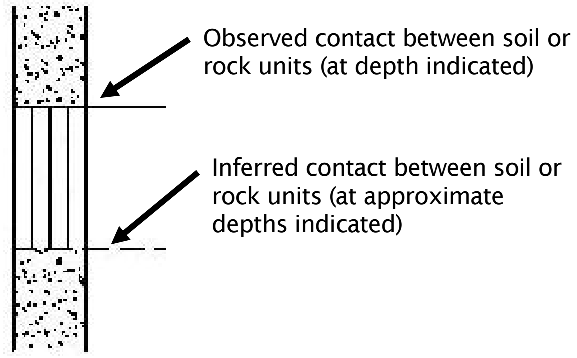
SYMBOL	SAMPLING DESCRIPTION
◧	Location of sample obtained in general accordance with ASTM D 1586 Standard Penetration Test with recovery
▮	Location of sample obtained using thin-wall Shelby tube or Geoprobe® sampler in general accordance with ASTM D 1587 with recovery
▤	Location of sample obtained using 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



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

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

GEOTECHNICAL TESTING EXPLANATIONS

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

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

Appendix A
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RELATIVE DENSITY - COARSE-GRAINED SOILS

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 SOILS

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 SOILS (more than 50% retained on No. 200 sieve)	GRAVEL (more than 50% of coarse fraction retained on No. 4 sieve)	CLEAN GRAVELS (< 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
		GRAVELS 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 SANDS (<5% fines)	SW or SP	SAND
		SANDS WITH FINES (≥ 5% and ≤ 12% fines)	SW-SM or SP-SM	SAND with silt
			SW-SC or SP-SC	SAND with clay
		SANDS WITH FINES (> 12% fines)	SM	silty SAND
			SC	clayey SAND
			SC-SM	silty, clayey SAND
FINE-GRAINED SOILS (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 SOILS			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 Soils	Coarse-Grained Soils		Fine-Grained Soils	Coarse-Grained Soils
< 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

Appendix A
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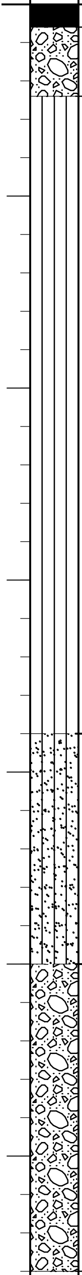
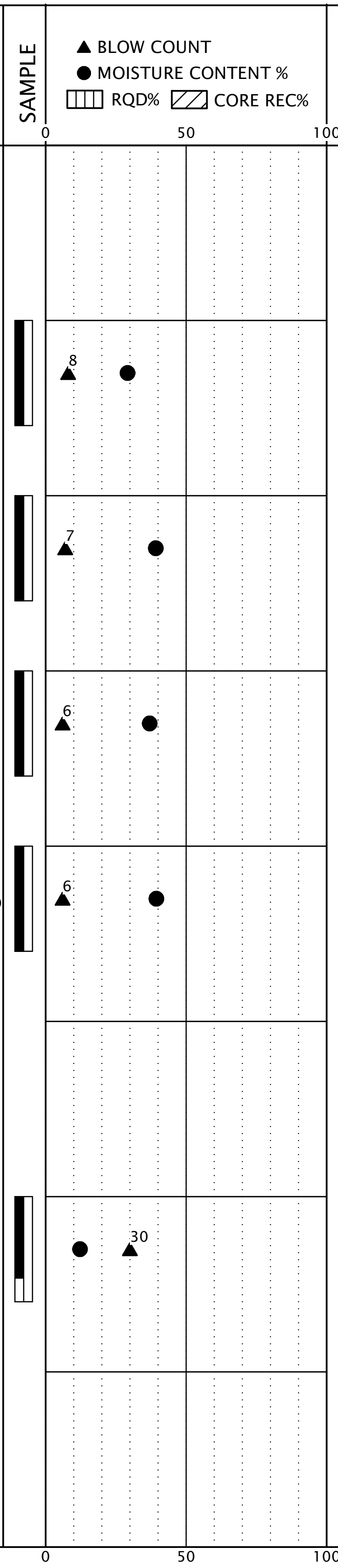
DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	INSTALLATION AND COMMENTS
0.0	ASPHALT CONCRETE (3.0 inches).			
	AGGREGATE BASE (11.0 inches).	0.3		
	Medium stiff, brown SILT (ML), trace sand; moist, sand is fine.	1.2		
2.5			PP	PP = 1.5 tsf
5.0			PP	PP = 1.75 tsf
7.5	minor sand at 7.5 feet		PP	PP = 1.0 tsf
	Loose, brown, silty SAND (SM); moist, fine.	9.5		
10.0			P200	P200 = 53%
12.5	Medium dense, brown GRAVEL with sand (GP), trace silt; moist, sand is medium. wet at 13.0 feet	12.5		Hard at 12.5 feet.
15.0	dense at 15.0 feet			Hard at 15.0 feet.
	Exploration terminated at a depth of 16.5 feet due to refusal. Hammer efficiency factor is unknown. SPT completed using two wraps and a cathead.	16.5		Surface elevation was not measured at the time of exploration.
17.5				
20.0				





◁ 13.0 feet, during drilling

DRILLED BY: Dan J. Fischer Excavating, Inc.
LOGGED BY: CR
COMPLETED: 07/12/17

BORING METHOD: solid-stem auger (see document text)
BORING BIT DIAMETER: 4 inches

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BORING B-1

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FIGURE A-1

Appendix A
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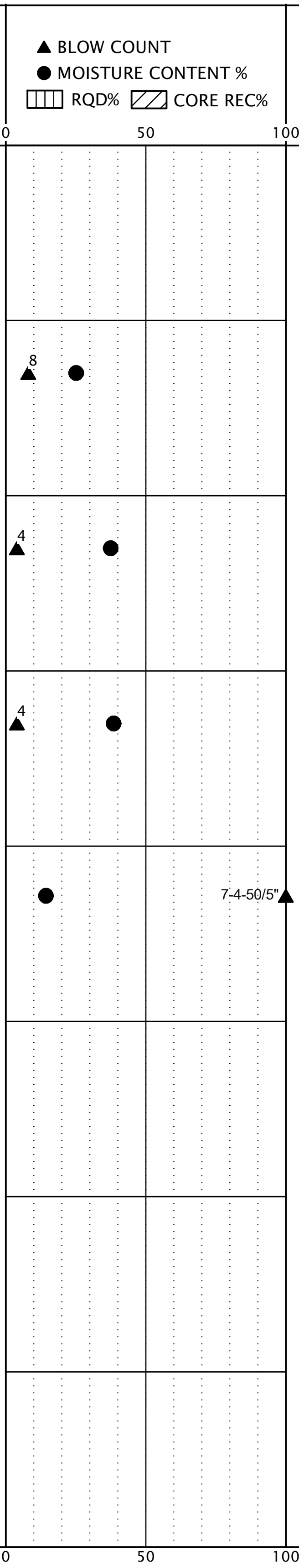
DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	INSTALLATION AND COMMENTS
0.0	ASPHALT CONCRETE (6.0 inches).	0.5			
	AGGREGATE BASE (7.0 inches).	1.1			
	Medium stiff, brown SILT (ML), trace sand; moist, sand is fine.				
2.5			PP	▮	PP = 1.25 tsf
5.0			PP	▮	PP = 1.0 tsf
7.5		8.5	PP	▮	PP = 0.75 tsf
	Loose, brown, silty SAND (SM); moist, fine.	10.0			
10.0	Very dense, gray-brown, silty GRAVEL with sand (GM); moist to wet, sand is fine to medium.	11.5		▮	
	Exploration terminated at a depth of 11.5 feet due to refusal. Hammer efficiency factor is unknown. SPT completed using two wraps and a cathead.				Surface elevation was not measured at the time of exploration.
12.5					
15.0					
17.5					
20.0					



DRILLED BY: Dan J. Fischer Excavating, Inc.
LOGGED BY: CR
COMPLETED: 07/12/17

BORING METHOD: solid-stem auger (see document text)
BORING BIT DIAMETER: 4 inches

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FIGURE A-2

BORING LOG CMILWAUKIE-2-01-B1_5.GPJ GEODESIGN.GDT PRINT DATE: 8/25/17:RC:KT

Appendix A
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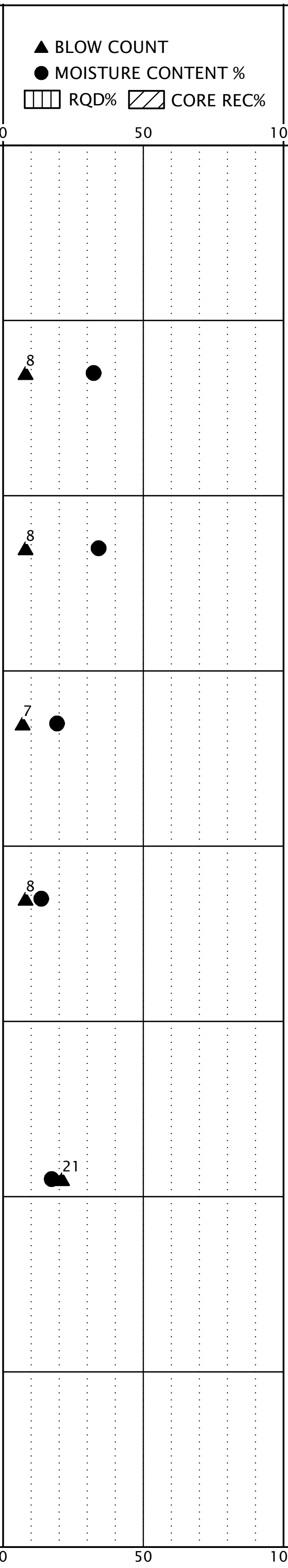
DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	BLOW COUNT	INSTALLATION AND COMMENTS
0.0	ASPHALT CONCRETE (3.0 inches).	0.3			
	AGGREGATE BASE (~9.0 inches).	1.0			
	Medium stiff, brown SILT (ML), trace sand; moist, sand is fine.				
2.5			PP	8	PP = 1.0 tsf
5.0			PP	8	PP = 1.5 tsf
7.5				7	
	Loose, brown, silty SAND (SM); moist, fine.	8.0			
	Loose, brown SAND (SP); moist, medium, several stratified layers of SILT (1 to 3 inches thick).	9.5			
10.0			P200	8	P200 = 16%
12.5					
	Medium dense, brown, silty GRAVEL with sand (GM); wet, sand is medium.	13.0			Hard drilling at 13.0 feet.
15.0				21	
	Exploration terminated at a depth of 15.5 feet due to drilling refusal. Hammer efficiency factor is unknown. SPT completed using two wraps and a cathead.	15.5			Surface elevation was not measured at the time of exploration.
17.5					
20.0					

▽ 14.3 feet, during drilling



DRILLED BY: Dan J. Fischer Excavating, Inc.
LOGGED BY: CR
COMPLETED: 07/12/17

BORING METHOD: solid-stem auger (see document text)
BORING BIT DIAMETER: 4 inches

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BORING B-3

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FIGURE A-3

BORING LOG CMILWAUKIE-2-01-B1_5.GPJ GEODESIGN.GDT PRINT DATE: 8/25/17:RC:KT

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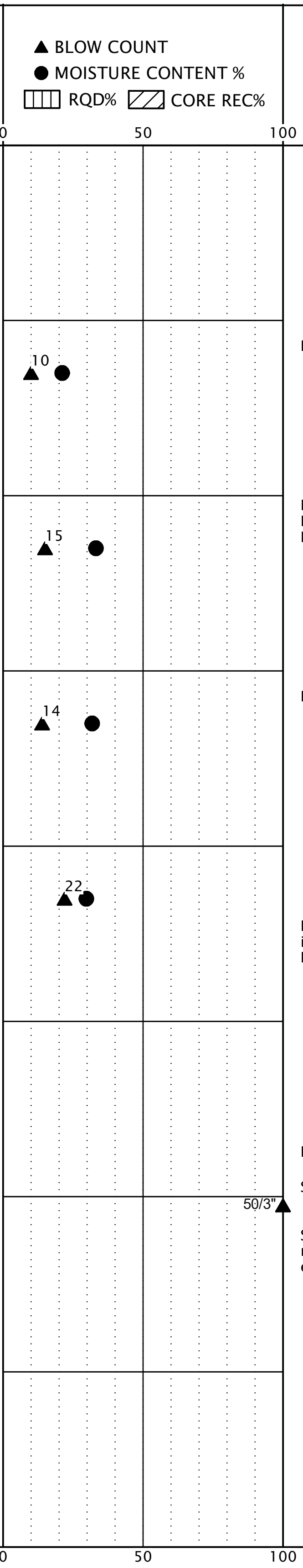
DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	BLOW COUNT	INSTALLATION AND COMMENTS
0.0	Stiff, brown and gray SILT (ML), trace sand; moist, sand is fine.				
2.5			PP	10	PP = >3.0 tsf
	Stiff, brown and gray CLAY (CH), trace sand; moist, sand is fine. gray with orange mottles at 5.5 feet	4.5			
5.0			ATT PP	15	PP = 1.75 tsf LL = 57% PL = 24%
7.5			PP	14	PP = 1.0 tsf
	Medium dense, brown SAND with silt (SP-SM); wet, fine.	8.5			
10.0			PP	22	Blow counts possibly influenced by gravel. PP = 0.5 tsf
	Very stiff, gray SILT (ML); wet.	11.0			
12.5					⊻ 13.3 feet, during drilling
	Very dense, brown, silty GRAVEL with sand (GM); wet, sand is fine to medium.	14.0			Hard at 14.0 feet.
15.0				50/3"	SPT hammer bouncing.
	Exploration terminated at a depth of 15.3 feet due to drilling refusal. Hammer efficiency factor is unknown. SPT completed using two wraps and a cathead.	15.3			Surface elevation was not measured at the time of exploration.
17.5					
20.0					



DRILLED BY: Dan J. Fischer Excavating, Inc.
LOGGED BY: CR
COMPLETED: 07/12/17

BORING METHOD: solid-stem auger (see document text)
BORING BIT DIAMETER: 4 inches

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BORING B-4

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FIGURE A-4

Appendix A
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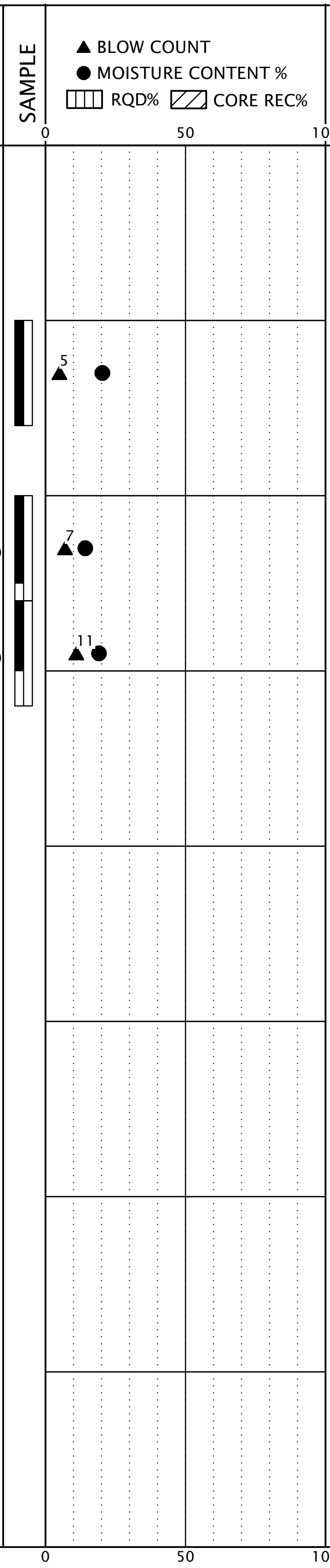
DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	INSTALLATION AND COMMENTS
0.0	Loose, brown, silty SAND (SM); moist, fine (3-inch-thick root zone).			
5.0				P200 = 24%
	Loose, brown SAND with silt (SP-SM); moist.	5.8	P200	Infiltration test: 0.3 inch per hour at 6.0 feet.
	Medium dense, brown GRAVEL with sand and silt (GP-GM); moist, sand is fine to medium.	6.5	P200	Hard at 6.5 feet. P200 = 27%
7.5	Exploration terminated at a depth of 8.0 feet due to refusal. Hammer efficiency factor is unknown. SPT completed using two wraps and a cathead.	8.0		Surface elevation was not measured at the time of exploration.



DRILLED BY: Dan J. Fischer Excavating, Inc.

LOGGED BY: CR

COMPLETED: 07/12/17

BORING METHOD: solid-stem auger (see document text)

BORING BIT DIAMETER: 4 inches

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BORING B-5

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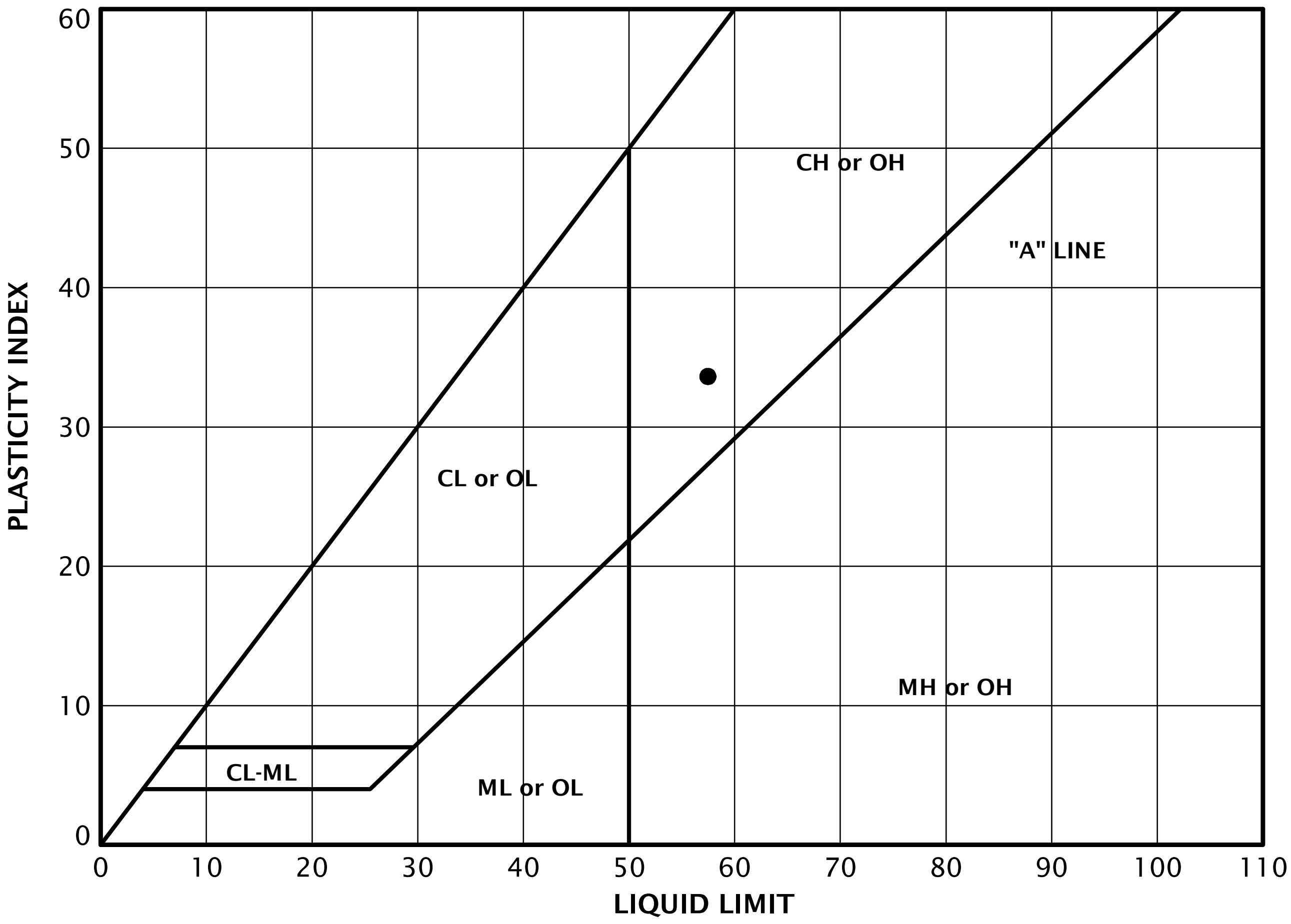
FIGURE A-5

BORING LOG CMILWAUKIE-2-01-B1_5.GPJ GEODESIGN.GDT PRINT DATE: 8/25/17:RC:KT



KEY	EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	MOISTURE CONTENT (PERCENT)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
●	B-4	5.0	43	57	24	33

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ATTERBERG LIMITS TEST RESULTS

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FIGURE A-6

Appendix A
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SAMPLE INFORMATION			MOISTURE CONTENT (PERCENT)	DRY DENSITY (PCF)	SIEVE			ATTERBERG LIMITS		
EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	ELEVATION (FEET)			GRAVEL (PERCENT)	SAND (PERCENT)	P200 (PERCENT)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
B-1	2.5		29							
B-1	5.0		39							
B-1	7.5		37							
B-1	10.0		39				53			
B-1	15.0		12							
B-2	2.5		25							
B-2	5.0		37							
B-2	7.5		39							
B-2	10.0		14							
B-3	2.5		32							
B-3	5.0		34							
B-3	7.5		19							
B-3	10.0		14				16			
B-3	14.0		17							
B-4	2.5		21							
B-4	5.0		33					57	24	33
B-4	7.5		32							
B-4	10.0		30							
B-5	2.5		20							
B-5	5.0		14				24			
B-5	6.5		19				27			

LAB SUMMARY CMILWAUKIE-2-01-B1_5.GPJ GEODESIGN.GDT PRINT DATE: 8/4/17:KT

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SUMMARY OF LABORATORY DATA

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FIGURE A-7

APPENDIX B

APPENDIX B

SITE-SPECIFIC SEISMIC HAZARD EVALUATION

INTRODUCTION

The information in this appendix summarizes the results of a site-specific seismic hazard evaluation for the proposed improvements at Ledding Library in Milwaukie, Oregon. This seismic hazard evaluation was performed to meet the requirement of the 2014 SOSSC.

SITE CONDITIONS

REGIONAL GEOLOGY

The site is located within the Portland Basin, which is separated from by the Tualatin Basin by the Tualatin Mountains (Portland Hills) to the west. Geologic mapping by Ma et al. (2012) and Beeson et al. (1989) shows the near-surface geology mapped as catastrophic Missoula flood deposits (channel facies). The Missoula flood deposits generally consists of a varying mix of unconsolidated deposits of sand, silt, and gravel sediment, which were deposited in major flood events. Since being deposited, the deposits have been modified by recent alluvium (Beeson et al., 1989). The Missoula flood deposits are underlain by undifferentiated sediments, which are commonly fine-grained sediments that overlay basalt bedrock in the site vicinity. The thickness is highly variably and ranges from less than 15 feet to greater than 200 feet (Beeson et al., 1989). The undifferentiated sediments are underlain by Eocene (54 million to 33 million years old) Basalt of Waverly Heights, a sequence of subaerial basaltic lava flows and associated undifferentiated sedimentary rocks (Beeson et al., 1989).

SUBSURFACE CONDITIONS

A detailed description of site subsurface conditions is presented in the main report.

SEISMIC SETTING

Earthquake Source Zones

Three scenario earthquakes were considered for this study consistent with the local seismic setting. Two of the possible earthquake sources are associated with the CSZ, and the third event is a shallow local crustal earthquake that could occur in the North American plate. The three earthquake scenarios are discussed below.

Regional Events

The CSZ is the region where the Juan de Fuca Plate is being subducted beneath the North American Plate. This subduction is occurring in the coastal region between Vancouver Island and northern California. Evidence has accumulated suggesting that this subduction zone has generated eight great earthquakes in the last 4,000 years, with the most recent event occurring approximately 300 years ago (Weaver and Shedlock, 1991). The fault trace is mapped approximately 50 to 120 km off the Oregon Coast.

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

1. An interface event earthquake on the seismogenic part of the interface between the Juan de Fuca Plate and the North American Plate on the CSZ. This source is reportedly capable of generating earthquakes with a moment magnitude of between 8.5 and 9.0.
2. A deep intraplate earthquake on the seismogenic part of the subducting Juan de Fuca Plate. These events typically occur at depths of between 30 and 60 km. This source is capable of generating an event with a moment magnitude of up to 7.5.

Local Events

A significant earthquake could occur on a local fault near the site within the design life of the facility. Such an event would cause ground shaking at the site that could be more intense than the CSZ events, though the duration would be shorter. Figure B-1 shows the locations of faults with potential Quaternary movement within a 20-mile radius of the site (USGS, 2014a; PNSN, 2014). Figure B-2 shows the interpreted locations of seismic events that occurred between 1833 and 2014 (USGS, 2014b). The most significant faults in the site vicinity are the Oatfield fault and Portland Hills fault. Table B-1 presents the closest mapped distance and mapped length of these faults.

Table B-1. Closest Crustal Faults

Source	Closest Mapped Distance[1] (km)	Mapped Length[1] (km)
Oatfield fault	1.0	24
Portland Hills fault	2.3	49

1. Reported by USGS (USGS, 2014a)

Oatfield Fault

The northwest-striking Oatfield fault forms northeast-facing escarpments in volcanic rocks of the Miocene CRBG in the Tualatin Mountains and northern Willamette Valley. The fault may be part of the Portland Hills-Clackamas River structural zone. The Oatfield fault is primarily mapped as a very high-angle, reverse fault with apparent down-to-the-southwest displacement, but a few kilometer-long reach of the fault with down-to-the-northeast displacement is mapped in the vicinity of the Willamette River. This apparent change in displacement direction along strike may reflect a discontinuity in the fault trace or could reflect the right-lateral, strike-slip displacement that characterizes other parts of the Portland Hills-Clackamas River structural zone. The fault has also been modeled as a 70-degree, east-dipping reverse fault. Reverse displacement with a right-lateral, strike-slip component is consistent with the tectonic setting, mapped geologic relations, and microseismicity in the area. Fault scarps on surficial deposits have not been described, but exposures in a light rail tunnel showing offset of approximately 1 M_a Boring Lava across the fault indicate Quaternary displacement (Personius, 2002a).

Portland Hills Fault

The northwest-striking Portland Hills fault forms the prominent linear northeastern margin of the Tualatin Mountains (Portland Hills) and the southwestern margin of the Portland Basin; this basin may be a right-lateral, pull-apart basin in the forearc of the CSZ or a piggyback synclinal basin formed between antiformal uplifts of the Portland fold belt. The fault is part of the Portland Hills-Clackamas River structural zone, which controlled the deposition of Miocene CRBG lavas in the region. The crest of the Portland Hills is defined by the northwest-striking Portland Hills anticline. Sense of displacement on the Portland Hills fault is poorly known and controversial. The fault was originally mapped as a down-to-the-northeast normal fault. The fault has also been mapped as part of a regional-scale zone of right-lateral oblique slip faults and as a steep escarpment caused by asymmetrical folding above a southwest-dipping blind thrust. Reverse displacement with a right-lateral, strike-slip component may be most consistent with the tectonic setting, mapped geologic relations, aeromagnetic data, and microseismicity in the area. Fault scarps on surficial Quaternary deposits have not been described along the fault trace, but some geomorphic (steep, linear escarpment, triangular facets, over-steepened, and knick-pointed tributaries) and geophysical (aeromagnetic, seismic reflection, and ground penetrating radar) evidence suggest Quaternary displacement (Personius, 2012b).

DESIGN EARTHQUAKE

We determined acceleration response spectra for the three postulated scenarios discussed above by using the USGS Interactive Mapping Project that provides a probabilistic site response spectrum for the site assuming bedrock conditions. We assumed an MCE that has a 2 percent probability of exceedance in a 50-year period, as required by the 2014 SOSSC. Some of the major contributing sources to the PGA reported by USGS are presented in Table 2.

Table 2. Partial List of Faults Considered

Source	Magnitude[1] (M_w)	Distance[1] (km)
Cascadia Megathrust (Deep Interface)	9.10	82.70
Portland Hills	6.75	2.93
Cascadia Megathrust (Middle Interface)	8.92	132.72
Grant Butte 50	6.19	8.23

1. Reported by USGS (USGS, 2014)

Figure B-3 shows the site-specific bedrock spectrum as reported by USGS. The soil profile at the site is classified as a Site Class D as prescribed by Section 1613 of SOSSC. Accordingly, the bedrock response spectrum has been amplified using the factors prescribed by SOSSC for Site Class D. Table 3 presents the factors.

Table 3. SOSSC 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 = 0.984 g	S_1 = 0.421 g
Site Coefficient, F	F_a = 1.107	F_v = 1.579
Adjusted Spectral Acceleration, S_M	S_{MS} = 1.088 g	S_{M1} = 0.665 g

Figure B-3 shows adjusted spectrum appropriate for use in design of structures at the site.

GEOLOGIC HAZARDS

In addition to ground shaking, site-specific geologic conditions can influence the potential for earthquake damage. Deep deposits of loose or soft alluvium can amplify ground motions, resulting in increased seismic loads on structures. Other geologic hazards are related to soil failure and permanent ground deformation. Permanent ground deformation could result from liquefaction, lateral spreading, landsliding, and fault rupture. The following sections provide additional discussion regarding potential seismic hazards that could affect the planned development.

FAULT SURFACE RUPTURE

The Oatfield fault is mapped 0.6 mile northeast of the site and the Portland Hills fault is mapped 1.4 miles southwest of the site. Consequently, it is our opinion that the probability of surface fault rupture beneath the site is low.

LIQUEFACTION

Liquefaction is caused by a rapid increase in pore water pressure that reduces the effective stress between soil particles to near zero. Granular soil, which relies on interparticle friction for strength, is susceptible to liquefaction until the excess pore pressures can dissipate. In general, loose, saturated sand soil with low silt and clay content is the most susceptible to liquefaction. Silty soil with low plasticity is moderately susceptible to liquefaction under relatively higher levels of ground shaking

Based on a review of the available information, soil types encountered, and groundwater depth, it is our opinion that liquefaction is not considered a hazard under design levels of ground shaking.

LATERAL SPREAD

Lateral spread is a liquefaction-related seismic hazard. Development areas subject to lateral spreading are typically gently sloping or flat sites underlain by liquefiable sediments adjacent to an open face, such as riverbanks. Liquefied soil adjacent to open faces may “flow” in that direction, resulting in surface cracking and lateral displacement towards the open face (i.e., riverbank). Since the site has low susceptibility to liquefaction, lateral spreading is expected to be negligible at this site.

GROUND MOTION AMPLIFICATION

The soil profile at the site is classified as a Site Class D as prescribed by Section 1613.5.5 of SOSSC. Accordingly, the bedrock response spectrum has been appropriately amplified using the factors prescribed by the code for Site Class D.

LANDSLIDE

Earthquake-induced landsliding generally occurs in steeper slopes comprised of relatively weak soil deposits. The site and surrounding area are relatively flat, and seismically induced landslides are not considered a site hazard.

SETTLEMENT

Settlement due to earthquakes is most prevalent in relatively deep deposits of dry, clean sand. We do not anticipate that seismic-induced settlement in addition to liquefaction-induced settlement will occur during design levels of ground shaking.

SUBSIDENCE/UPLIFT

Subduction zone earthquakes can cause vertical tectonic movements. The movements reflect coseismic strain release accumulation associated with interplate coupling in the subduction zone.

Based on our review of the literature, the locked zone of the CSZ is located in excess of 90 miles from the site. Consequently, we do not anticipate that subsidence or uplift is a significant design concern.

LURCHING

Lurching is a phenomenon generally associated with very high levels of ground shaking, which cause localized failures and distortion of the soil. The anticipated ground accelerations shown on Figure C-3 are below the threshold required to induce lurching of the site soil.

SEICHE AND TSUNAMI

The site is inland and elevated away from tsunami inundation zones and away from large bodies of water that may develop seiches. Seiches and tsunamis are not considered a hazard in the site vicinity.

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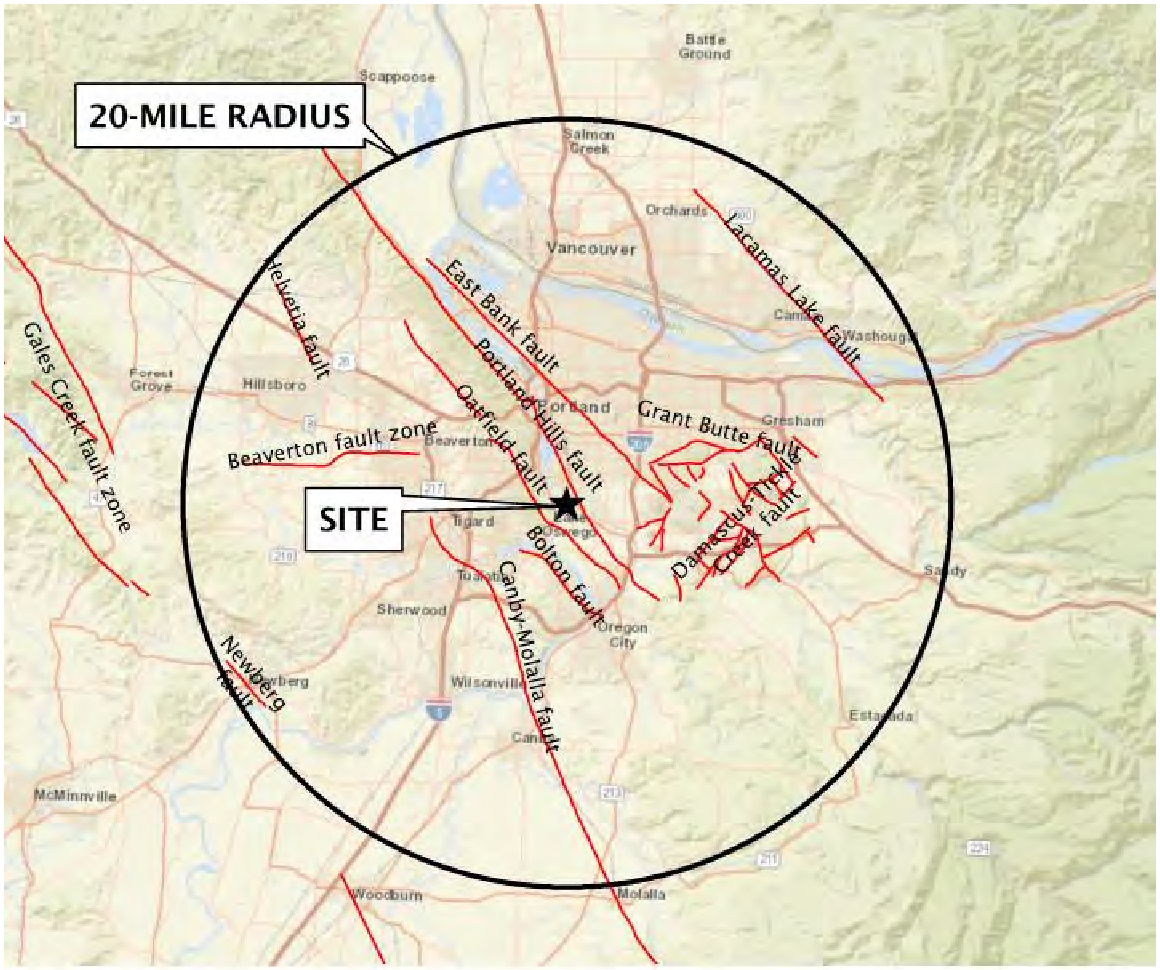
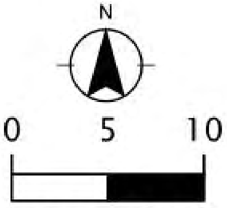
Appendix A
Scott



LEGEND

—— QUATERNARY FAULT

FAULTS PROVIDED BY THE USGS FAULT AND FOLD DATABASE (2006)



(SCALE IN MILES)

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CMILWAUKIE-2-01

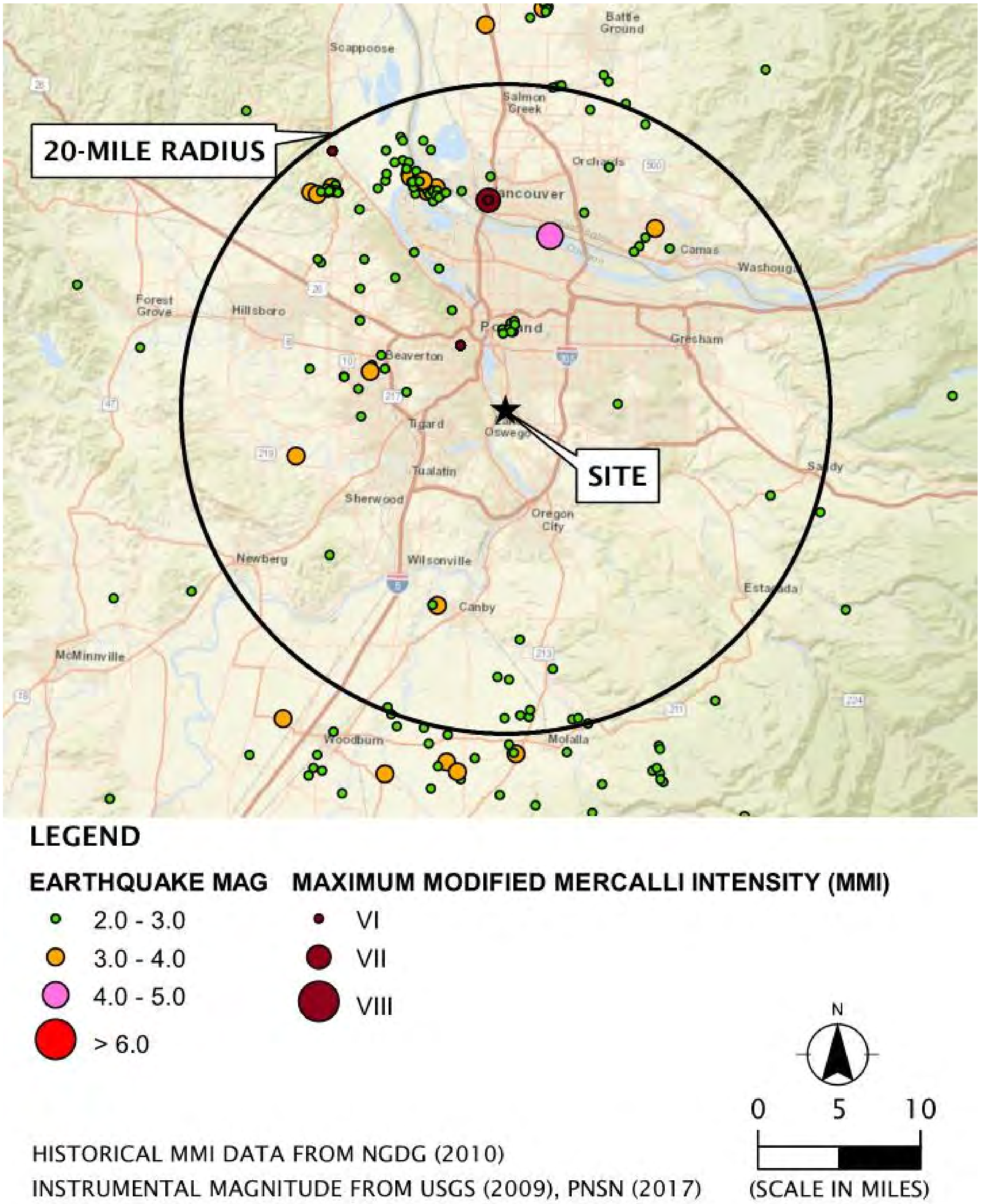
AUGUST 2017

QUATERNARY FAULT MAP

LEDDING LIBRARY OF MILWAUKIE
MILWAUKIE, OR

FIGURE B-1

Appendix A
Scott



HISTORICAL MMI DATA FROM NGDG (2010)
INSTRUMENTAL MAGNITUDE FROM USGS (2009), PNSN (2017)

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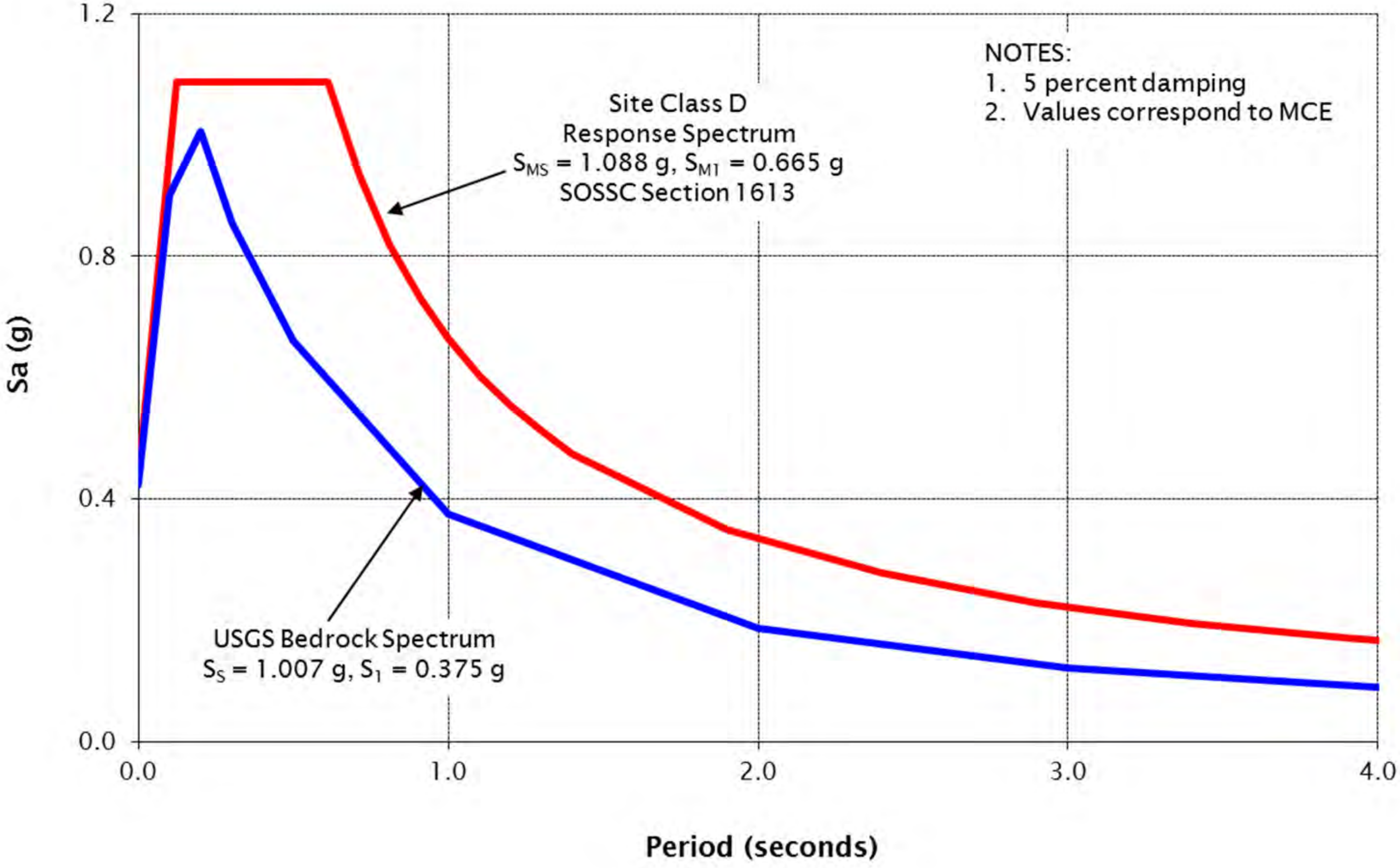
CMILWAUKIE-2-01	**HISTORICAL SEISMICITY MAP**	
AUGUST 2017	LEDDING LIBRARY OF MILWAUKIE MILWAUKIE, OR	**FIGURE B-2**

Appendix A
Scott



Site Class D
Response Spectrum
S_{MS} = 1.088 g, S_{M1} = 0.665 g
SOSSC Section 1613

USGS Bedrock Spectrum
S_S = 1.007 g, S_1 = 0.375 g

NOTES:
1. 5 percent damping
2. Values correspond to MCE

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CMILWAUKIE-2-01

AUGUST 2017

SITE RESPONSE SPECTRA

LEDDING LIBRARY OF MILWAUKIE
MILWAUKIE, OR

FIGURE B-3

Appendix A

Scott

ACRONYMS AND ABBREVIATIONS

ACRONYMS AND ABBREVIATIONS

AASHTO	American Association of State Highway and Transportation Officials
AC	asphalt concrete
ACP	asphalt concrete pavement
ASTM	American Society for Testing and Materials
BGS	below ground surface
CRBG	Columbia River Basalt Group
CSZ	Cascadia Subduction Zone
ESAL	equivalent single-axle load
FHWA	Federal Highway Administration
g	gravitational acceleration (32.2 feet/second^2)
H:V	horizontal to vertical
IBC	International Building Code
km	kilometers
MCE	maximum considered earthquake
MSL	mean sea level
OSHA	Occupational Safety and Health Administration
OSSC	Oregon Standard Specifications for Construction (2015)
pcf	pounds per cubic foot
PG	performance grade
PGA	peak ground acceleration
psf	pounds per square foot
psi	pounds per square inch
SOSSC	State of Oregon Structural Specialty Code
SPT	standard penetration test
USGS	U.S. Geological Survey

Appendix A

Scott

www.geodesigninc.com

June 1, 2022

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

Attention: Adam Moore

Report of Infiltration Testing Services
Milwaukie City Parks
Milwaukie, Oregon
Project: Milwaukie-12-01

INTRODUCTION

NV5 is pleased submit the results of our infiltration testing conducted at two Milwaukie parks: Bowman-Brae Park and Balfour Park. Figure 1 shows the approximate site locations. The test results will be used to evaluate the feasibility of underground injection control (UIC) drywell infiltration systems at each park.

SCOPE OF SERVICES

The purpose of our services was to evaluate the infiltration characteristics of the soil and the feasibility of UIC drywell infiltration systems by conducting infiltration testing. Our specific scope of services, conducted in accordance with our revised proposal dated April 13, 2022, is summarized as follows:

- Coordinated and managed the field investigation, including public and private utility locates and scheduling contractors and NV5 staff.
- Requested One-Call utility locates. Hired a private locate service to provide additional utility location services.
- Drilled one boring at each site to depths of 24.3 and 28.8 feet below ground surface (BGS) at Bowman-Brae Park and Balfour Park, respectively.
- Conducted infiltration testing using the encased falling head procedure at Balfour Park.
- Collected soil samples for laboratory testing, and maintained a detailed log of soil and groundwater conditions encountered in each exploration.

- Performed the following laboratory tests on soil samples collected from the explorations:
    - Five natural moisture content determinations in general accordance with ASTM D2216
    - Two fines content tests (percent passing the U.S. Standard No. 200 sieve) in general accordance with ASTM C117 and/or ASTM D1140 on a sample collected at the infiltration depth at Balfour Park and at 20 feet BGS at Bowman-Brae Park
- Prepared this report that presents our findings, conclusions, and recommendations. This report includes the following:
    - Description of infiltration testing procedures
    - Exploration logs
    - Laboratory results
    - Infiltration testing results

SURFACE CONDITIONS

BOWMAN-BRAE PARK

Bowman-Brae Park is located at the northwest corner of SE Bowman Street and SE Brae Street in Milwaukie, Oregon. The park is bound by SE Bowman Street to the south, neighboring residences to the north and east, and a maintenance shop and grass field to the west. The approximate location of the exploration conducted at the park is shown on Figure 2. The park is generally flat and elevations range from 87 to 83 feet mean sea level (MSL). Surface conditions at the park generally consist of short, maintained grass.

BALFOUR PARK

Balfour Park is located at 3039 SE Balfour Street, approximately 200 feet west of the intersection with SE 32nd Avenue in in Milwaukie, Oregon. Balfour Park is bound by SE Balfour Street to the south and neighboring residences to the north, east, and west. The approximate location of the exploration conducted at the park is shown on Figure 3. The park slopes down from east to west with elevations ranging from 150 to 130 feet MSL. Surface conditions at the park generally consist of short, maintained grass with two narrow paths in the south-central section. Trees and bushes are also present along the perimeter and northern portion of the park.

SUBSURFACE CONDITIONS

GENERAL

Our subsurface exploration program consisted of drilling one boring within each of the park boundaries (B-1 and B-2) to depths of 24.3 and 28.8 feet BGS, respectively. Drilling refusal was encountered in both borings on the underlying gravel. We conducted infiltration testing in B-2 at a depth of 20 feet BGS. The approximate locations of the explorations are shown on Figures 2 and 3. A description of the explorations and laboratory testing program, the exploration logs, and results of our laboratory testing are presented in the Attachment.

Subsurface conditions generally consist of silt, clay, and/or sand overlying very dense gravel to the depths explored in both borings. The following sections provide a detailed description of the units encountered.

BOWMAN-BRAE PARK

Topsoil and grass roots were encountered up to approximately 6 inches BGS followed by stiff clay with sand and organics to a depth of 4.5 feet BGS. Underlying the clay is stiff, sandy silt that becomes soft at 7.5 feet BGS and very soft at 10 feet BGS. Underlying the silt is loose, silty sand at 13 feet BGS. The soil becomes increasingly sandy and denser with depth and changes to very dense gravel at 21 feet BGS to the depth explored. Wet conditions were first encountered at 7.5 feet BGS. After drilling, the groundwater level was monitored and measured at 10.3 feet BGS. Laboratory testing indicates that the moisture content of the soil ranged from 32 to 41 percent and the percent fines content of a sample collected at 20 feet BGS was 29 percent at the time of our explorations.

BALFOUR PARK

A thin section of asphalt concrete and 6 inches of aggregate base were encountered at the surface of boring B-2. Elsewhere at the site, topsoil and grass roots were observed up to approximately 6 inches BGS. Below the surface, soft to stiff silt with sand and interbeds of sandy silt was encountered. At 10 feet BGS, very dense gravel with varying amounts of clay, silt, and sand was encountered to the depth explored. Laboratory testing indicates that the moisture content of the silt is approximately 37 percent and moisture content of the gravel is 12 percent. Based on laboratory testing, the percent fines content is 17 percent at the infiltration depth.

GROUNDWATER

Wet soil conditions were observed within the soil samples collected at depths of 7.5 feet BGS at boring B-1 and greater. Groundwater was measured in B-1 at Bowman-Brae Park after drilling at approximately 10.3 feet BGS. Groundwater was not observed in B-2 at Balfour Park. The depth of groundwater may fluctuate in response to seasonal changes, prolonged rainfall changes in surface topography, and other factors not observed in this study.

INFILTRATION TESTING

Due to the amount and depth of water seepage observed in boring B-1, infiltration testing was not performed. Accordingly, on-site infiltration at Bowman-Brae Park is not feasible. Infiltration testing for Balfour Park was conducted within B-2 at a depth of 20 feet BGS in general accordance with the encased falling head procedure described in Appendix E of the Clackamas County Stormwater Standards.[1] Testing was performed using a 6-inch inside diameter pipe (hollow-stem auger) and approximately 12 to 24 inches of water head after soaking. Table 1 summarizes the measured infiltration rates and fines contents.

Table 1. Infiltration Test Results

Exploration	Depth (feet BGS)	Soil Type at Test Depth	Observed Infiltration Rate (inches per hour)	Percent Passing U.S. Standard No. 200 Sieve
Balfour Park (B-2)	20	Clayey GRAVEL with sand (GC)	4.4	17

---

[1] Clackamas County, 2013, *Stormwater Standards*, Clackamas County Service District No. 1, July 1, 2013.

Fines content was determined on a representative soil sample collected at the depth of the completed infiltration test at Balfour Park. The infiltration rate provided in Table 1 is unfactored.

The infiltration rate shown in Table 1 is a short-term field rate and factors of safety have not been applied for the type of infiltration system being considered. Appropriate correction factors should be applied by the project civil engineer to determine long-term infiltration parameters. Without additional testing, from a geotechnical perspective, we recommend a minimum factor of safety of at least 2 be applied to the field infiltration value presented in Table 1 to account for soil variability with depth. The infiltration system design engineer should determine and apply appropriate remaining correction factor values or factors of safety to account for the degree of in-system filtration, system maintenance, vegetation, potential for siltation, etc.

The infiltration flow rate of a disposal system will diminish over time as suspended solids and precipitates in the stormwater slowly clog the void spaces between the soil particles. Eventually the system may fail and need to be replaced. We recommend that the system include an overflow that is connected to a suitable discharge point. Finally, stormwater infiltration systems will cause localized high groundwater levels; therefore, they should not be located near basement walls, retaining walls, or other embedded structures unless these are specifically designed to account for the resulting hydrostatic pressure.

LIMITATIONS

We have prepared this report for use by the 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 and test results indicate 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 contact us if you have questions regarding this report.

Sincerely,

NV5

Zane M. Rogers, P.E.
Project Manager

Krey D. Younger, P.E., G.E.
Principal Engineer

ZMR:KDY:kt
Attachments
One copy submitted (via email only)
Document ID: Milwaukie-12-01-060122-geolr.docx
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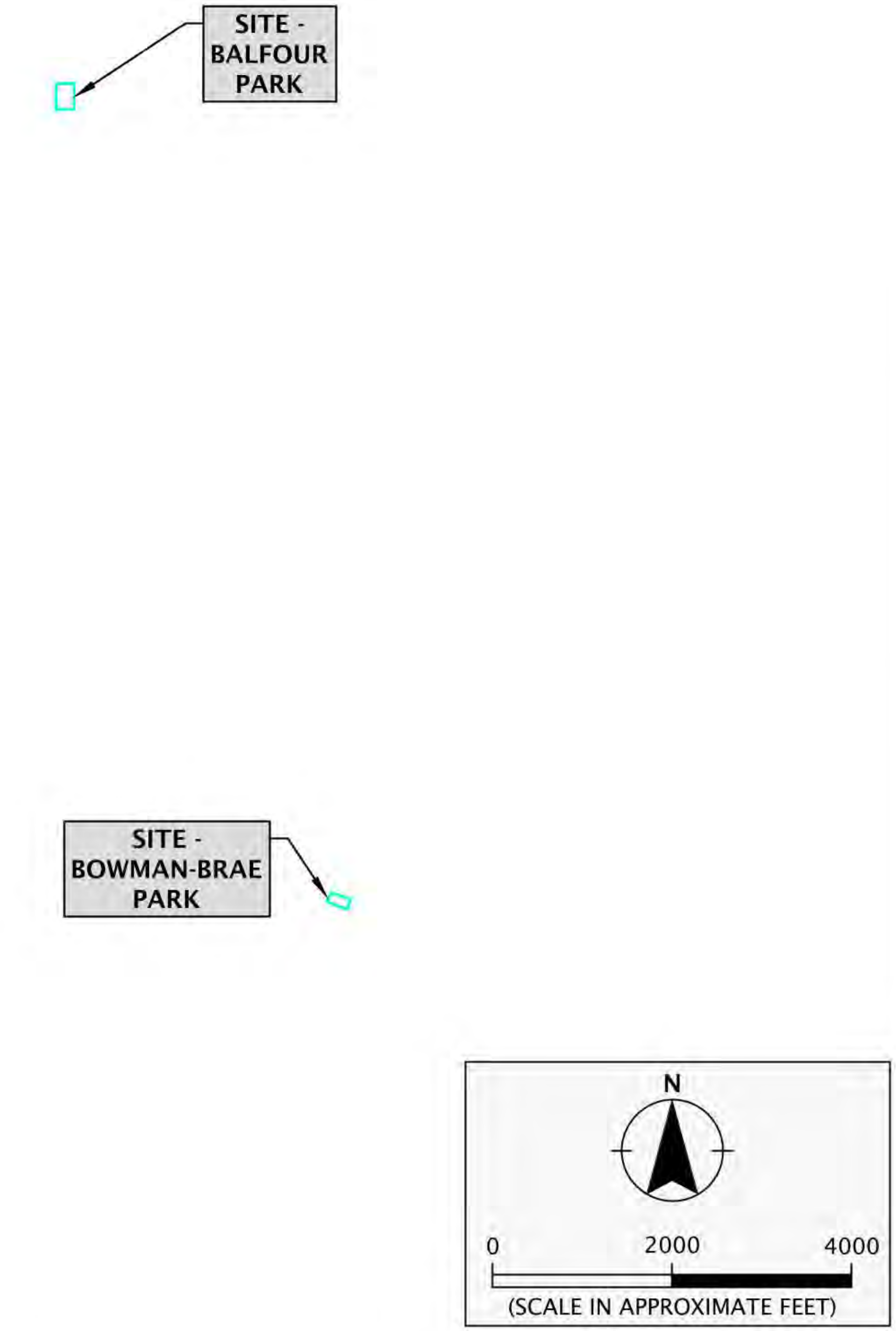
FIGURES



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

NV5	MILWAUKIE-12-01	**VICINITY MAP**	
	JUNE 2022	MILWAUKIE CITY PARKS MILWAUKIE, OR	**FIGURE 1**

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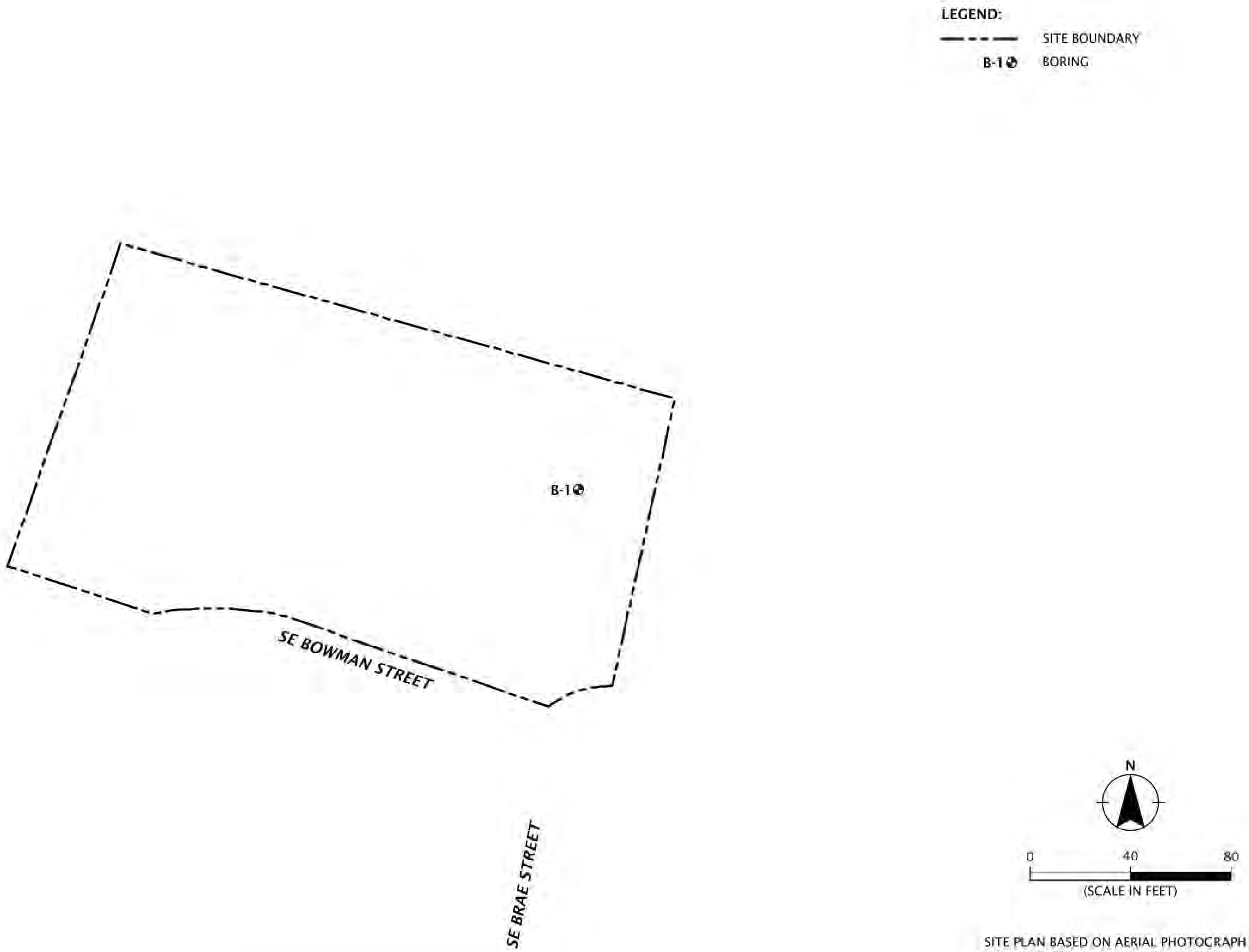
Appendix A
Balfour & Bowman



SITE PLAN BASED ON AERIAL PHOTOGRAPH
OBTAINED FROM GOOGLE EARTH PRO®
MAY 20, 2022

FIGURE 2

SITE PLAN - BOWMAN-BRAE PARK

MILWAUKIE CITY PARKS
MILWAUKIE, OR

MILWAUKIE-12-01

JUNE 2022

NV5

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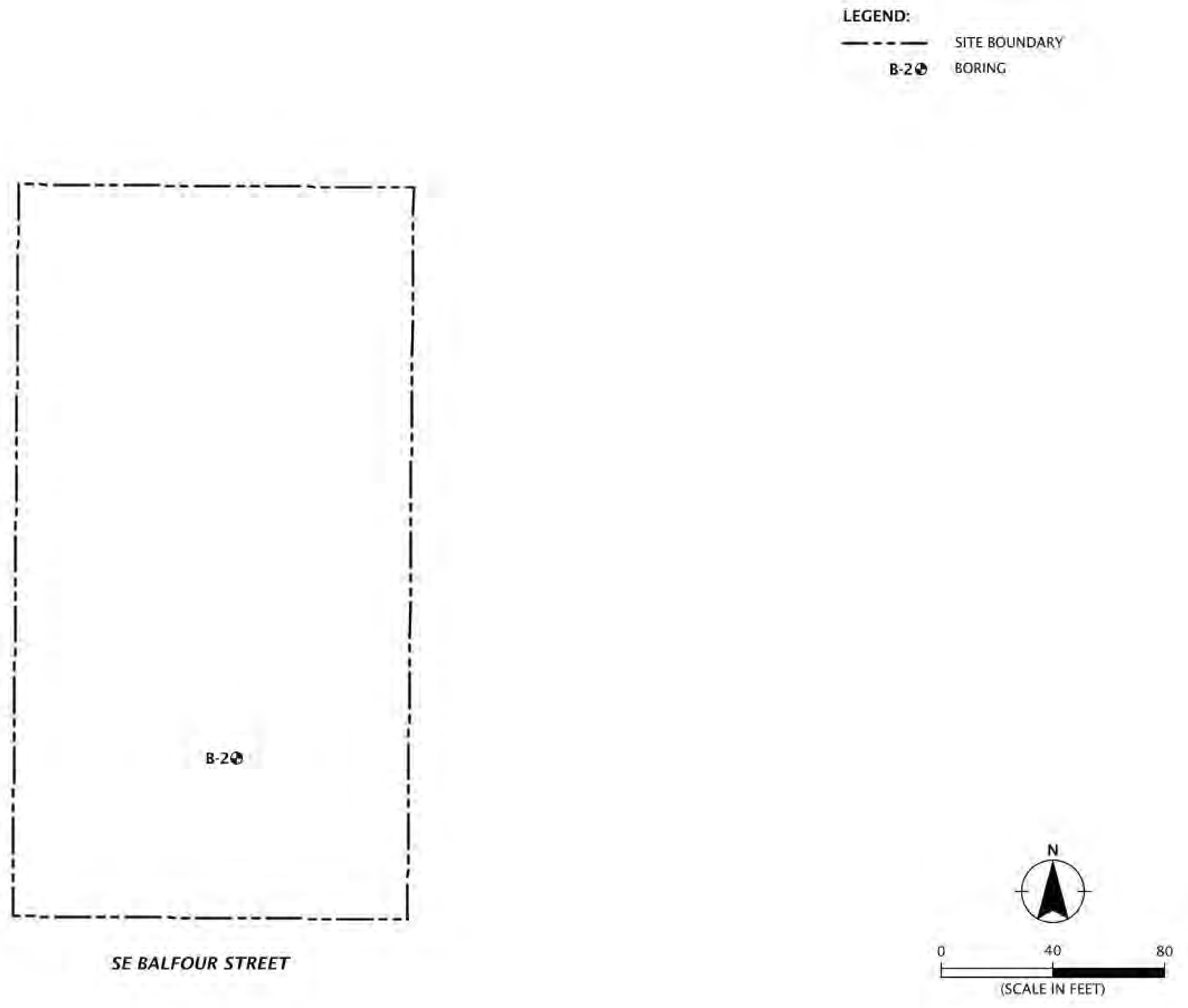
Appendix A
Balfour & Bowman



SITE PLAN BASED ON AERIAL PHOTOGRAPH
OBTAINED FROM GOOGLE EARTH PRO®
MAY 20, 2022

FIGURE 3

SITE PLAN - BALFOUR PARK

MILWAUKIE CITY PARKS
MILWAUKIE, OR

MILWAUKIE-12-01

JUNE 2022

NV5

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ATTACHMENT

ATTACHMENT

FIELD EXPLORATION

GENERAL

We explored subsurface conditions at the site by drilling two borings (B-1 and B-2) to depths of 24.3 and 28.8 feet BGS. The location of the explorations is shown on Figures 2 and 3. The explorations were completed on May 19, 2022, by Western States Soil Conservation, Inc. of Hubbard, Oregon, using a truck-mounted drill rig and hollow-stem auger. The exploration logs are presented in this attachment

The exploration locations were determined by pacing from existing site features and should be considered accurate to the degree implied by the methods used. A member of our geology staff observed the explorations.

SOIL SAMPLING

Disturbed soil samples were collected from the borings using 1½-inside-diameter, split-spoon (SPT) samplers in general accordance with ASTM D1586. Each sampler was driven into the soil with a 140-pound hammer free falling 30 inches. Each sampler was driven a total distance of 18 inches. The number of blows required to drive the sampler 12 inches is recorded on the exploration logs, unless otherwise noted. Disturbed samples were collected from the split barrel for subsequent classification and index testing. Sampling methods and intervals are shown on the exploration logs.

The average efficiency of the automatic SPT hammer used by Western States Soil Conservation, Inc. was 77.5 percent. The calibration testing results are presented at the end of this attachment.

SOIL CLASSIFICATION

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

LABORATORY TESTING

CLASSIFICATION

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

MOISTURE CONTENT

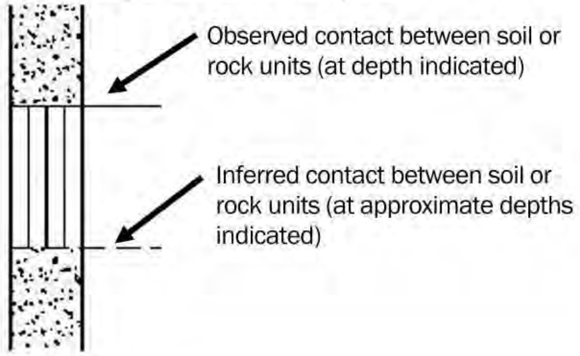
The natural moisture content of select soil samples was determined in general accordance with ASTM D2216. The natural moisture content is a ratio of the weight of the water to soil in a test sample and is expressed as a percentage. The test results are presented in this attachment.

PARTICLE-SIZE ANALYSIS

Particle-size analysis was performed on select soil samples in general accordance with ASTM C117 or ASTM D1140. This test is a quantitative determination of the amount of material finer than the U.S. Standard No. 200 sieve expressed as a percentage of soil weight. The test results are presented in this attachment.

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

Graphic Log of Soil and Rock Types



GEOTECHNICAL TESTING EXPLANATIONS

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

ENVIRONMENTAL TESTING EXPLANATIONS

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

NV5	EXPLORATION KEY	TABLE A-1

Appendix A
Balfour & Bowman

RELATIVE DENSITY - COARSE-GRAINED SOIL

Relative Density	**Standard Penetration Test (SPT) 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 Test (SPT) 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.

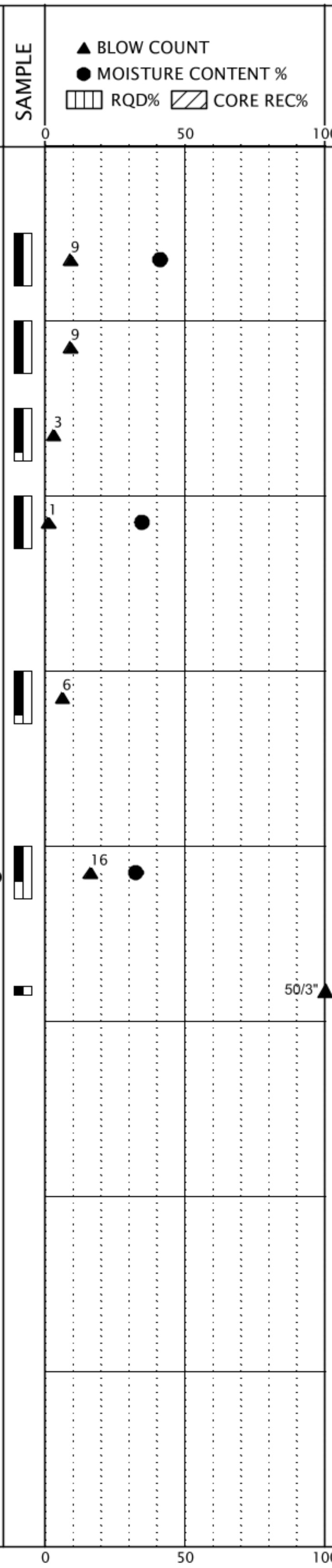
Percent	**Silt and Clay In:**		**Percent**	**Sand and Gravel In:**	
	Fine-Grained Soil	**Coarse-Grained Soil**		**Fine-Grained Soil**	**Coarse-Grained Soil**
< 5	trace	trace	< 5	trace	trace
5 – 12	minor	with	5 – 15	minor	minor
> 12	some	silty/clayey	15 – 30	with	with
			> 30	sandy/gravelly	Indicate %

NV5

SOIL CLASSIFICATION SYSTEM

TABLE A-2

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	INSTALLATION AND COMMENTS
0	Stiff, gray-brown with orange mottled CLAY (CL), trace sand and organics; moist (topsoil to 4 to 6 inches, 2-inch-thick root zone).			
5	Stiff, brown with orange mottled, sandy SILT (ML); moist, sand is fine to medium.	4.5		
	soft; wet, interbeds of dark brown SAND (0.5 to 1 inch thick) at 7.5 feet light brown; sand is fine, rapid dilatancy at 8.0 feet very soft, gray at 10.0 feet			Water at 7.5 feet. ▼ 10.3 feet, after drilling
15	Loose, gray, silty SAND (SM), trace debris (wood bark); wet, sand is fine.	13.0		
20	Medium dense, gray SAND (SP), trace silt; wet, sand is medium, interbeds of SILT, trace debris (wood) (approximately 4 inches thick).	18.0	P200	P200 = 29%
	Very dense, gray with green mottled, silty GRAVEL with sand (GM); wet, gravel is angular.	21.0		
25	Exploration terminated at a depth of 24.3 feet due to refusal. Hammer efficiency factor is 77.5 percent.	24.3		Surface elevation was not measured at the time of exploration.



DEPTH FEET: 0, 5, 10, 15, 20, 25, 30, 35, 40

DRILLED BY: Western States Soil Conservation, Inc.
LOGGED BY: H. Herinckx
COMPLETED: 05/19/22

BORING METHOD: hollow-stem auger (see document text)
BORING BIT DIAMETER: 6 inches

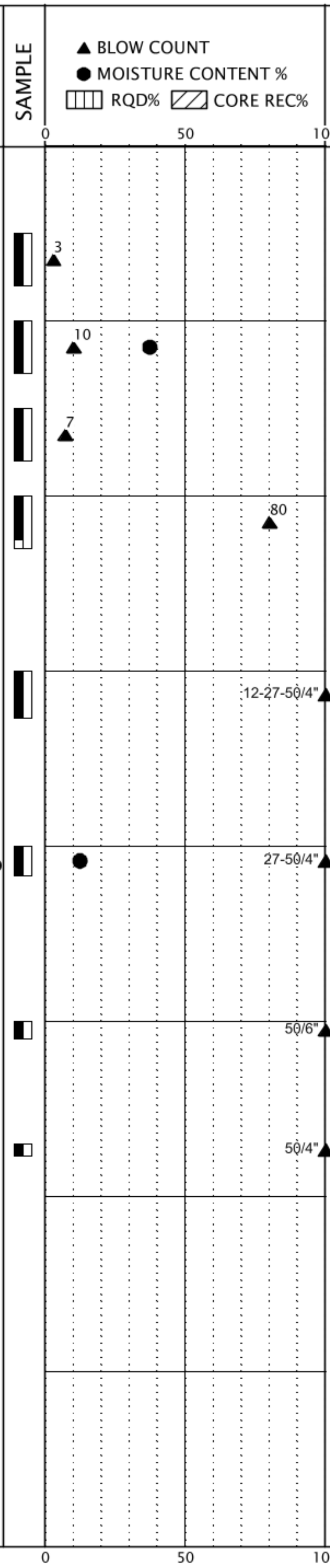
MILWAUKIE-12-01
JUNE 2022

BORING B-1

MILWAUKIE CITY PARKS
MILWAUKIE, OR

FIGURE A-1

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	INSTALLATION AND COMMENTS
0	ASPHALT CONCRETE (1.0 inch).	0.1		Topsoil to 4 to 6 inches, 2- to 3-inch-thick root zone inferred on site.
	AGGREGATE BASE (6.0 inches).	0.6		
	Soft, brown SILT with sand (ML), trace organics; moist, sand is fine to medium.			
5	stiff at 5.0 feet			
	medium stiff, without organics; moist to wet, interbeds of sandy SILT at 7.5 feet			
10	Very dense, brown-gray with yellow-orange mottled GRAVEL with silt and sand (GP-GM); moist, gravel is rounded to angular.	10.0		
	Very dense, brown with yellow mottled, clayey GRAVEL with sand (GC); moist, gravel is fine to coarse and subrounded to subangular, sand is fine to coarse.	13.5		
15				
20			P200	Infiltration test at 20.0 feet. P200 = 17%
	Very dense, gray-brown GRAVEL with clay and sand (GP-GC); moist, gravel is rounded to subangular.	23.0		
25				
	Exploration terminated at a depth of 28.8 feet due to refusal. Hammer efficiency factor is 77.5 percent.	28.8		Surface elevation was not measured at the time of exploration.
30				
35				
40				



DRILLED BY: Western States Soil Conservation, Inc.
LOGGED BY: H. Herinckx
COMPLETED: 05/19/22

BORING METHOD: hollow-stem auger (see document text)
BORING BIT DIAMETER: 6 inches

MILWAUKIE-12-01
JUNE 2022

BORING B-2

MILWAUKIE CITY PARKS
MILWAUKIE, OR

FIGURE A-2

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		41							
B-1	10.0		34							
B-1	20.0		32				29			
B-2	5.0		37							
B-2	20.0		12				17			

LAB SUMMARY - GDI-NV5 MILWAUKIE-12-02-B1_2.GPJ GDI_NV5.GDT PRINT DATE: 5/27/22:KT

MILWAUKIE-12-01

JUNE 2022

SUMMARY OF LABORATORY DATA

MILWAUKIE CITY PARKS
MILWAUKIE, OR

FIGURE A-3

Pile Dynamics, Inc.
SPT Analyzer Results

RIG #4
PDA-S Ver. 2021.34 - Printed: 12/27/2021

Summary of SPT Test Results

Project: WSSC-8-06, Test Date: 12/23/2021

FMX: Maximum Force
VMX: Maximum Velocity
BPM: Blows/Minute

EFV: Maximum Energy
ETR: Energy Transfer Ratio - Rated

Instr. Length ft	Blows Applied /6"	**N Value**	**N60 Value**	Average FMX kips	Average VMX ft/s	Average BPM bpm	Average EFV ft-lb	Average ETR %
60.00	**4-6-15**	**21**	**27**	40	13.0	51.6	267	76.4
60.00	**5-11-8**	**19**	**24**	41	13.0	58.5	288	82.4
60.00	**7-14-15**	**29**	**37**	41	13.0	57.0	274	78.2
60.00	**7-12-18**	**30**	**38**	40	13.0	49.9	266	76.0
60.00	**4-19-19**	**38**	**49**	40	12.5	51.7	267	76.2
			Overall Average Values:	41	12.9	53.3	271	77.5
			Standard Deviation:	1	0.6	3.4	9	2.7
			Overall Maximum Value:	43	15.1	58.9	296	84.5
			Overall Minimum Value:	37	11.8	38.1	251	71.7

Appendix B

MLL Preliminary Stormwater Report

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Harper
Houf Peterson
Righellis Inc.

Milwaukie Ledding Library

Preliminary
Stormwater Management Report

January 11, 2018

Prepared For:

Hacker Architects
733 SW Oak St
Portland, OR 97205

THA-29

Prepared By:

Harper Houf Peterson Righellis Inc.
205 SE Spokane Street, Suite 200
Portland, OR 97202
P: 503-221-1131 F: 503-221-1171

Alex Simpson, PE

HHPR

ENGINEERS ◆ PLANNERS
LANDSCAPE ARCHITECTS ◆ SURVEYORS

Design Review – Preliminary Stormwater Management Report
Milwaukie Ledding Library

Prepared by: Harper Houf Peterson Righellis, Inc.
Date: January 11, 2018

Project Overview and Description:

The new Milwaukie Ledding Library project is located at 10660 SE 21st Avenue in Milwaukie, OR. The total site area is 1.77 acres. It is bordered to the west by private apartments, to the south by SE Harrison St and to the east by an existing pond and Spring Creek. The proposed project will construct a new library building with associated parking lot and stormwater management facilities.

Methodology

The site's impervious surfaces will be managed per the City of Milwaukie's Stormwater Design Standards, updated in January 2014. The City of Milwaukie refers to the 2016 City of Portland Stormwater Management Manual (SWMM) for design of water quality and flow control facilities. Per the SWMM, the Stormwater Infiltration and Discharge Hierarchy is to be used to determine the feasibility of the stormwater option to be used for the site. The following addresses each category in the Hierarchy:

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

> *On-site infiltration with vegetated infiltration facilities is not feasible for this project due to the low infiltration rates on site (less than 1 in/hr.) and existing site constraints.*

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

> *On-site infiltration with vegetated infiltration facilities is not feasible for this project due to the low infiltration rates on site (less than 1 in/hr) and existing site constraints.*

Category 3: Requires onsite detention with vegetated facilities that overflow to a drainage way, river, or storm-only pipe.

> ***This category applies to the site. The entire site, with the exception of a portion of the building roof, discharge to a storm-only pipe. Therefore, the SWMM requires that post-developed peak flows be maintained at their respective pre-developed peak flows for the 2, 5, 10-year events. The portion of building roof that discharges directly to the existing pond must limit the 2-year post-developed rate to ½ of the 2-year pre-developed rate, as well as match the post-developed peak rate to the respective pre-developed 5, 10, and 25 year rates.***

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

> *This category does not apply, as there is not a combined sewer system nearby and category 3 will be met.*

Drainage Design & Analysis:

Pre-developed conditions, as stated in the City of Milwaukie standards, are the existing conditions prior to redevelopment. The existing project area consists of the library and asphalt pavement parking lot.

The existing library building had a roof area of approximately 7,300 SF and discharged directly to the pond without flow control or water quality treatment. The proposed development only contributes a portion of the new building roof (Basin E = 6,270 SF) to the pond and provides both water quality and flow control. Therefore, the peak flow conveyed to the pond is reduced per City requirements. The total project's impervious area is increased by approximately 9,144 SF (0.21 ac). Flow control is provided through the stormwater planters and the overall peak flows leaving the site are decreased to meet City requirements. See EX-1, EX-2 and the 'Stormwater Flow Control' section of this report for additional information.

Stormwater facilities were sized using the City of Portland SWMM and Presumptive Approach Calculator (PAC) to provide both water quality and flow control for the project. They are all designed with 2" of freeboard, a varying amount of ponding depth (see PAC printouts), 18" of treatment growing medium, and 12" of drain rock with a perforated underdrain pipe that will connect to the site's storm system. The planters are also lined with an impervious liner due to poor site infiltration and proximity to the building.

There are two existing stormwater swales located in the SW corner of the site that provide stormwater management for a portion of SE 21st Avenue. These existing swales were constructed as part of the N. Main Streetscape Improvement Project in 2005. According to the approved stormwater design, these swales provide existing stormwater management for 5,600 SF (0.13 ac) of impervious drive aisle. These swales will be retained and will provide management for a reduced area of approximately 4,200 SF (0.10 ac) from the proposed drive aisle. See exhibit EX-2 for further clarification.

Stormwater Quality Treatment

In order to provide water quality treatment for the new parking lot and building roof, stormwater planters and a Contech Stormfilter catch basin are used. See Table 1 below and refer to the basin map and PAC output attached for clarification.

Table 1: Stormwater Basin Summary

Basin	Impervious Area (sf)	Treatment Method	Stormwater Facility Size
A (North prkg lot)	4,900	Stormwater Planter	120 sf
B (Center prkg lot)	4,400	Stormfilter WQ Catch Basin	1-cartridge
C (South prkg lot)	3,150	Stormwater Planter	150 sf
D (North bldg. roof)	5,266	Stormwater Planter	100 sf
E (East bldg. roof)	6,270	Stormwater Planter	300 sf
F (South bldg. roof)	11,858	Stormwater Planter	490 sf
G (South prkg lot)	4,200	Existing Swales (SW)	425 sf

Stormwater Flow Control

Flow control is provided through the stormwater planters in order to meet City of Portland requirements. See Table 2 below for a flow control summary. Per the City of Portland 2016 Stormwater Management Manual, on-site infiltration is not feasible when the site has infiltration rates less than 2.0 inches per hour. This site has infiltration rates of 1” per hour or less (without a factor of safety). Refer to the infiltration section 3.4 of the geotechnical report completed by GeoDesign, Inc. on August 25, 2017.
The SWMM requires that post-developed peak flows be maintained at their respective pre-developed peak flows for the 2, 5, 10-year events when discharging to the storm only system. Basins A, B, C, D, and F all meet this criteria.

Flows that discharge directly to the existing pond must limit the 2-year post-developed rate to ½ of the 2-year pre-developed rate, as well as match the post-developed peak rate to the respective pre-developed 5, 10, and 25-year rates. Basin E (east building roof) meets this criteria.

Table 2: Flow Control Summary

Basin	Pre-dev. 2-year peak (cfs)	Pre-dev. 5-year peak (cfs)	Pre-dev. 10-year peak (cfs)	Post-dev. 2-year peak (cfs)	Post-dev. 5-year peak (cfs)	Post-dev. 10-year peak (cfs)
A (North prkg lot)	0.069	0.084	0.100	0.069	0.084	0.100
B Center prkg lot)	0.062	0.076	0.090	0.062	0.076	0.090
C (South prkg lot)	0.044	0.054	0.064	0.009	0.009	0.020
D (North bldg. roof)	0.074	0.091	0.107	0.074	0.091	0.107
F (South bldg. roof)	0.072	0.105	0.140	0.023	0.042	0.124
TOTAL	**0.321**	**0.410**	**0.501**	**0.237**	**0.302**	**0.441**

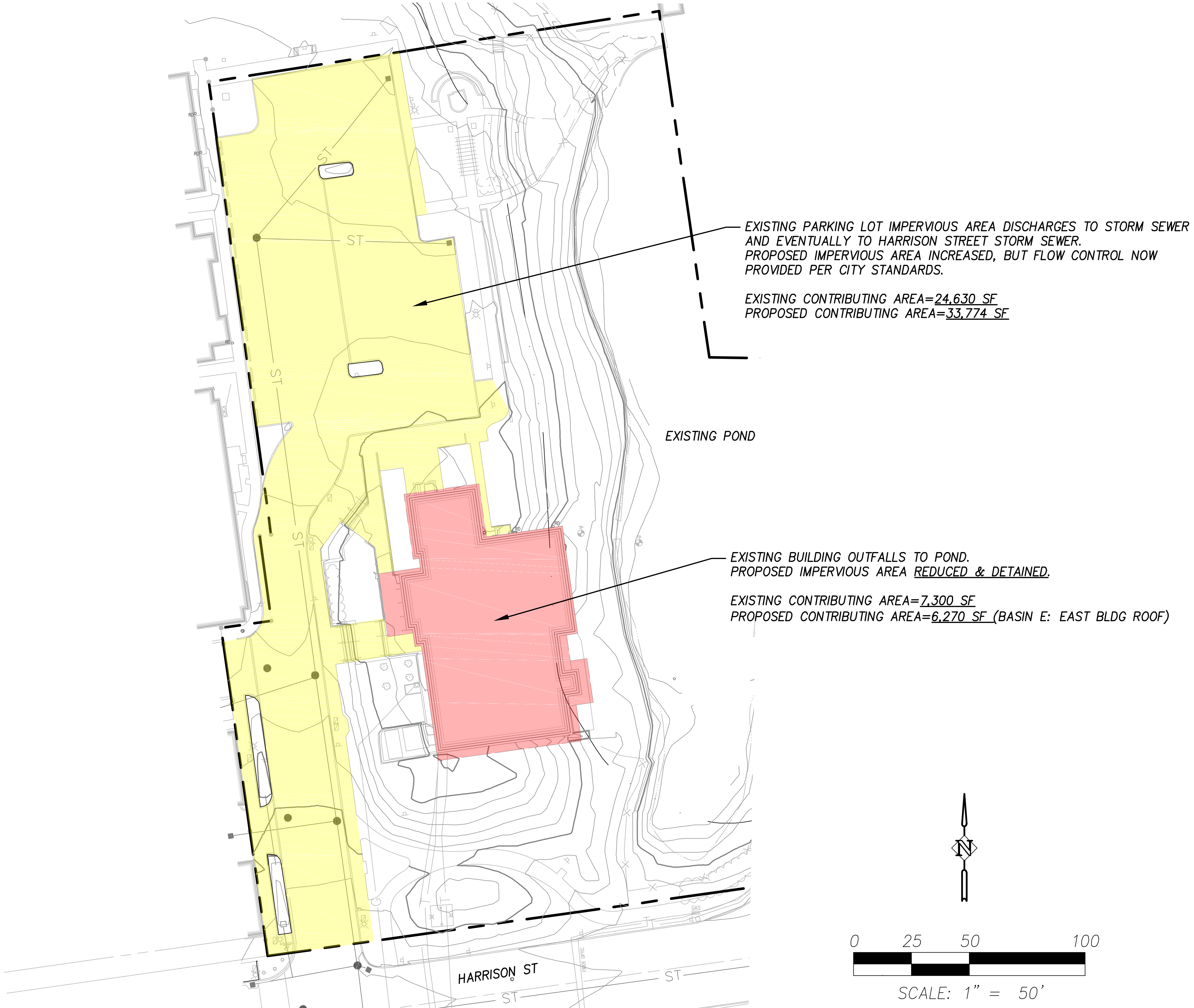
Basin	Pre-dev. ½ of 2-year peak (cfs)	Pre-dev. 5-year peak (cfs)	Pre-dev. 10-year peak (cfs)	Pre-dev. 25-year peak (cfs)	Post-dev. 2-year peak (cfs)	Post-dev. 5-year peak (cfs)	Post-dev. 10-year peak (cfs)	Post-dev. 25-year peak (cfs)
E (East bldg. roof)	0.038	0.097	0.118	0.138	0.014	0.014	0.026	0.076

As seen in the tables above, the total post-developed release rates for the project are less than their respective pre-developed release rates as required by the City of Portland's SWMM.

Engineering Conclusions:

The proposed development has appropriate stormwater facilities and a system that fulfills the required conveyance, water quality and water quantity based on City of Milwaukie and City of Portland requirements and standards. No downstream deficiencies are expected.

BASIN MAP



EXISTING PARKING LOT IMPERVIOUS AREA DISCHARGES TO STORM SEWER AND EVENTUALLY TO HARRISON STREET STORM SEWER.
PROPOSED IMPERVIOUS AREA INCREASED, BUT FLOW CONTROL NOW PROVIDED PER CITY STANDARDS.

EXISTING CONTRIBUTING AREA=24,630 SF
PROPOSED CONTRIBUTING AREA=33,774 SF

EXISTING BUILDING OUTFALLS TO POND.
PROPOSED IMPERVIOUS AREA REDUCED & DETAINED.

EXISTING CONTRIBUTING AREA=7,300 SF
PROPOSED CONTRIBUTING AREA=6,270 SF (BASIN E: EAST BLDG ROOF)

DATE	NO.	DESCRIPTION

R E V I S I O N S

DESIGNED: AJS
DRAWN: HHPR
CHECKED: HHPR
DATE: JAN. 2018

Harper **H**ouf **P**eterson **R**ighellis **I**nc.
ENGINEERS • PLANNERS
LANDSCAPE ARCHITECTS • SURVEYORS
205 SE Spokane Street, Suite 200, Portland, OR 97202
phone: 503.221.1131 www.hhpr.com fax: 503.221.1171

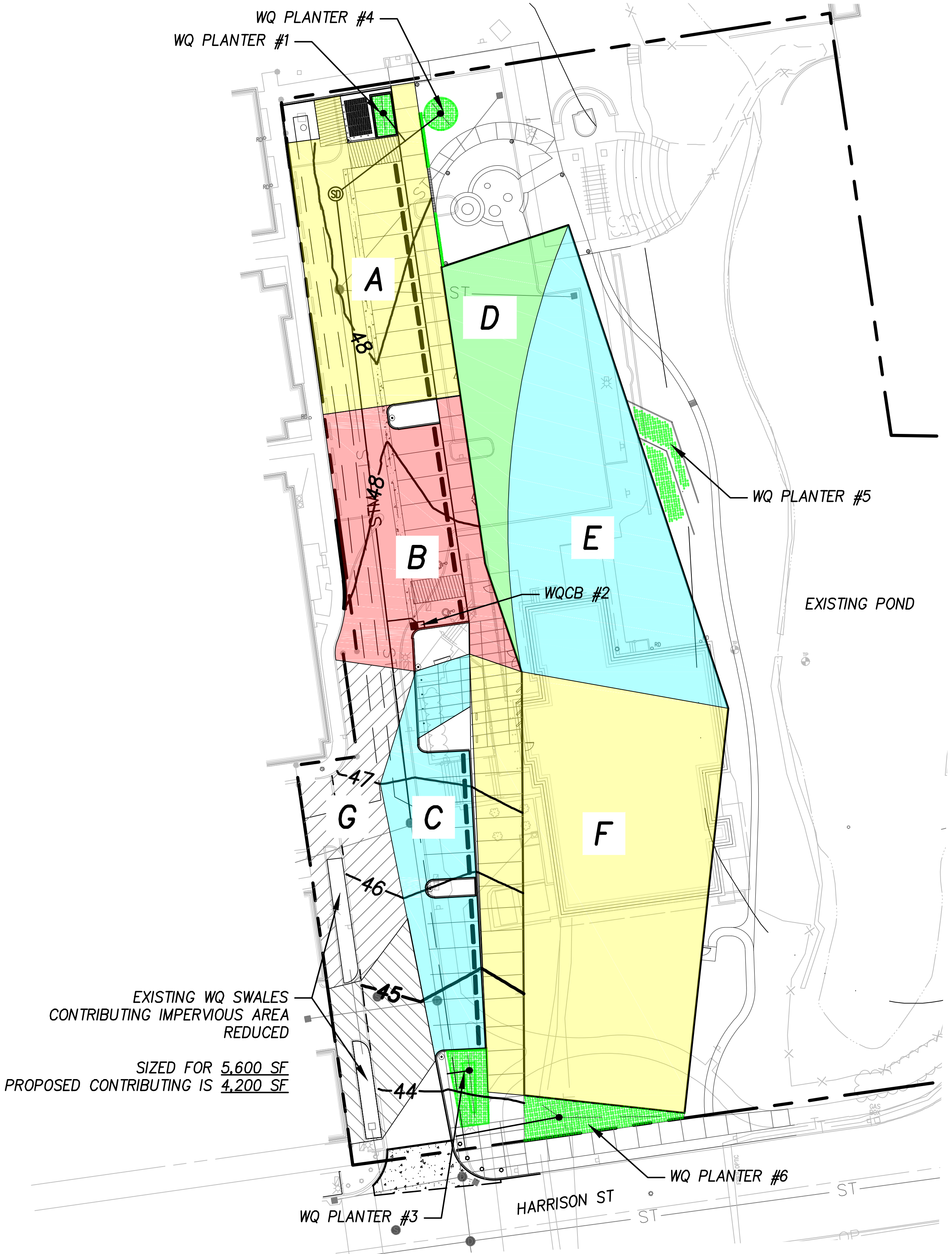
EXISTING STORMWATER EXHIBIT
MILWAUKIE LEDDING LIBRARY
MILWAUKIE, OR

SHEET NO.
EX-1

JOB NO.
THA-29

P:\THA (THA Architecture)\THA-29 (Ledding Library)\THA29-DWGS\Exhibit\ THA29-Storm Exhibit EXISTING.dwg

P:\THA (THA Architecture)\THA-29 (Ledding Library)\THA29-DWGS\Exhibit\ THA29-Storm Exhibit.dwg

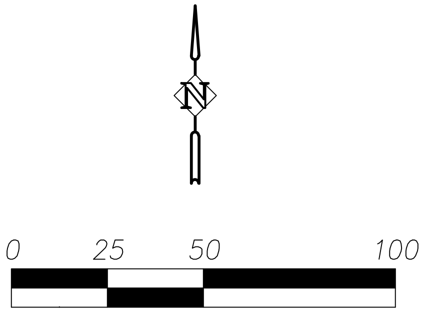


EXISTING WQ SWALES CONTRIBUTING IMPERVIOUS AREA REDUCED

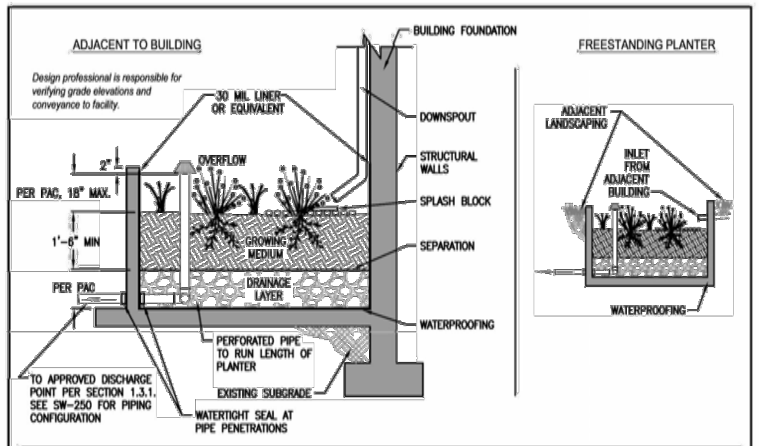
SIZED FOR 5,600 SF
PROPOSED CONTRIBUTING IS 4,200 SF

Basin	Impervious Area	WQ Facility	Ultimate Discharge Location
A (North prkg lot)	4,900 SF	1	Storm Sewer (Harrison St)
B (Center prkg lot)	4,400 SF	2	Storm Sewer (Harrison St)
C (South prkg lot)	3,150 SF	3	Storm Sewer (Harrison St)
D (North bldg roof)	5,266 SF	4	Storm Sewer (Harrison St)
E (East bldg roof)	6,270 SF	5	Existing Pond (East)
F (South bldg roof)	11,858 SF	6	Storm Sewer (Harrison St)

WQ Facility	Stormwater Facility Type	Basin Area	Proposed Facility Size
1	WQ Planter (Flat)	4,900 SF	120 SF
2	WQ CB (1-CART)	4,400 SF	1-cartridge
3	WQ Planter (Flat)	3,150 SF	150 SF
4	WQ Planter (Flat)	5,266 SF	100 SF
5	WQ Planter (Flat)	6,270 SF	300 SF
6	WQ Planter (Flat)	11,858 SF	490 SF



SCALE: 1" = 50'



1. Detail intended as an example. Detail must match PAC assumptions and/or design report.
2. Dimensions:
   Width of planter: 24" minimum.
   Depth of planter (from top of growing medium to overflow elevation): per PAC calculations.
   Longitudinal slope of planter: 0.5% or less.
3. Setbacks:
   Planters must be less than 30" in height above finish grade if within 5-feet of property line.
4. Planter Walls:
   Material must be monolithically poured concrete, unless otherwise approved. Walls must be included on foundation plans.
5. Waterproofing:
   If planter is monolithically poured no additional liner/waterproofing is required. Check state structural requirements for foundations.
6. Piping must be ABS Sch.40, cast iron, or PVS Sch.40. 3" pipe required for facilities draining up to 1500 s.f., otherwise 4" min. pipe. Piping must have 1% grade and follow the Uniform Plumbing Code.
7. Drain Layer:
   Determined by designer. Options include, but are not limited to drain mat, 3/4" washed round rock, or other approved system.
   Separation between drain and growing medium:
   Use appropriate filter fabric or a gravel lens (3/4 - 1/4 inch washed, crushed rock 2 to 3 inches deep), or as per approved design.
8. Overflow:
   Inlet elevation must allow for 2" of freeboard, minimum.
   Protect from debris and sediment with strainer or grate.
9. Growing Medium:
   18" minimum depth. Use sand/loam/compost 3-way mix, or approved mix that will support healthy plants.
   24" minimum depth is required if the lined facility is also meeting BDS landscape requirements.
10. Vegetation: Refer to plant list in SWMM, Section 2.4.1. Minimum container size is #1 container. # of plantings per 100sf of facility area:
    80 herbaceous plants OR;
    72 herbaceous plants and 4 small shrubs.
11. Splash Block: Install 4-6" washed river rock or splash pad for erosion control at inlets and downspout.
12. Inspections: Call BDS MR Inspection Line, (503) 823-7000, request 487. 3 inspections required.

- DRAWING NOT TO SCALE -

STORMWATER MANAGEMENT TYPICAL DETAILS

– Presumptive and Performance Design Approach –
Planter – lined

Bureau of Environmental Services

NUMBER
SW-230
7-1-16

DATE	NO.	DESCRIPTION	
			DESIGNED: AJS
			DRAWN: HHPR
			CHECKED: HHPR
		R E V I S I O N S	DATE: JAN. 2018

HHPR
Harper
Houf Peterson
Righellis Inc.
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STORMWATER EXHIBIT
MILWAUKIE LEDDING LIBRARY
MILWAUKIE, OR

SHEET NO.
EX-2

JOB NO.
THA-29

PAC CALCULATIONS

PAC Report

Project Name Milwaukie Ledding Library	Permit No.	Created 12/4/17 1:03 PM
Project Address 10660 SE 21st Avenue Milwaukie, OR 97222	Designer HHPR	Last Modified 1/11/18 1:26 PM
	Company HHPR	Report Generated 1/11/18 1:26 PM

Project Summary

New public library and site.

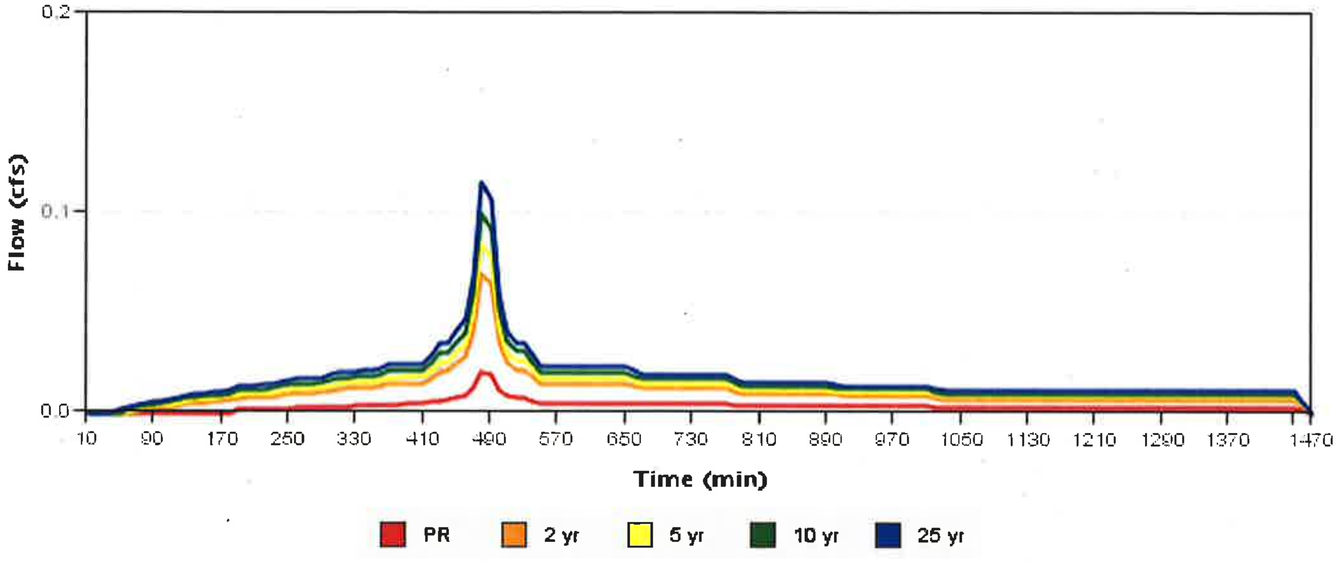
Catchment Name	Impervious Area (sq ft)	Native Soil Design Infiltration Rate	Hierarchy Category	Facility Type	Facility Config	Facility Size (sq ft)	Facility Sizing Ratio	PR Results	Flow Control Results
North Parking Lot	4900	0.00	3	Planter (Flat)	D	100	2%	Pass	Pass
East Roof	6270	0.00	3	Planter (Flat)	D	300	4.8%	Pass	Pass
South Roof	11858	0.00	3	Planter (Flat)	D	490	4.1%	Pass	Pass
North Roof	5266	0.00	3	Planter (Flat)	D	100	1.9%	Pass	Pass
Center Parking Lot	4400	0.00	3	WQ Catch Basin					
South Parking Lot (New Planter)	3150	0.00	3	Basin	D	51	8.1%	Pass	Pass

Catchment North Parking Lot

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	**C**
	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	**The post-development peak rates for the 2, 5 and 10-year design storms must be equal or less than the pre-development rates.**
	Impervious Area	**4900 sq ft** **0.112 acre**
	Time of Concentration (Tc)	**5**
	Pre-Development Curve Number (CN_{pre})	**98**
	Post-Development Curve Number (CN_{post})	**98**

⚠ Indicates value is outside of recommended range

SBUH Results



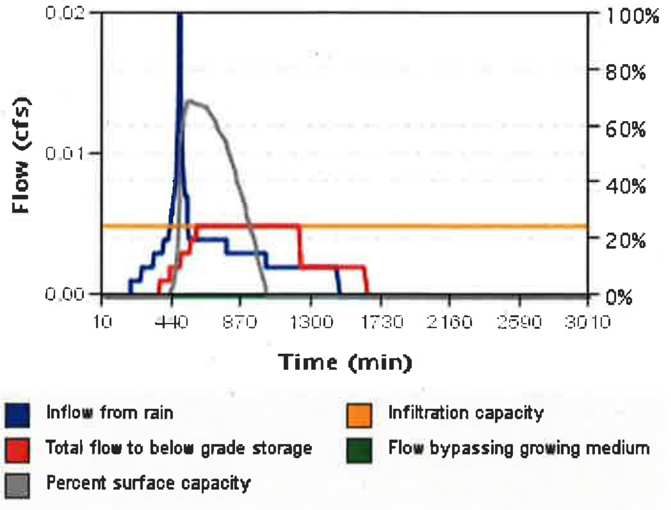
	Pre-Development Rate and Volume		Post-Development Rate and Volume	
	Peak Rate (cfs)	**Volume (cf)**	**Peak Rate (cfs)**	**Volume (cf)**
PR	0.02	256.039	0.02	256.039
2 yr	0.069	886.635	0.069	886.635
5 yr	0.084	1089.719	0.084	1089.719
10 yr	0.1	1293.106	0.1	1293.106
25 yr	0.115	1496.686	0.115	1496.686

Facility North Parking Lot

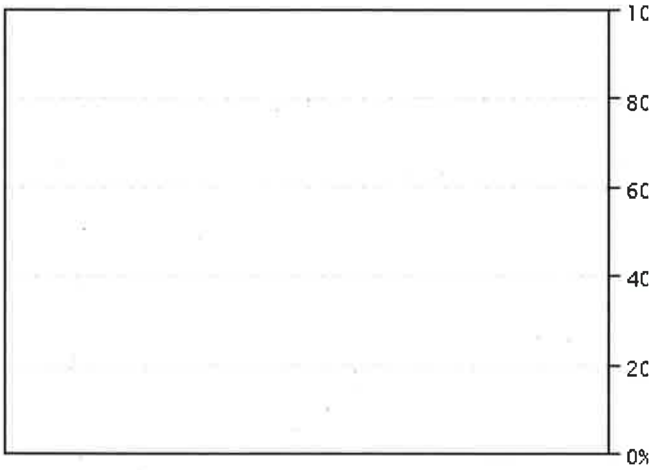
Facility Details	Facility Type	**Planter (Flat)**
	Facility Configuration	**D: Lined Facility with RS and Ud**
	Facility Shape	**Planter**
	Above Grade Storage Data	
	Bottom Area	**100 sq ft**
	Bottom Width	**4.00 ft**
	Storage Depth 1	**6.0 in**
	Growing Medium Depth	**18 in**
	Surface Capacity at Depth 1	**50.0 cu ft**
	Design Infiltration Rate for Native Soil	**0.000 in/hr**
	Infiltration Capacity	**0.005 cfs**
Facility Facts	Total Facility Area Including Freeboard	**100.00 sq ft**
	Sizing Ratio	**2%**
Pollution Reduction Results	Pollution Reduction Score	**Pass**
	Overflow Volume	**260.433 cf**
	Surface Capacity Used	**69%**
Flow Control Results	Flow Control Score	**Pass**
	Overflow Volume	**1292.309 cf**
	Surface Capacity Used	**100%**

	Post-development outflow (cfs)		**Pre-development inflow (cfs)**	
2 year	0.069	≤	0.069	**Pass**
5 year	0.084	≤	0.084	**Pass**
10 year	0.1	≤	0.1	**Pass**

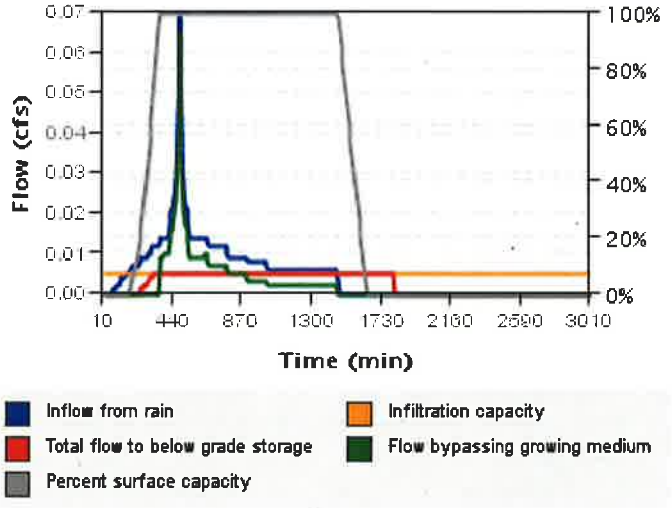
Pollution Reduction Event Surface Facility Modeling



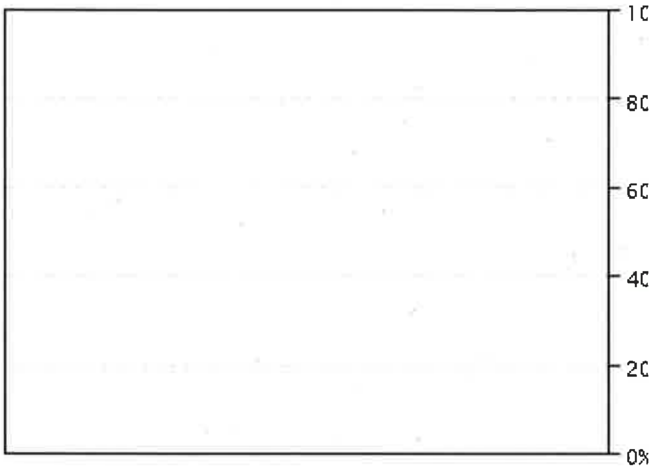
Pollution Reduction Event Below Grade Modeling



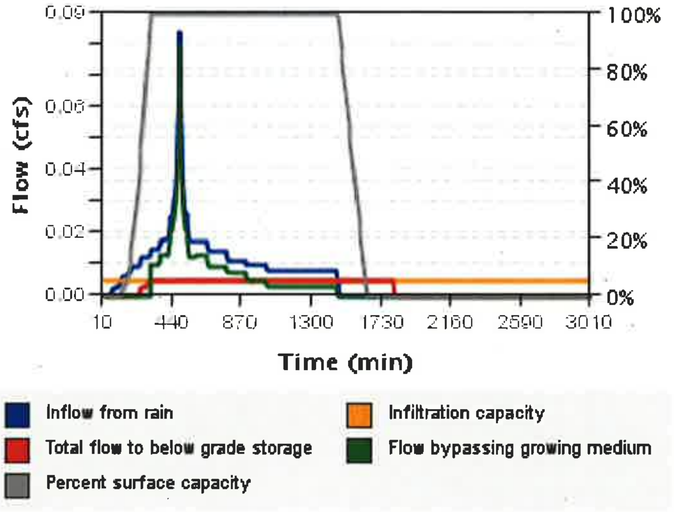
2 Year Event Surface Facility Modeling



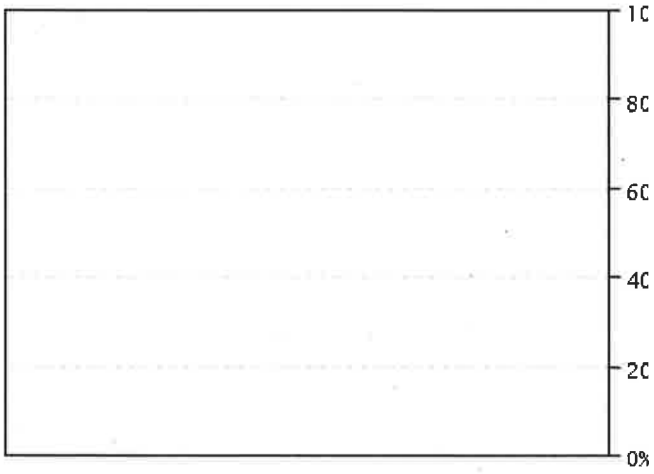
2 Year Event Below Grade Modeling

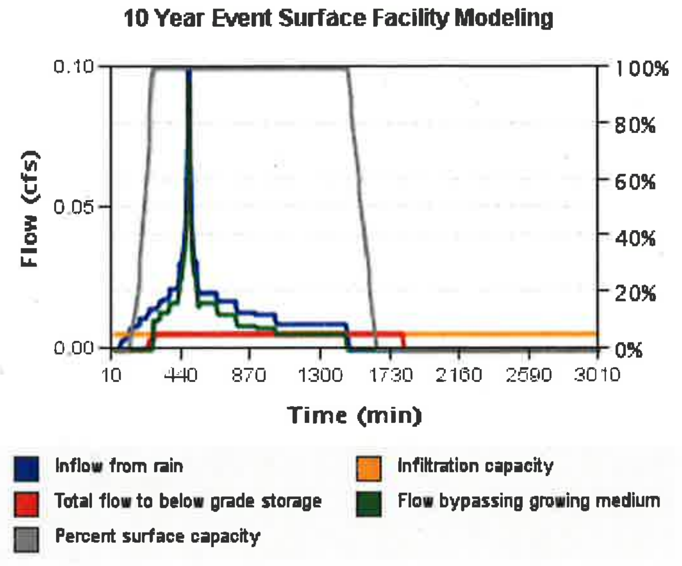


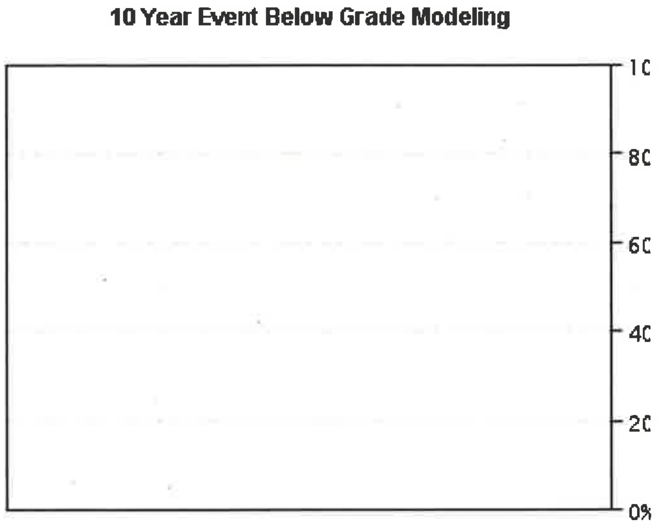
5 Year Event Surface Facility Modeling

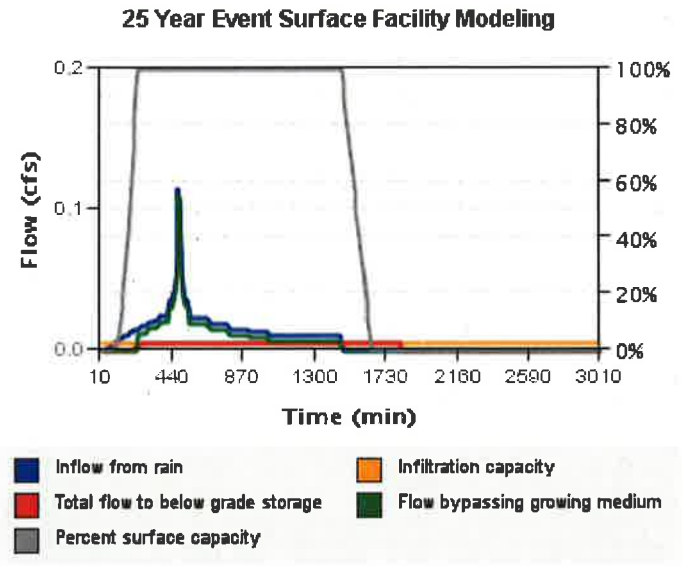


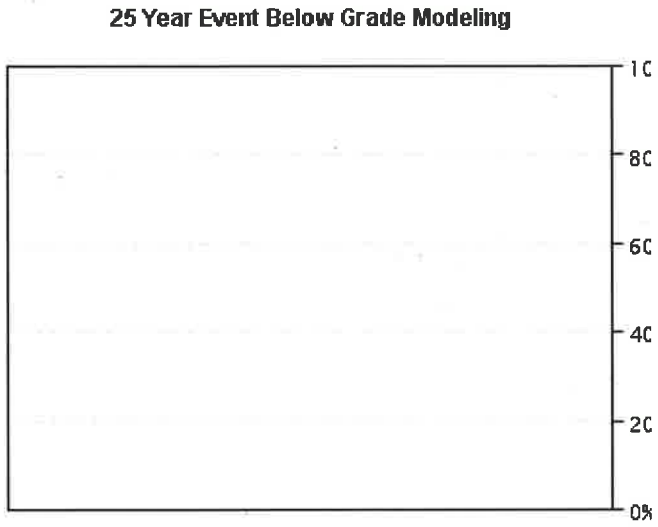
5 Year Event Below Grade Modeling




10 Year Event Surface Facility Modeling


10 Year Event Below Grade Modeling


25 Year Event Surface Facility Modeling

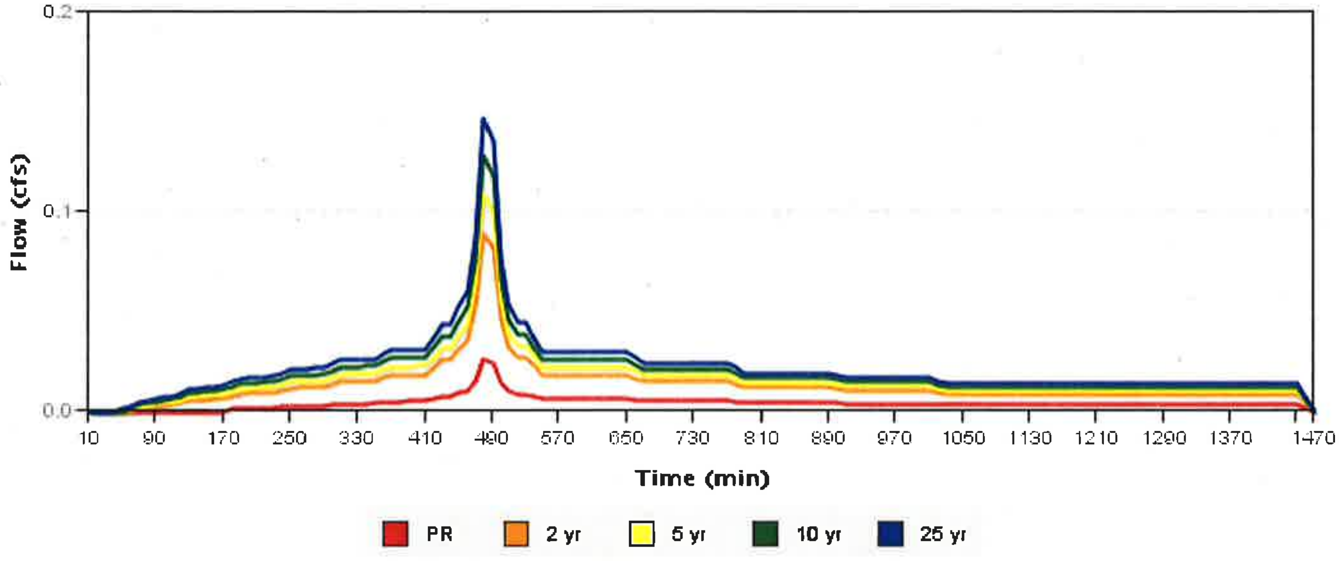

25 Year Event Below Grade Modeling

Catchment East 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	**B**
	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	**If discharging to an overland drainage system or to a storm sewer that discharges to an overland drainage system, including streams, drainageways, and ditches, the 2-year post-development peak flow must be equal or less than half of the 2-year pre-development rate and the 5, 10, and 25-year post-development peak rate must be equal or less than the pre-development rates for the corresponding design storms.**
	Impervious Area	**6270 sq ft** **0.144 acre**
	Time of Concentration (Tc)	**5**
	Pre-Development Curve Number (CN_{pre})	**95**
	Post-Development Curve Number (CN_{post})	**98**

⚠ Indicates value is outside of recommended range

SBUH Results



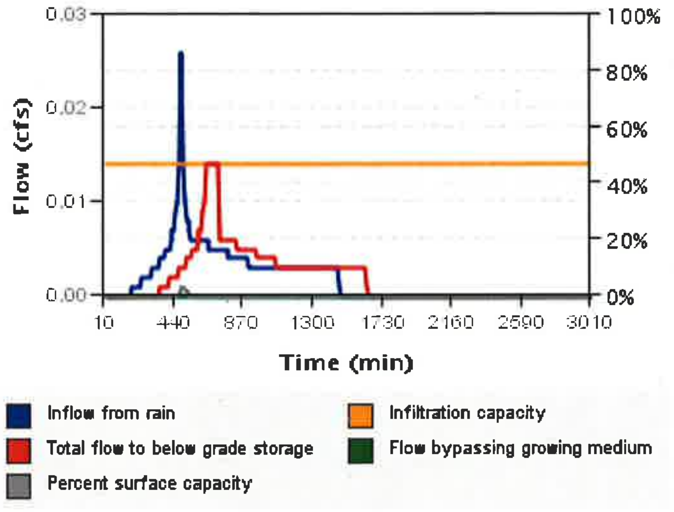
	Pre-Development Rate and Volume		Post-Development Rate and Volume	
	Peak Rate (cfs)	Volume (cf)	Peak Rate (cfs)	Volume (cf)
PR	0.015	219.367	0.026	327.625
2 yr	0.077	975.306	0.089	1134.531
5 yr	0.097	1228.832	0.108	1394.395
10 yr	0.118	1484.379	0.128	1654.648
25 yr	0.138	1741.246	0.147	1915.147

Facility East Roof

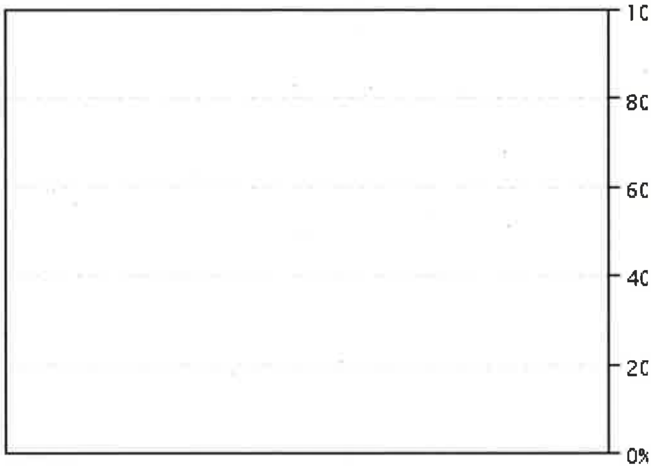
Facility Details	Facility Type	**Planter (Flat)**
	Facility Configuration	**D: Lined Facility with RS and Ud**
	Facility Shape	**Planter**
	Above Grade Storage Data	
	Bottom Area	**300 sq ft**
	Bottom Width	**5.00 ft**
	Storage Depth 1	**18.0 in**
	Growing Medium Depth	**18 in**
	Surface Capacity at Depth 1	**450.0 cu ft**
	Design Infiltration Rate for Native Soil	**0.000 in/hr**
	Infiltration Capacity	**0.014 cfs**
Facility Facts	Total Facility Area Including Freeboard	**300.00 sq ft**
	Sizing Ratio	**4.8%**
Pollution Reduction Results	Pollution Reduction Score	**Pass**
	Overflow Volume	**331.758 cf**
	Surface Capacity Used	**3%**
Flow Control Results	Flow Control Score	**Pass**
	Overflow Volume	**1651.338 cf**
	Surface Capacity Used	**100%**

	Post-development outflow (cfs)		**Pre-development inflow (cfs)**	
2 year	0.014	≤ ½ of	0.077	**Pass**
5 year	0.014	≤	0.097	**Pass**
10 year	0.026	≤	0.118	**Pass**
25 year	0.076	≤	0.138	**Pass**

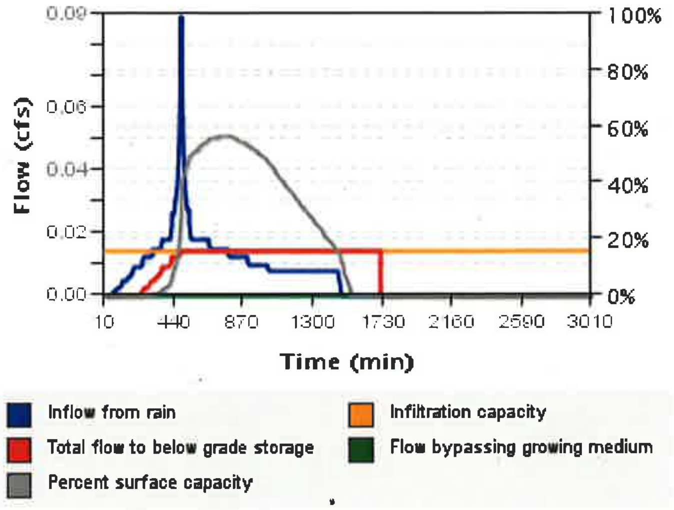
Pollution Reduction Event Surface Facility Modeling



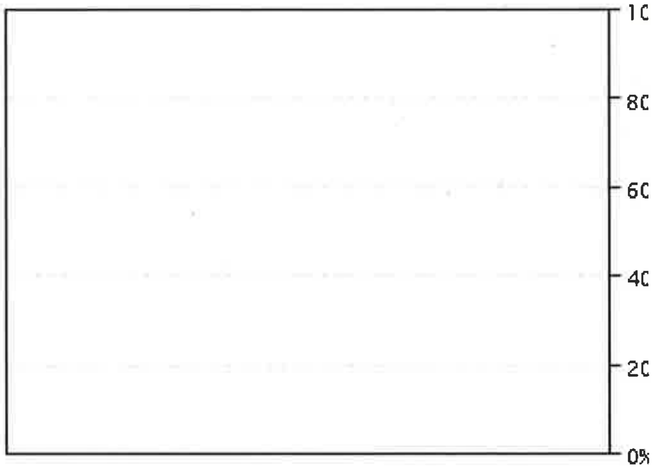
Pollution Reduction Event Below Grade Modeling



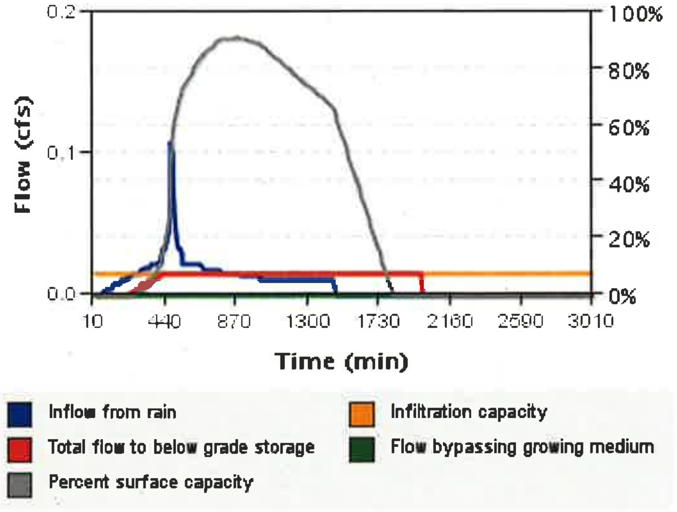
2 Year Event Surface Facility Modeling



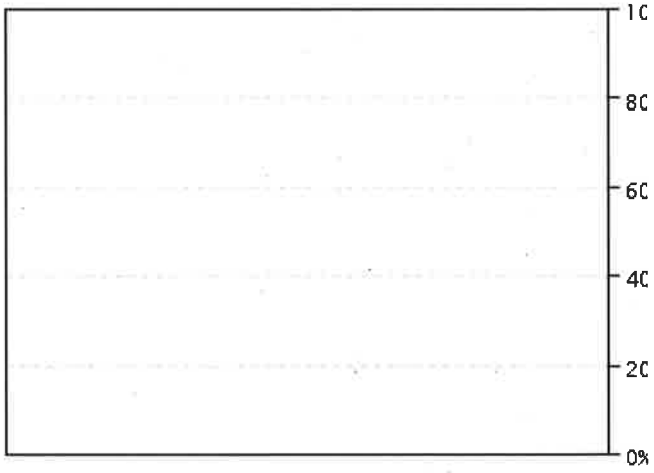
2 Year Event Below Grade Modeling



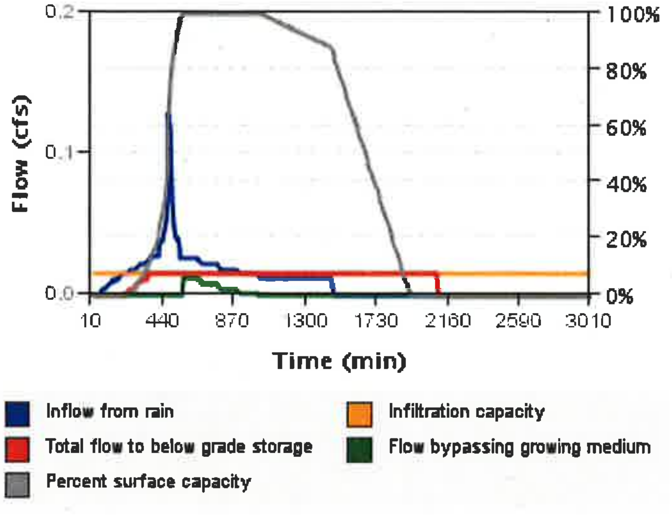
5 Year Event Surface Facility Modeling



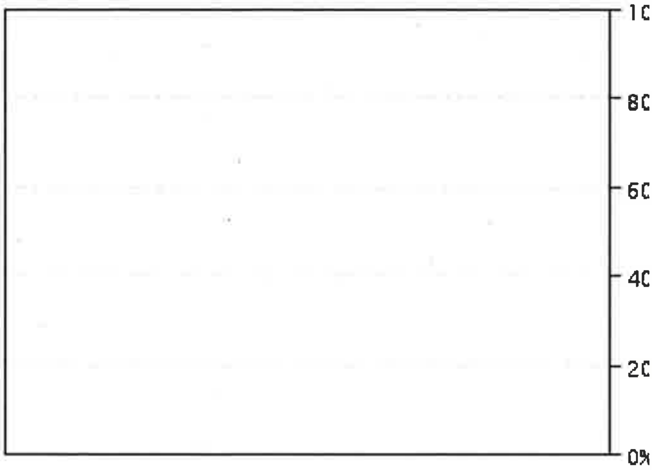
5 Year Event Below Grade Modeling



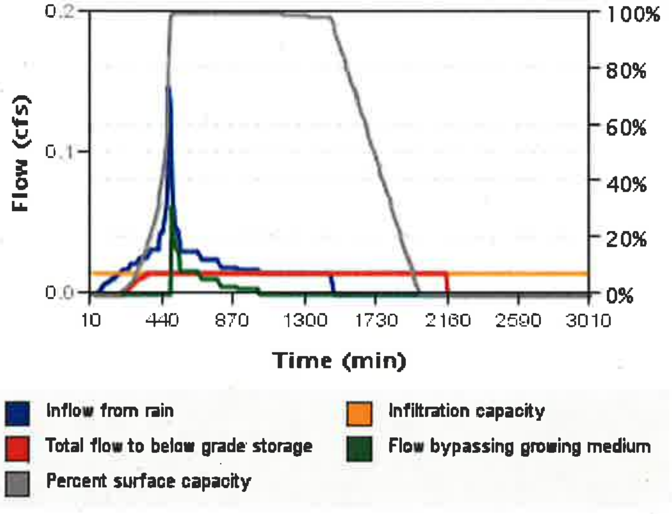
10 Year Event Surface Facility Modeling



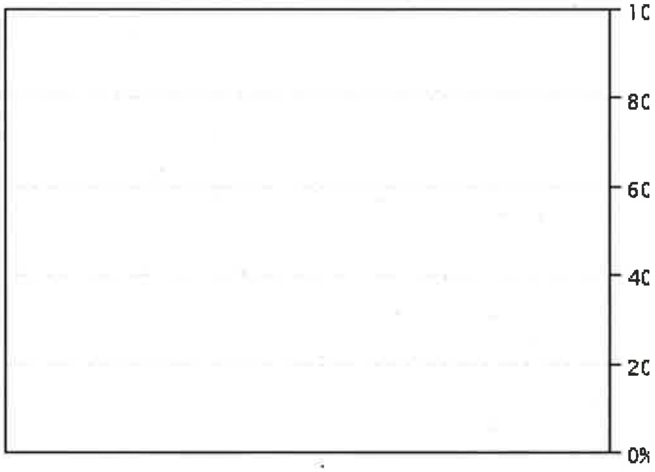
10 Year Event Below Grade Modeling



25 Year Event Surface Facility Modeling



25 Year Event Below Grade Modeling

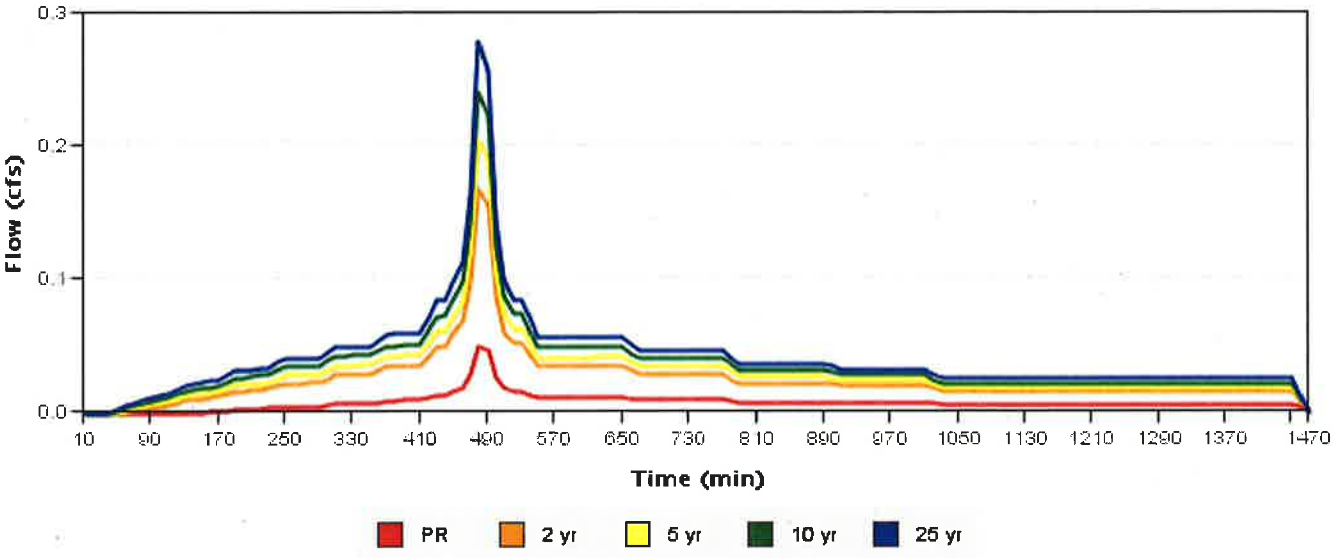


Catchment South Roof

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	**C**
	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	**The post-development peak rates for the 2, 5 and 10-year design storms must be equal or less than the pre-development rates.**
	Impervious Area	**11858 sq ft** **0.272 acre**
	Time of Concentration (Tc)	**5**
	Pre-Development Curve Number (CN_{pre})	**85**
	Post-Development Curve Number (CN_{post})	**98**

⚠ Indicates value is outside of recommended range

SBUH Results

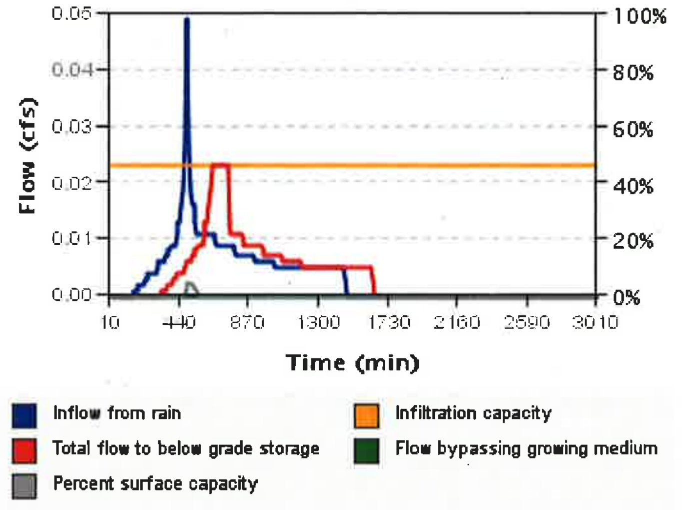


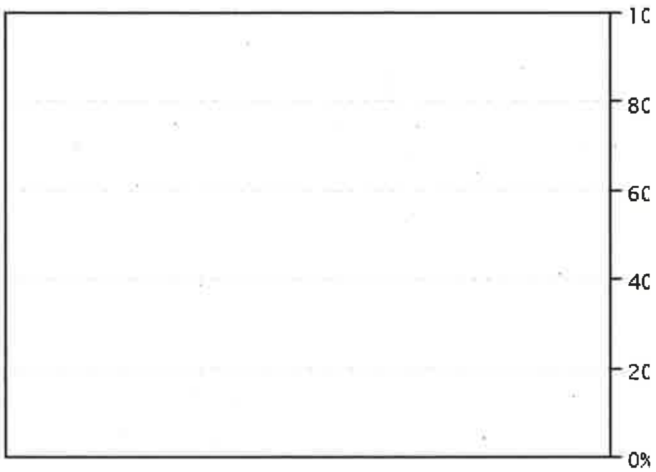
	Pre-Development Rate and Volume		Post-Development Rate and Volume	
	Peak Rate (cfs)	Volume (cf)	Peak Rate (cfs)	Volume (cf)
PR	0.002	100.319	0.049	619.614
2 yr	0.072	1086.337	0.167	2145.656
5 yr	0.105	1486.802	0.204	2637.119
10 yr	0.14	1906.723	0.241	3129.317
25 yr	0.176	2340.605	0.278	3621.979

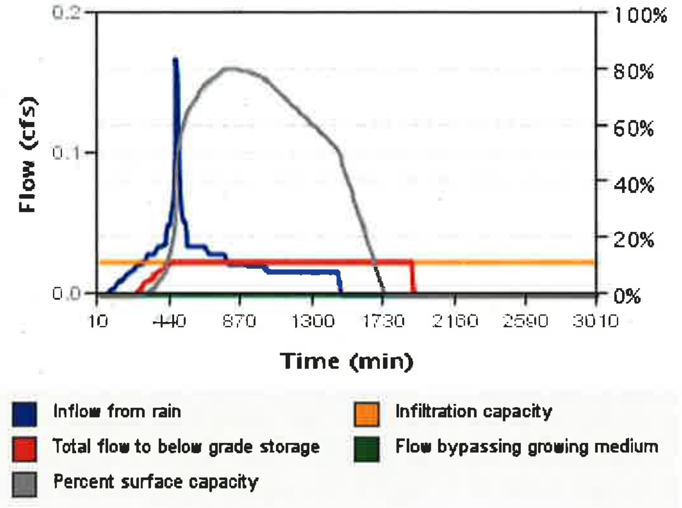
Facility South Roof

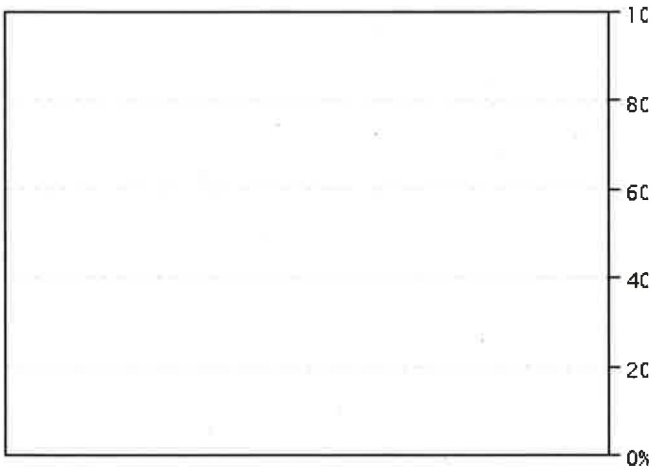
Facility Details	Facility Type	**Planter (Flat)**
	Facility Configuration	**D: Lined Facility with RS and Ud**
	Facility Shape	**Planter**
	Above Grade Storage Data	
	Bottom Area	**490 sq ft**
	Bottom Width	**5.00 ft**
	Storage Depth 1	**18.0 in**
	Growing Medium Depth	**18 in**
	Surface Capacity at Depth 1	**735.0 cu ft**
	Design Infiltration Rate for Native Soil	**0.000 in/hr**
	Infiltration Capacity	**0.023 cfs**
Facility Facts	Total Facility Area Including Freeboard	**490.00 sq ft**
	Sizing Ratio	**4.1%**
Pollution Reduction Results	Pollution Reduction Score	**Pass**
	Overflow Volume	**622.548 cf**
	Surface Capacity Used	**5%**
Flow Control Results	Flow Control Score	**Pass**
	Overflow Volume	**3144.533 cf**
	Surface Capacity Used	**100%**

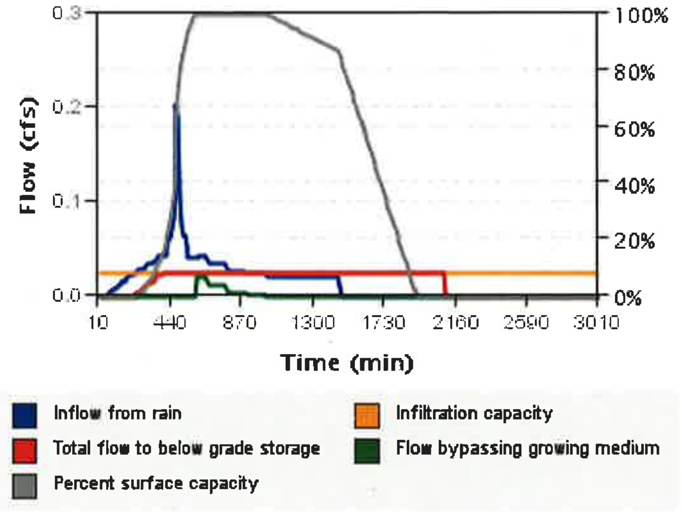
	Post-development outflow (cfs)		**Pre-development inflow (cfs)**	
2 year	0.023	≤	0.072	**Pass**
5 year	0.042	≤	0.105	**Pass**
10 year	0.124	≤	0.14	**Pass**

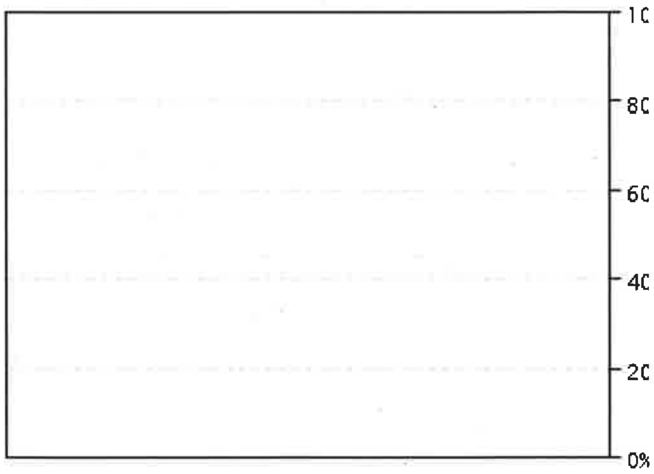

Pollution Reduction Event Surface Facility Modeling

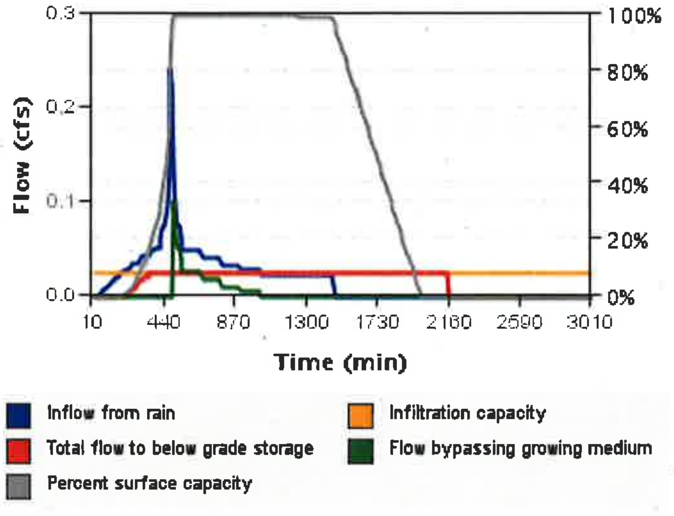

Pollution Reduction Event Below Grade Modeling

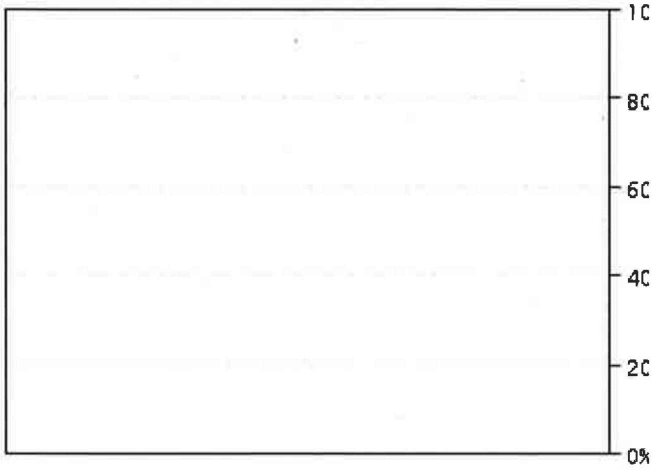

2 Year Event Surface Facility Modeling

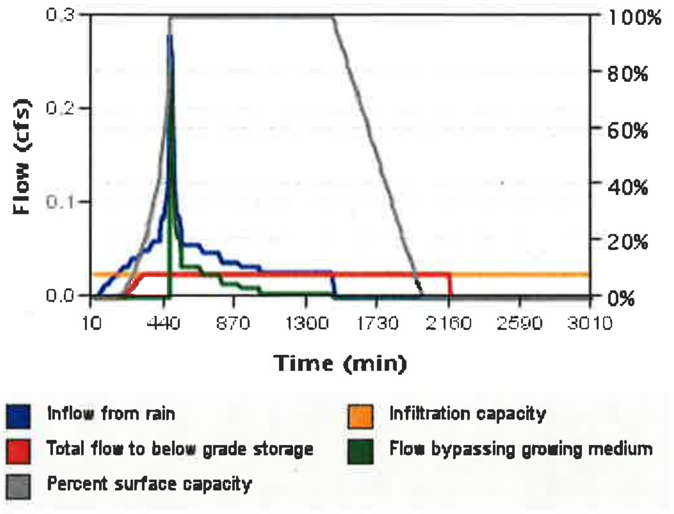

2 Year Event Below Grade Modeling

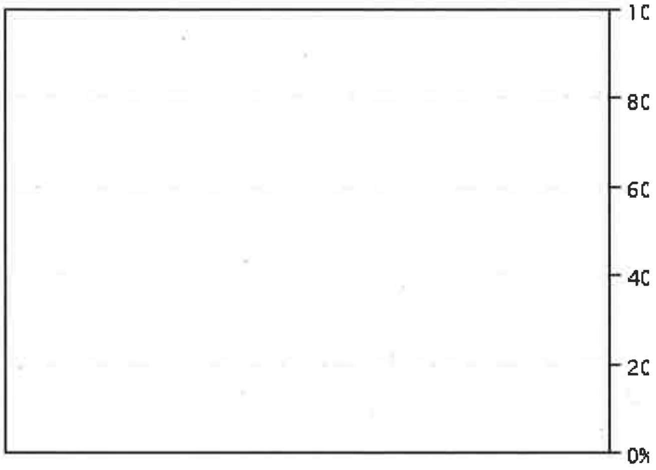

5 Year Event Surface Facility Modeling


5 Year Event Below Grade Modeling


10 Year Event Surface Facility Modeling


10 Year Event Below Grade Modeling


25 Year Event Surface Facility Modeling

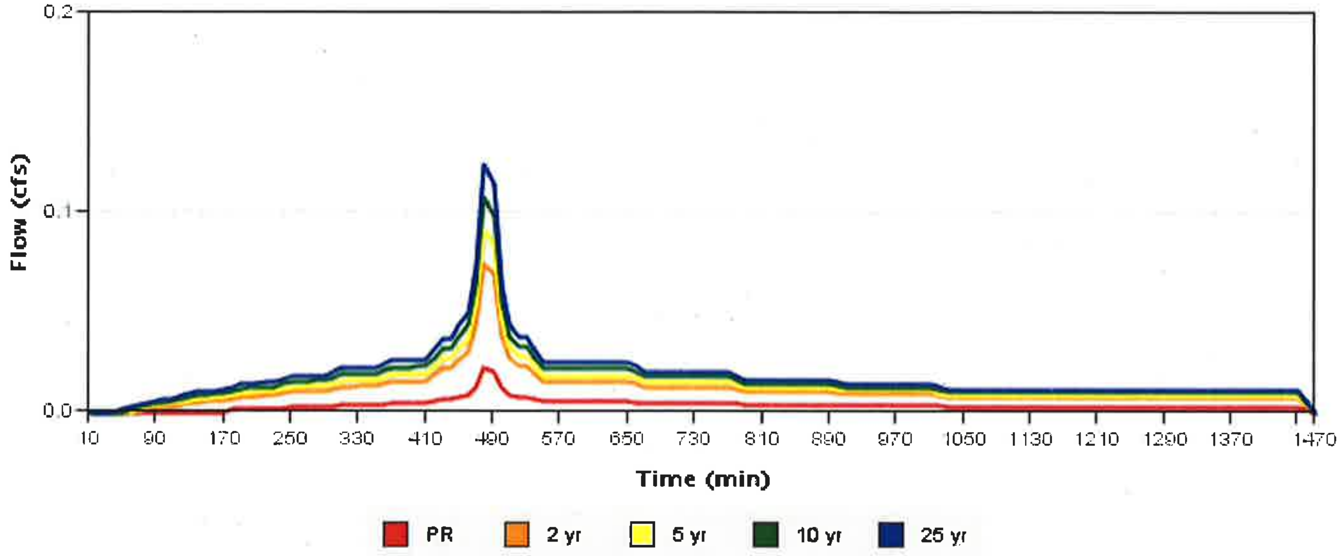

25 Year Event Below Grade Modeling

Catchment North 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	**C**
	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	**The post-development peak rates for the 2, 5 and 10-year design storms must be equal or less than the pre-development rates.**
	Impervious Area	**5266 sq ft** **0.121 acre**
	Time of Concentration (Tc)	**5**
	Pre-Development Curve Number (CN_{pre})	**98**
	Post-Development Curve Number (CN_{post})	**98**

⚠ Indicates value is outside of recommended range

SBUH Results



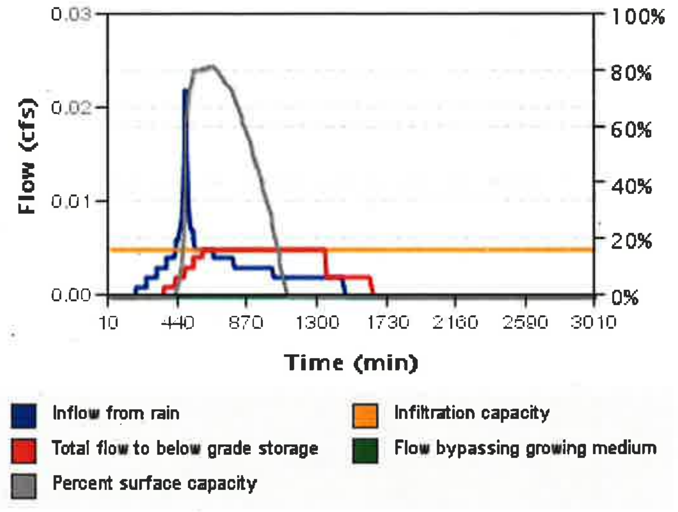
	Pre-Development Rate and Volume		Post-Development Rate and Volume	
	Peak Rate (cfs)	Volume (cf)	Peak Rate (cfs)	Volume (cf)
PR	0.022	275.163	0.022	275.163
2 yr	0.074	952.861	0.074	952.861
5 yr	0.091	1171.114	0.091	1171.114
10 yr	0.107	1389.693	0.107	1389.693
25 yr	0.124	1608.479	0.124	1608.479

Facility North Roof

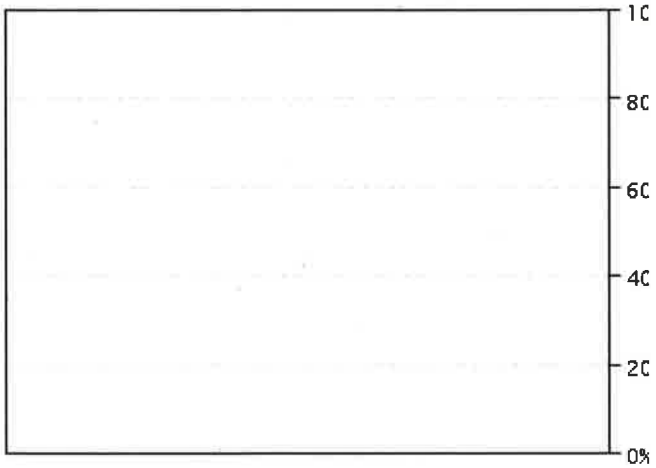
Facility Details	Facility Type	**Planter (Flat)**
	Facility Configuration	**D: Lined Facility with RS and Ud**
	Facility Shape	**Planter**
	Above Grade Storage Data	
	Bottom Area	**100 sq ft**
	Bottom Width	**10.00 ft**
	Storage Depth 1	**6.0 in**
	Growing Medium Depth	**18 in**
	Surface Capacity at Depth 1	**50.0 cu ft**
	Design Infiltration Rate for Native Soil	**0.000 in/hr**
	Infiltration Capacity	**0.005 cfs**
Facility Facts	Total Facility Area Including Freeboard	**100.00 sq ft**
	Sizing Ratio	**1.9%**
Pollution Reduction Results	Pollution Reduction Score	**Pass**
	Overflow Volume	**283.446 cf**
	Surface Capacity Used	**82%**
Flow Control Results	Flow Control Score	**Pass**
	Overflow Volume	**1391.158 cf**
	Surface Capacity Used	**100%**

	Post-development outflow (cfs)		**Pre-development inflow (cfs)**	
2 year	0.074	≤	0.074	**Pass**
5 year	0.091	≤	0.091	**Pass**
10 year	0.107	≤	0.107	**Pass**

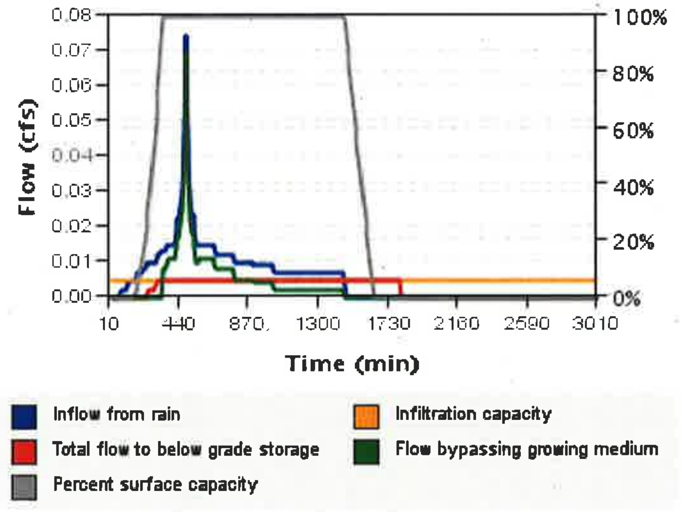
Pollution Reduction Event Surface Facility Modeling



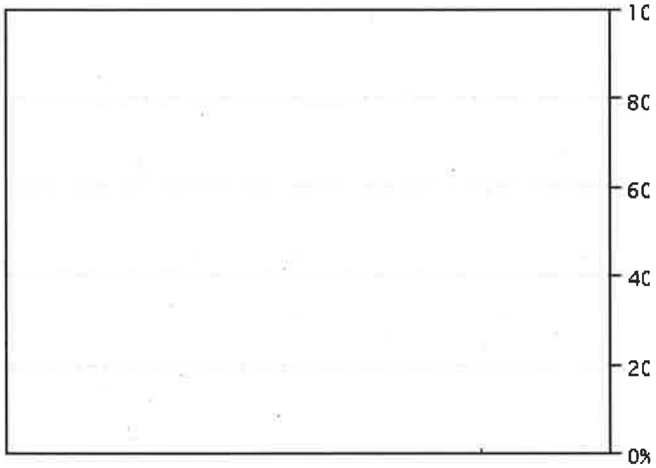
Pollution Reduction Event Below Grade Modeling



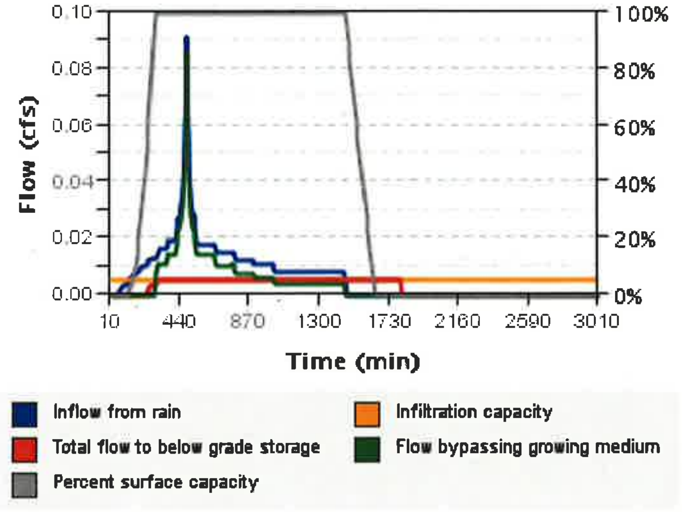
2 Year Event Surface Facility Modeling



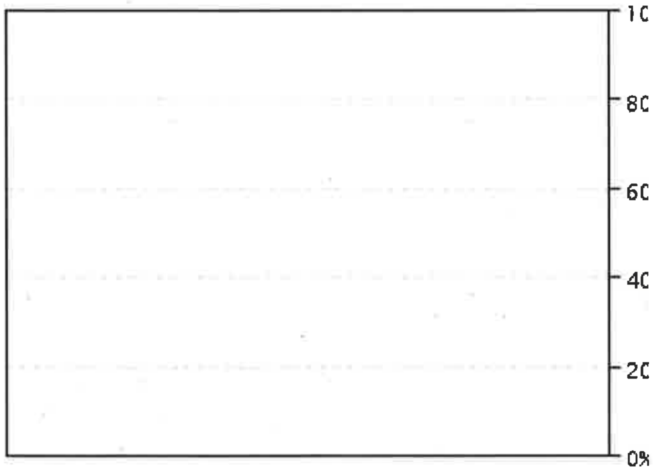
2 Year Event Below Grade Modeling



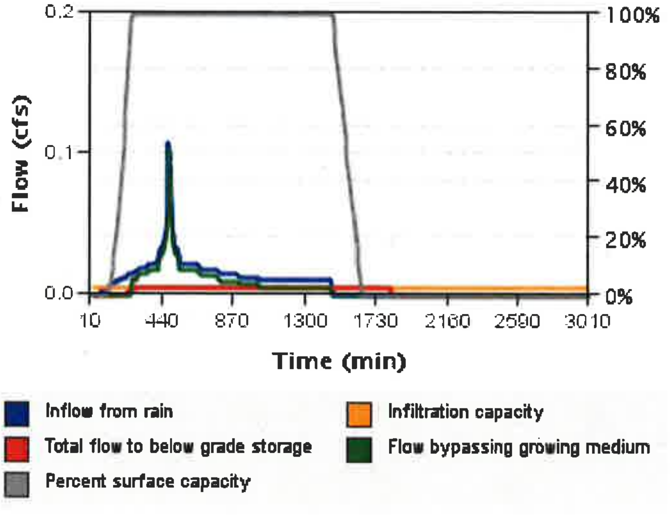
5 Year Event Surface Facility Modeling



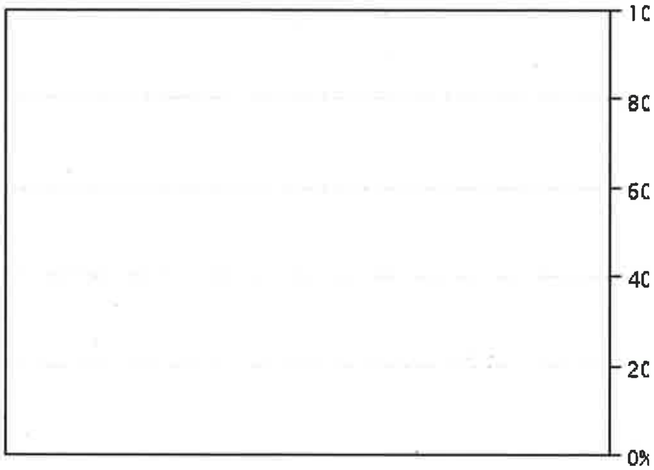
5 Year Event Below Grade Modeling



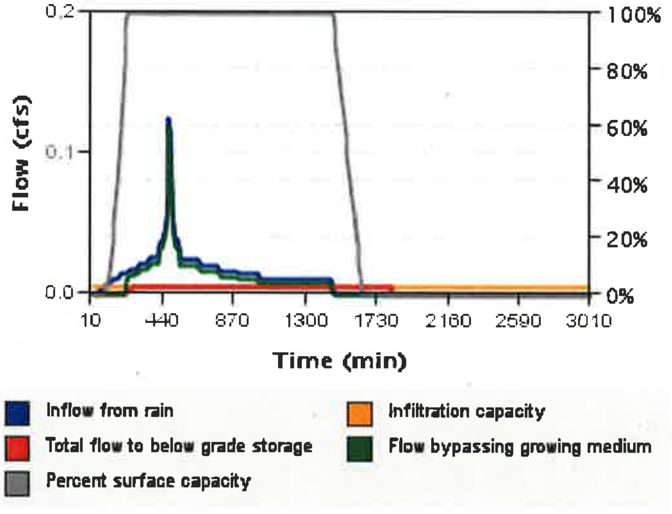
10 Year Event Surface Facility Modeling



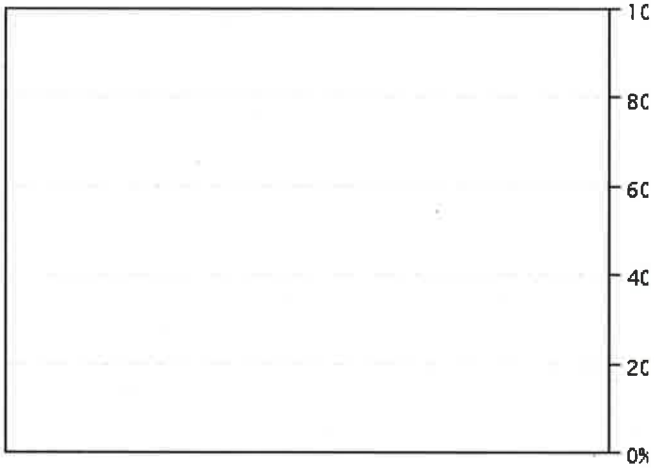
10 Year Event Below Grade Modeling



25 Year Event Surface Facility Modeling



25 Year Event Below Grade Modeling

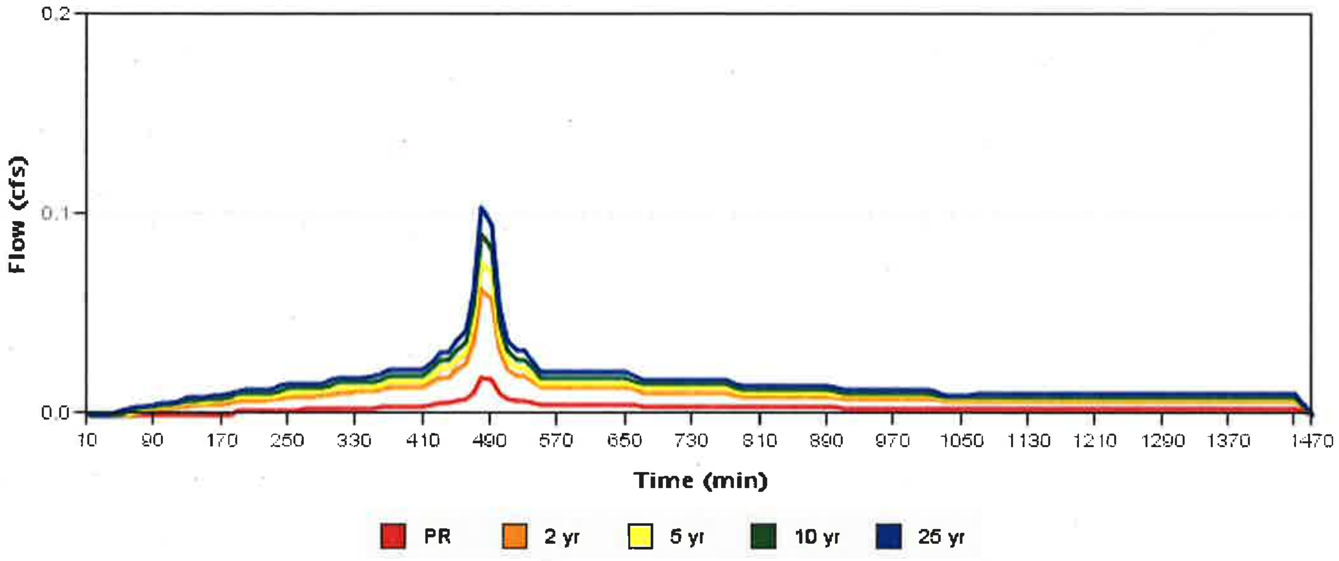


Catchment Center Parking Lot

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	**C**
	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	**The post-development peak rates for the 2, 5 and 10-year design storms must be equal or less than the pre-development rates.**
	Impervious Area	**4400 sq ft** **0.101 acre**
	Time of Concentration (Tc)	**5**
	Pre-Development Curve Number (CN_{pre})	**98**
	Post-Development Curve Number (CN_{post})	**98**

⚠ Indicates value is outside of recommended range

SBUH Results



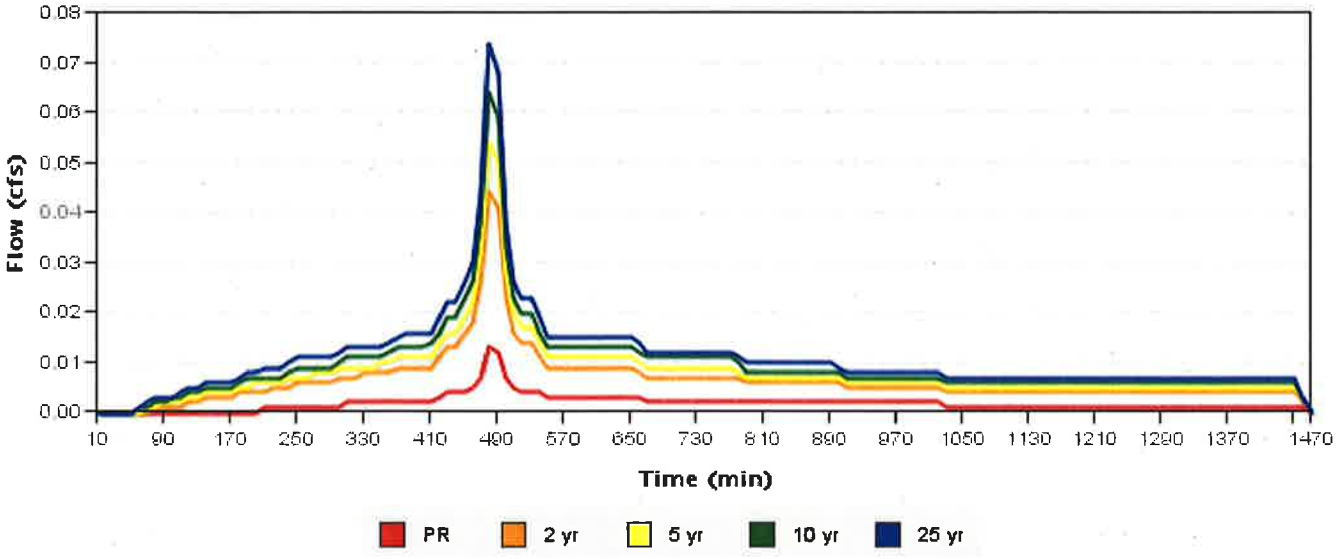
	Pre-Development Rate and Volume		Post-Development Rate and Volume	
	Peak Rate (cfs)	**Volume (cf)**	**Peak Rate (cfs)**	**Volume (cf)**
PR	0.018	229.912	0.018	229.912
2 yr	0.062	796.162	0.062	796.162
5 yr	0.076	978.523	0.076	978.523
10 yr	0.09	1161.157	0.09	1161.157
25 yr	0.103	1343.962	0.103	1343.962

Catchment South Parking Lot (New Planter)

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	**C**
	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	**The post-development peak rates for the 2, 5 and 10-year design storms must be equal or less than the pre-development rates.**
	Impervious Area	**3150 sq ft** **0.072 acre**
	Time of Concentration (Tc)	**5**
	Pre-Development Curve Number (CN_{pre})	**98**
	Post-Development Curve Number (CN_{post})	**98**

⚠ Indicates value is outside of recommended range

SBUH Results



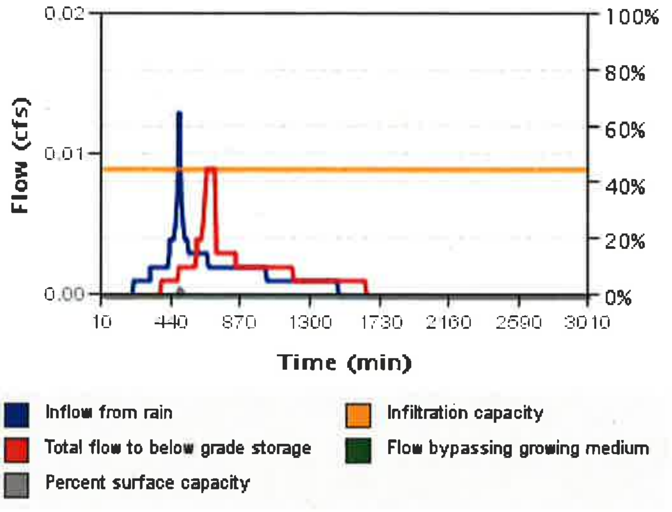
	Pre-Development Rate and Volume		Post-Development Rate and Volume	
	Peak Rate (cfs)	**Volume (cf)**	**Peak Rate (cfs)**	**Volume (cf)**
PR	0.013	164.596	0.013	164.596
2 yr	0.044	569.98	0.044	569.98
5 yr	0.054	700.533	0.054	700.533
10 yr	0.064	831.283	0.064	831.283
25 yr	0.074	962.155	0.074	962.155

Facility South Parking Lot (New Planter)

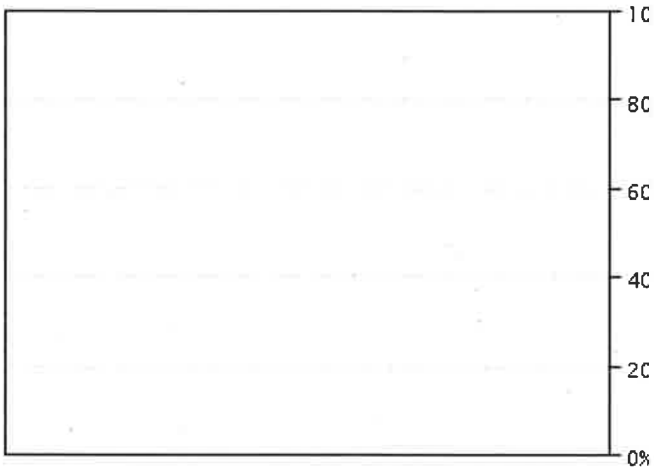
Facility Details	Facility Type	**Basin**
	Facility Configuration	**D: Lined Facility with RS and Ud**
	Facility Shape	**Amoeba**
	Above Grade Storage Data	
	Bottom Area	**51 sq ft**
	Bottom Perimeter Length	**41.00 ft**
	Side Slope	**3.0:1**
	Storage Depth 1	**18.0 in**
	Growing Medium Depth	**18 in**
	Freeboard Depth	**2.00 in**
	Surface Capacity at Depth 1	**168.8 cu ft**
	Design Infiltration Rate for Native Soil	**0.000 in/hr**
	Infiltration Capacity	**0.009 cfs**
Facility Facts	Total Facility Area Including Freeboard	**256.00 sq ft**
	Sizing Ratio	**8.1%**
Pollution Reduction Results	Pollution Reduction Score	**Pass**
	Overflow Volume	**168.826 cf**
	Surface Capacity Used	**3%**
Flow Control Results	Flow Control Score	**Pass**
	Overflow Volume	**834.416 cf**
	Surface Capacity Used	**100%**

	Post-development outflow (cfs)		**Pre-development inflow (cfs)**	
2 year	0.009	≤	0.044	**Pass**
5 year	0.009	≤	0.054	**Pass**
10 year	0.02	≤	0.064	**Pass**

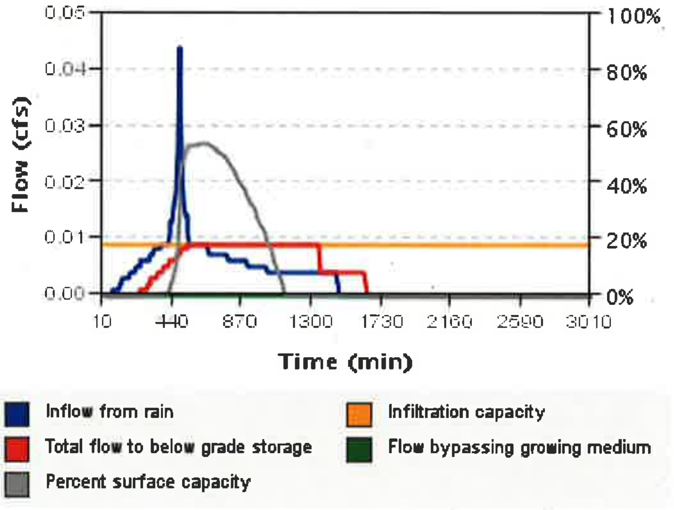
Pollution Reduction Event Surface Facility Modeling



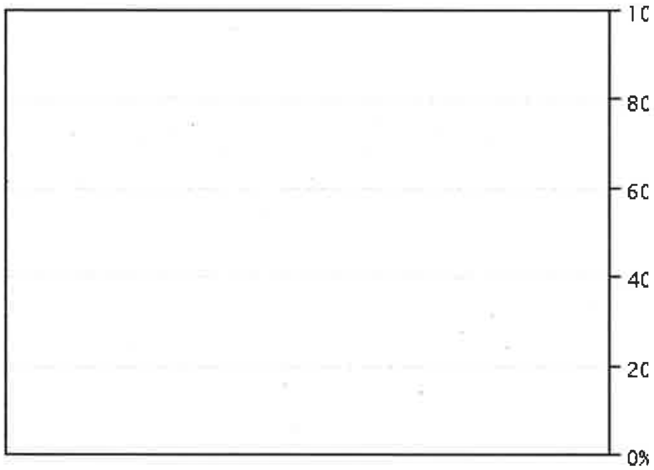
Pollution Reduction Event Below Grade Modeling



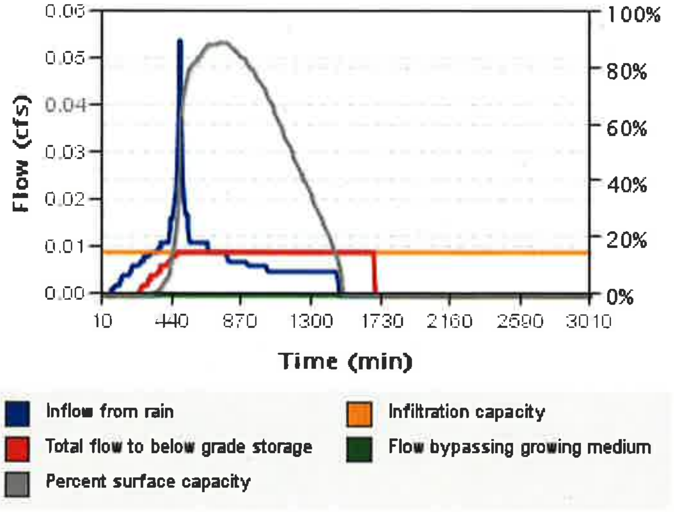
2 Year Event Surface Facility Modeling



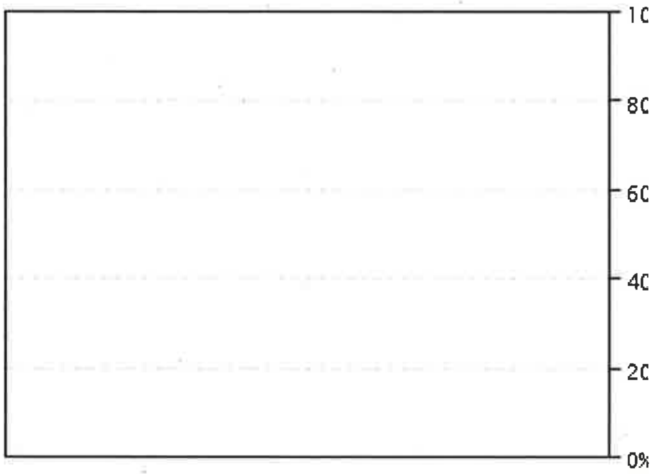
2 Year Event Below Grade Modeling



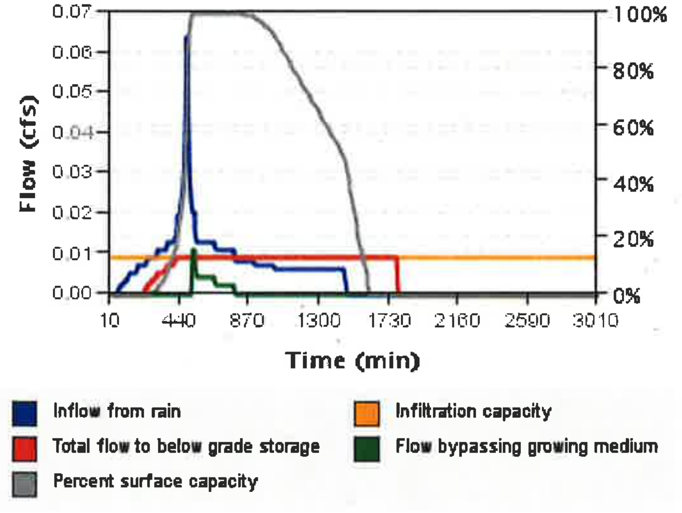
5 Year Event Surface Facility Modeling



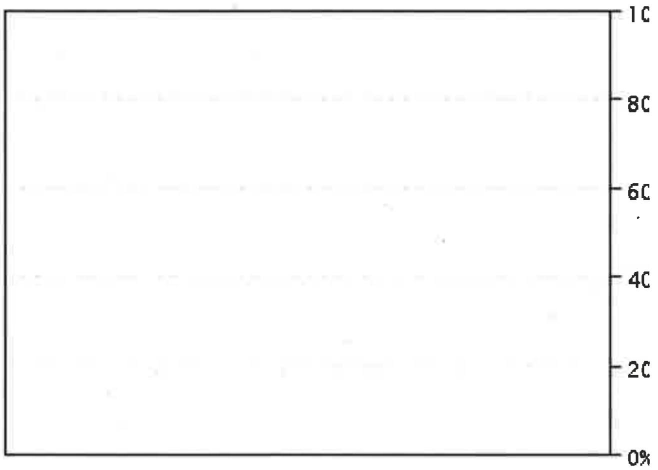
5 Year Event Below Grade Modeling



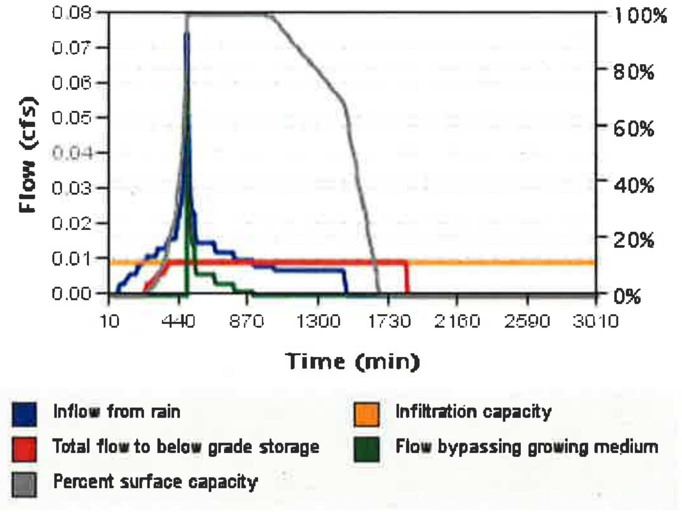
10 Year Event Surface Facility Modeling



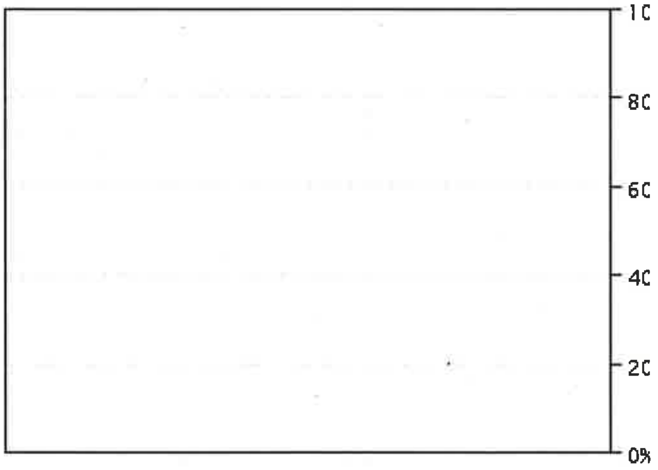
10 Year Event Below Grade Modeling



25 Year Event Surface Facility Modeling



25 Year Event Below Grade Modeling



GEOTECHNICAL REPORT

GeoDesign INC

REPORT OF GEOTECHNICAL ENGINEERING SERVICES

Ledding Library of Milwaukie Renovation and Expansion
10660 SE 21st Avenue
Milwaukie, Oregon

For
City of Milwaukie
August 25, 2017

GeoDesign Project: CMilwaukie-2-01

GEODESIGN INC

August 25, 2017

PlanB Consultancy
696 McVey Avenue
Lake Oswego, OR 97034

Attention: Amy Winterowd

Report of Geotechnical Engineering Services
Ledding Library of Milwaukie Renovation and Expansion
10660 SE 21st Avenue
Milwaukie, Oregon
GeoDesign Project: CMilwaukie-2-01

GeoDesign, Inc. is pleased to submit THIS report of geotechnical engineering services for the proposed renovation and expansion of the Ledding Library of Milwaukie located at 10660 SE 21st Avenue in Milwaukie, Oregon. Our services for this project were conducted in accordance with our proposal dated March 24, 2017.

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

Sincerely,

GeoDesign, Inc.

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

cc: Jordan Henderson, PlanB Consultancy (via email only)

JTW:BAS:kt
Attachments
One copy submitted (via email only)
Document ID: CMilwaukie-2-01-082517-geor.docx
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EXECUTIVE SUMMARY

The following is a summary of our findings and recommendations for design and construction of the proposed library renovation and expansion. 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.

- Based on the assumed foundation loads, the proposed structures can be supported on shallow foundations bearing on granular pads constructed on firm native soil or soil compacted as structural fill as presented in the "Shallow Foundations" section.
- The on-site soils 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 soils currently is above optimum and drying will be required if used as structural fill.
- The on-site soils will provide inadequate support for construction equipment during periods 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.
- Based on our explorations, the near-surface soils at the site generally consist of fine-grained silt and clay. Based on our infiltration testing, the site has little to no infiltration capacity.
- The soils encountered during our subsurface explorations are not susceptible to liquefaction under design levels of ground shaking

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1.0 INTRODUCTION

GeoDesign, Inc. is pleased to submit this geotechnical engineering report for the proposed renovation and expansion of the Ledding Library of Milwaukie located at 10660 SE 21st Avenue in Milwaukie, Oregon. Figure 1 shows the site relative to existing topographic and physical features. Figure 2 shows the approximate site boundaries and our approximate exploration locations.

The exploration logs and laboratory testing results are presented in Appendix A. Our site-specific seismic evaluation is presented in Appendix B. Acronyms and abbreviations used herein are defined at the end of this document.

1.1 PROJECT UNDERSTANDING

The site encompasses Tax Lot 11E36BB011800, Parcel Number 00026803. The parcel is currently developed with the existing Ledding Library building and includes an AC-paved parking area and landscaped areas with walkways. We understand that plans are preliminary and currently being developed; however, they may consist of expansion of the library into the existing parking areas and/or landscaped areas. In addition, development plans will also include renovations to the existing library building.

Based on preliminary information provided by ABHT Structural Engineers, isolated column loads are anticipated to be between 150 and 200 kips and continuous wall loads are anticipated to be between 3 and 6 kips per linear foot. We anticipate maximum floor loads will be 100 psf. The building addition will be classified as a special occupancy structure and will require a site-specific seismic evaluation per the current SOSSC.

2.0 SCOPE OF SERVICES

The purpose of our geotechnical engineering services was to characterize site subsurface conditions and provide geotechnical engineering recommendations for use in design and construction of the proposed development. Our scope of work is presented as follows:

- Reviewed readily available published geologic data and our in-house files for existing information on subsurface conditions in the site vicinity.
- Explored subsurface conditions by drilling five borings to depths ranging between 8.0 and 16.5 feet BGS.
- Classified the materials encountered in the explorations, and maintained a detailed log of each exploration.
- Completed laboratory testing on disturbed soil samples collected from the explorations as follows:
    - Twenty-one moisture content determinations in general accordance with ASTM D 2216
    - Four particle-size determinations in general accordance with ASTM C 117 and ASTM D 1140
    - One Atterberg limits tests in general accordance with ASTM D 4318

- Provided recommendations for site preparation and grading, including clearing and grubbing, demolition, temporary and permanent slopes, fill placement criteria, suitability of on-site soil for fill, subgrade preparation, and recommendations for wet weather construction.
- Provided foundation support recommendations for the proposed building addition. Our recommendations include preferred foundation type, allowable bearing capacity, and lateral resistance parameters.
- 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.
- Provided pavement design recommendations for AC paving, including subbase, base course, and AC paving thickness.
- Provided recommendations for seismic design factors in accordance with the procedures outlined in the 2012 IBC and 2014 SOSSC.
- Conducted a site-specific seismic hazard evaluation as required for the public "occupied structure" in accordance with procedures in the 2014 SOSSC.
- Prepared this geotechnical engineering report that presents our findings, conclusions, and recommendations.

3.0 SITE CONDITIONS

3.1 SURFACE CONDITIONS

The approximately 1.8-acre property is currently developed with the existing Ledding Library building and includes an AC-paved parked area and landscaped areas with walkways. The building expansion will likely extend to the south of the existing structure into the landscape area or north into the existing parking lot. The site is relatively level with grade changes between approximately 42 and 47 feet MSL.

3.2 SUBSURFACE CONDITIONS

3.2.1 General

Our subsurface exploration program consisted of drilling five borings (B-1 through B-5) to depths ranging between 8.0 and 16.5 feet BGS. Borings B-1 through B-3 were drilled in the AC parking lot and B-4 and B-5 were drilled in existing landscape areas. Drilling refusal was encountered in all borings on the underlying gravel and silty gravel. We conducted infiltration testing in B-5 at a depth of 6.0 feet BGS. The approximate locations of the explorations are shown on Figure 2. A more detailed description of the exploration and laboratory testing programs, the exploration logs, and results of our laboratory testing are presented in Appendix A.

Subsurface conditions generally consist of silt and clay, over silty sand and sand with interbeds of silt, overlying medium dense to dense gravel. The following sections provide a more detailed description of the units encountered.

3.2.2 Pavement Section

Borings B-1 through B-3 were completed in the existing AC-paved parking lot. The AC varied from 3.0 to 6.0 inches thick and the aggregate base was observed to be 7.0 to 11.0 inches thick. Table 1 presents the thickness of the AC and aggregate base encountered at the boring locations.

Table 1. Existing Pavement Thicknesses

Boring	AC Thickness (inches)	Base Thickness (inches)
B-1	3.0	11.0
B-2	6.0	7.0
B-3	3.0	9.0

3.2.3 Silt and Clay

Below the AC and aggregate base and from the surface in B-4 we encountered brown to gray medium stiff to stiff silt and clay with trace to minor amounts of sand to depths ranging between 8.0 and 9.5 feet BGS in B-1 through B-4. A layer of very stiff silt was also observed between depths of 11.0 and 14.0 feet BGS in B-4. Laboratory analysis of the silt and clay indicates the moisture content ranged between 19 and 39 percent at the time of testing.

3.2.4 Sand

Loose to medium dense, brown silty sand and sand with silt was observed at depths ranging between 8.0 and 13.0 feet BGS below the silt and from the ground surface to a depth of 6.5 feet BGS in B-5. Interbedded layers of silt were observed throughout the silty sand and sand with silt. Laboratory analysis of the silty sand and sand with silt indicates the moisture content ranged from 14 to 39 percent at the time of testing.

3.2.5 Gravel

We encountered medium dense, brown to gray, silty gravel to gravel with sand starting at depths ranging between 6.5 and 14.0 feet BGS and extending to the maximum depth explored of 16.5 feet BGS. Laboratory testing indicates the moisture content ranged from 12 to 19 percent at the time of testing.

3.3 GROUNDWATER

Groundwater was observed in the three deeper borings during drilling. The depths to the observed groundwater are summarized in Table 2.

Table 2. Groundwater Measurements

Boring	Depth (feet BGS)
B-1	13.0
B-3	14.3
B-4	13.3

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. In addition, we expect the depth to groundwater may be associated with the water level of the pond and Spring Creek located along the east side of the property.

3.4 INFILTRATION TESTING

Infiltration testing was completed to assist in the evaluation of potential stormwater infiltration facilities for the project. We conducted one infiltration test in B-5 at a depth of 6.0 feet BGS. The infiltration test was performed using the encased falling head method using a 6-inch-inside diameter casing and approximately 12 inches of water head. Laboratory testing was performed to determine the percent fines content at the infiltration test depth. Table 3 summarizes the unfactored infiltration test results and the amount of fines present at the depth of the infiltration test.

Table 3. Infiltration Test Results

Boring	Depth (feet BGS)	Material	Observed Infiltration Rate[1] (inches per hour)	Percent Fines[2]
B-5	6.0	Sand with Silt	0.3	27

1. Infiltration rates are measured rates with no factor of safety.
2. Fines content: material passing the U.S. Standard No. 200 sieve

Given the infiltration test results, fine-grained soils present across the site, relatively shallow groundwater, and without additional testing, it is our opinion that the site has little to no infiltration capacity.

4.0 CONCLUSIONS

Based on the results of our subsurface explorations and engineering analyses, it is our opinion that the site can be developed as proposed. The primary geotechnical considerations for the project are summarized in the "Executive Summary." Our specific recommendations are provided in the following sections.

5.0 DESIGN

5.1 GENERAL

The following sections provide our design recommendations for the project. All site preparation and structural fill should be prepared as recommended in the "Construction" section.

5.2 SHALLOW FOUNDATIONS

5.2.1 General

Based on the results of our explorations and analysis, the proposed library addition can be supported by conventional spread footings resting on granular pads underlain by undisturbed native soil or structural fill overlying firm native soil. Foundations should not be established on undocumented fill, soft soil, or soil containing deleterious material. If present, this material should be removed and replaced with granular pads.

The granular pads should be a minimum of 4 inches thick, increasing to a minimum of 6 inches thick during the wet winter months, and extend 6 inches beyond the margins of the footings for every foot excavated below the base grade of the footing. The granular pads should consist of imported granular material, as defined in the "Structural Fill" section. The imported granular material should be compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D 1557, or, as determined by one of our geotechnical staff, until well-keyed. We recommend that a member of our geotechnical staff observe the prepared footing subgrade and the prepared granular pad.

5.2.2 Dimensions and Capacities

Continuous wall and isolated spread footings should be at least 18 and 24 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.

Footings bearing on subgrade prepared as recommended above should be sized based on an allowable bearing pressure of 2,500 psf. This is a net bearing pressure; the weight of the footing and overlying backfill can be ignored in calculating footing sizes. The recommended allowable bearing pressure applies to the total of dead plus long-term live loads and may be doubled for short-term loads such as those resulting from wind or seismic forces.

5.2.3 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 native soil and structural fill is 250 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. The passive resistance should be reduced to 120 pcf below groundwater.

For footings in contact with native soil, a coefficient of friction equal to 0.30 may be used when calculating resistance to sliding. For footings in contact with granular fill, a coefficient of friction equal to 0.40 may be used when calculating resistance to sliding.

5.2.4 Settlement

Based on the anticipated foundation loads, post-construction settlement of footings and floor slabs founded as recommended is anticipated to be less than 1 inch. Differential settlements between similarly loaded, newly constructed foundation elements should be approximately one-half of the total settlement. Differential settlement between new and existing foundation elements that are structurally tied together will likely be negligible and approaching the total settlement if structurally isolated.

5.2.5 Subgrade Observation

All footing and floor subgrades should be evaluated by a representative of GeoDesign to evaluate the bearing conditions. Observations should also confirm that all loose or soft material, organics, unsuitable fill, prior topsoil zones, and softened subgrades (if present) have been removed. Localized deepening of footing excavations may be required to penetrate deleterious material.

5.3 FLOOR SLABS

Satisfactory subgrade support for building floor slabs supporting up to 100 psf areal loading can be obtained on the existing undisturbed native silt and clay or on structural fill. To help reduce moisture transmission and slab shifting, we recommend a minimum 6-inch-thick layer of floor slab base rock be placed and compacted over a subgrade that has been prepared in conformance with the "Site Preparation" section. The floor slab base rock should meet the requirements in the "Materials" section and be compacted to at least 95 percent of ASTM D 1557.

While groundwater is unlikely to be encountered within the slab subgrade material, the native soil is fine grained and will tend to maintain a high moisture content. In areas where moisture-sensitive floor slab and flooring will be installed, the installation of a vapor barrier is warranted in order to reduce the potential for moisture transmission through and efflorescence growth on the slab and flooring. In addition, flooring manufacturers often require vapor barriers to protect flooring and flooring adhesives and they will warrant their product only if a vapor barrier is installed according to their recommendations.

Slabs should be reinforced according to their proposed use and per the structural engineer's recommendations. Load-bearing concrete slabs may be designed assuming a modulus of subgrade reaction, k, of 150 psi per inch.

5.4 RETAINING STRUCTURES

5.4.1 Assumptions

Retaining walls may be needed to address grade changes. Our retaining wall design recommendations are based on the following assumptions: (1) the walls consist of conventional, cantilevered retaining walls, (2) the walls are less than 8 feet in height, (3) the backfill is drained, and (4) the backfill has a slope flatter than 4H:1V. Re-evaluation of our recommendations will be required if the retaining wall design criteria for the project varies from these assumptions.

5.4.2 Wall Design Parameters

For unrestrained retaining walls, an active pressure of 35 pcf equivalent fluid pressure should be used for design. For embedded building walls, a superimposed seismic lateral force should be calculated based on a dynamic force of $7.0H^2$ pounds per lineal foot of wall, where H is the height of the wall in feet, and applied a distance of 0.6H from the base of the wall. Where retaining walls are restrained from rotation prior to being backfilled, a pressure of 55 pcf equivalent fluid pressure should be used for design.

If surcharges (e.g., retained slopes, building foundations, vehicles, steep slopes, terraced walls, etc.) are located within a horizontal distance from the back of a wall equal to twice the height of the wall, additional pressures will need to be accounted for in the wall design. Our office should be contacted for appropriate wall surcharges based on the actual magnitude and configuration of the applied loads.

The base of the wall footing excavations should extend a minimum of 18 inches below lowest adjacent grade. The footing excavations should then be lined with a minimum 4-inch-thick layer of compacted imported granular material, as described in the "Materials" section.

The wall footings should be designed in accordance with the guidelines provided in the appropriate portion of the "Shallow Foundations" section.

5.4.3 Wall Drainage and Backfill

The above design parameters have been provided assuming that back-of-wall drains will be installed to prevent buildup of hydrostatic pressures behind all walls. If a drainage system is not installed, our office should be contacted for revised design forces.

The backfill material placed behind the walls and extending a horizontal distance of ½H, where H is the height of the retaining wall, should consist of retaining wall select backfill placed and compacted in conformance with the "Structural Fill" section.

A minimum 6-inch-diameter, perforated collector pipe should be placed at the base of the walls. The pipe should be embedded in a minimum 2-foot-wide zone of angular drain rock that is wrapped in a drainage geotextile fabric and extends up the back of the wall to within 1 foot of the finished grade. The drain rock and drainage geotextile fabric should meet specifications provided in the "Materials" section. The perforated collector pipes should discharge at an appropriate location away from the base of the wall. The discharge pipe(s) should not be tied directly into stormwater drain systems, unless measures are taken to prevent backflow into the drainage system of the wall.

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

5.5 SEISMIC DESIGN CONSIDERATIONS

5.5.1 IBC Parameters

Based on our explorations, the following design parameters can be applied if the building is designed using the applicable provisions of the 2012 IBC and 2014 SOSSC. The parameters in Table 4 are appropriate for code-level seismic design obtained from USGS seismic design maps (USGS, 2014). We performed a site-specific seismic evaluation study, the results of this study are presented in Appendix B.

Table 4. IBC Seismic Design Parameters

Seismic Design Parameter	Short Period (T_s = 0.2 second)	1 Second Period (T_1 = 1.0 second)
MCE Spectral Acceleration, S	S_s = 0.984 g	S_1 = 0.421 g
Site Class	D	
Site Coefficient, F	F_a = 1.11	F_v = 1.58
Adjusted Spectral Acceleration, S_M	S_{MS} = 1.088 g	S_{M1} = 0.665 g
Design Spectral Response Acceleration Parameters, S_D	0.726 g	0.443 g

5.6 PAVEMENTS

5.6.1 Design Assumptions and Parameters

We anticipate some re-grading and re-paving may be needed to accommodate the building addition and site improvements. Pavements should be installed on undisturbed native subgrade, scarified and re-compacted soil, or new engineered fills described in the "Site Preparation" and "Structural Fill" sections.

Our pavement recommendations are based on the following assumptions:

- The top 12 inches of soil subgrade is compacted to at least 92 percent of its maximum dry density, as determined by ASTM D 1557, or until proof rolling with heavy equipment indicates that is it firm and unyielding.
- Resilient moduli of 3,700 psi and 20,000 psi were assumed for the subgrade and base rock, respectively.
- No traffic growth.
- A pavement design life of 20 years.
- Initial and terminal serviceability indices of 4.2 and 2.5, respectively.
- Reliability of 75 percent and standard deviation of 0.49.

We do not have specific information on the frequency of vehicles expected at the site. Consequently, we have provided pavement sections for automobile parking and heavy-duty areas with high automobile traffic and occasional heavy vehicles (i.e., garbage trucks, delivery trucks, semi-trucks, etc.). The breakdown of the type and frequency of the trucks used in our analysis are presented in Table 5. If any of these assumptions vary from project design values, our office should be contacted with the appropriate information so that the pavement designs can be revised.

Table 5. Truck Traffic Breakdown

FHWA Class Group	Description	Percent
5	2-axle, single unit	60
6	3-axle, single unit	30
7	4-axle, single unit	0
8	tractor/trailer 3- to 4-axle	10
9	tractor/trailer 3- to 4-axle	0
10	tractor/trailer 3- to 4-axle	0
11	5-axle, multi-trailer	0
12	6-axle, multi-trailer	0

Our pavement design recommendations assuming a maximum of five trucks per day are presented in Table 6.

Table 6. Recommended Standard Pavement Sections

Pavement Use	Trucks per Day[1]	ESALs	AC (inches)	Base Rock (inches)
Automobile Parking	0	10,000	2.5	8.0
Heavy Duty[1]	5	30,000	3.0	9.0

1. See Table 5 for the assumed breakdown of the trucks.

All thicknesses are intended to be the minimum acceptable. The design of the recommended pavement section is based on the assumption that construction will be completed during an extended period of dry weather. Wet weather construction could require an increased thickness of aggregate base. The AC and aggregate base should meet the requirements outlined in the “Materials” section.

Construction traffic should be limited to non-building, unpaved portions of the site or haul roads. Construction traffic should not be allowed on new pavements. If construction traffic is to be allowed on newly constructed road sections, an allowance for this additional traffic will need to be made in the design pavement section. The aggregate base does not account for construction traffic, and haul roads and staging areas should be used as described in the “Construction” section.

If any of these assumptions are incorrect, our office should be contacted with the appropriate information so that the pavement designs can be revised.

5.7 DRAINAGE

5.7.1 Surface Water Control

The ground surface around the structure should be sloped away from its foundations at a minimum 2 percent gradient for a distance of at least 5 feet. Downspouts should discharge into solid, smooth-walled drainage pipes that carry the collected water away from the building foundations. Trapped planter areas should not be created adjacent to buildings without providing means for positive drainage (e.g., swales or catch basins).

5.7.2 Foundation Drainage

We recommend installing footing drains around the perimeter of the proposed building addition. The footing drains should consist of a filter fabric-wrapped, drain rock-filled trench that extends at least 2 feet below the lowest adjacent grade (i.e., slab subgrade elevation). A minimum 4-inch-diameter, perforated pipe should be placed at the base to collect water that gathers in the drain rock. The drain rock and drainage geotextile fabric should meet the specifications outlined in the "Materials" section.

5.8 PERMANENT SLOPES

Permanent cut and fill slopes should not exceed 2H:1V. Slopes within stormwater facilities should not exceed 3H:1V. Access roads and pavements should be located at least 5 feet from the top of cut and fill slopes. The setback should be increased to 10 feet for buildings. The 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.

6.0 CONSTRUCTION

6.1 SITE PREPARATION

6.1.1 Demolition

Demolition should include removal of existing structures, pavements, and utilities that are present underneath areas to be improved. Demolished material should be transported off site for disposal or recycled and used on site if the material is acceptable for use as structural fill. Excavations remaining from site preparation activities should be backfilled with structural fill where below planned site grades. The base of excavations should be excavated to expose firm subgrade before filling. Utility lines abandoned under new structural elements should be completely removed and backfilled with structural fill in accordance with the recommendations provided in the "Structural Fill" section.

6.1.2 Stripping and Grubbing

The existing topsoil and vegetation should be stripped and removed from all proposed building and pavement areas and for a 5-foot margin around such areas. The actual stripping depth should be based on field observations at the time of construction. Stripped material should be transported off site for disposal or used in landscaped areas. Greater depths may be necessary to remove localized zones of organic material or deeper root zones.

Trees should also be removed from improved areas. Root balls should be grubbed out to the depth of the roots. Based on our experience, the grubbing depth required to remove tree root balls will be approximately 2.5 to 3 feet BGS and the grubbing depth to remove brush roots will be approximately 1 foot to 2 feet BGS. Depending on the methods used to remove the root balls, considerable disturbance and loosening of the subgrade could occur during site grubbing. We recommend that soil disturbed during grubbing operations be removed to expose firm subgrade. The resulting excavations should be backfilled with structural fill.

6.1.3 Subgrade Evaluation

Upon completion of stripping and subgrade stabilization, and prior to the placement of fill or pavement improvements, the exposed subgrade should be evaluated by proof rolling. The subgrade should be proof rolled with a fully loaded dump truck or similarly heavy, rubber-tired construction equipment to identify soft, loose, or unsuitable areas. A member of our geotechnical staff should observe the proof rolling to evaluate yielding of the ground surface. During wet weather, subgrade evaluation should be performed by probing with a foundation probe rather than proof rolling. Areas that appear soft or loose should be improved in accordance with subsequent sections of this report.

6.2 CONSTRUCTION CONSIDERATIONS

The fine-grained soils present on this site are easily disturbed. If not carefully executed, site preparation, utility trench work, and excavations can create extensive soft areas and significant repair costs can result. Earthwork planning, regardless of the time of year, should include considerations for minimizing subgrade disturbance.

If construction occurs during or extends into the wet season, or if the moisture content of the surficial soil is more than a couple percentage points above optimum, site stripping and cutting may need to be accomplished using track-mounted equipment. Likewise, the use of granular haul roads and staging areas will be necessary for support of construction traffic during the rainy season or when the moisture content of the surficial soil is more than a few percentage points above optimum. The base rock thickness for pavement areas is intended to support post-construction design traffic loads. This design base rock thickness will likely 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, staging and haul roads with increased thicknesses of base rock will be required.

The amount of staging and haul road areas, as well as the required thickness of granular material, will vary with the contractor’s sequencing of a project and type/frequency of construction equipment. Based on our experience, between 12 and 18 inches of imported granular material is generally required in staging areas and between 18 and 24 inches in haul roads areas. A geotextile fabric is commonly placed below the imported granular material. The actual thickness will depend on the contractor’s means and methods and should be the contractor’s responsibility. The imported granular material, stabilization material, and geotextile are described in the “Materials” section.

6.3 EXCAVATION

6.3.1 Excavation and Shoring

Temporary excavation sidewalls should stand vertical to a depth of approximately 4 feet, provided groundwater seepage is not observed in the sidewalls. Open excavation techniques may be used to excavate trenches with depths between 4 and 8 feet, provided the walls of the excavation are cut at a slope of 1.5H:1V and groundwater seepage is not present. At this inclination, the slopes with loose sand may ravel and require some ongoing repair. Excavations should be flattened if excessive sloughing or raveling occurs. In lieu of large and open cuts, approved temporary shoring may be used for excavation support. A wide variety of shoring and dewatering systems are available. Consequently, we recommend that the contractor be responsible for selecting the appropriate shoring and dewatering systems.

If box shoring is used, it should be understood that box shoring is a safety feature used to protect workers and does not prevent caving. If the excavations are left open for extended periods of time, caving of the sidewalls may occur. The presence of caved material will limit the ability to properly backfill and compact the trenches. The contractor should be prepared to fill voids between the box shoring and the sidewalls of the trenches with sand or gravel before caving occurs.

If shoring is used, we recommend that the type and design of the shoring system be the responsibility of the contractor, who is in the best position to choose a system that fits the overall plan of operation. All excavations should be made in accordance with applicable OSHA and state regulations.

6.3.2 Trench Dewatering

Shallow excavations (less than 5 feet) will not likely encounter groundwater. However, perched groundwater may be encountered after prolonged wet periods. Dewatering systems are best designed by the contractor. It may be possible to remove groundwater encountered by pumping from a sump in the trenches. More intense use of pumps may be required at certain times of the year and where more intense seepage occurs. Removed water should be routed to a suitable discharge point.

If groundwater is present at the base of utility trench excavations, we recommend placing up to 12 inches of stabilization material at the base of the excavations. Trench stabilization material should meet the requirements provided in the "Structural Fill" section.

We note that these recommendations are for guidance only. The dewatering of excavations is the sole responsibility of the contractor, as the contractor is in the best position to select these systems based on their means and methods.

6.3.3 Safety

All excavations should be made in accordance with applicable OSHA requirements and regulations of the state, county, and local jurisdiction. While this report describes certain approaches to excavation and dewatering, the contract documents should specify that the contractor is responsible for selecting excavation and dewatering methods, monitoring the excavations for safety, and providing shoring (as required) to protect personnel and adjacent structural elements.

6.4 MATERIALS

6.4.1 Structural Fill

6.4.1.1 General

Fill should be placed on subgrade that has been prepared in conformance with the "Site Preparation" section. A variety of material may be used as structural fill at the site. However, all material used as structural fill should be free of organic matter or other unsuitable material and should meet the specifications provided in OSSC 00330 (Earthwork), OSSC 00400 (Drainage and

Sewers), and OSSC 02600 (Aggregates), depending on the application. A brief characterization of some of the acceptable materials and our recommendations for their use as structural fill is provided below.

6.4.1.2 On-Site Soil

The material at the site should be suitable for use as general structural fill provided it is properly moisture conditioned; free of debris, organic material, and particles over 4 inches in diameter; and meets the specifications provided in OSSC 00330.12 (Borrow Material).

Based on laboratory test results, the moisture content of the on-site soil will be significantly above the optimum required for compaction. Therefore, moisture conditioning (drying) will be required to use the on-site fine-grained soil for structural fill. Extended dry weather and sufficient area to dry the soil will be required to adequately condition the soil for use as structural fill. The on-site fine-grained soil should not be used as structural fill during the wet season. We note that during summer the near-surface (within 2 to 3 BGS) soils can become dry and require the addition of water to moisture condition for compaction.

When used as structural fill, the on-site fine-grained soils should be placed in lifts with a maximum uncompacted thickness of 8 inches and compacted to not less than 92 percent of the maximum dry density, as determined by ASTM D 1557.

6.4.1.3 Imported Granular Material

Imported granular material used as structural fill should be pit- or quarry-run rock, crushed rock, or crushed gravel and sand and should meet the specifications provided in OSSC 00330.14 (Selected Granular Backfill) or OSSC 00330.15 (Selected Stone Backfill). The imported granular material should also be angular, fairly well graded between coarse and fine material, have less than 5 percent by dry weight passing the U.S. Standard No. 200 sieve, and have at least two fractured faces.

Imported granular material should be placed in lifts with a maximum uncompacted thickness of 12 inches and compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D 1557. During the wet season or when wet subgrade conditions exists, the initial lift should be approximately 18 inches in uncompacted thickness and should be compacted by rolling with a smooth-drum roller without using vibratory action.

6.4.1.4 Stabilization Material

Stabilization material should consist of pit- or quarry-run rock, crushed rock, or crushed gravel and should meet the specifications provided in OSSC 00330.16 (Stone Embankment Material). In addition, the material should have a maximum particle size of 6 inches, less than 5 percent by dry weight passing the U.S. Standard No. 4 sieve, and at least two mechanically fractured faces. The material should be free of organic matter and other deleterious material. Stabilization material should be placed in lifts between 12 and 18 inches thick and compacted to a firm condition.

Where the stabilization material is used for staging or construction haul roads, a geotextile should be placed as a barrier between the soil subgrade and the imported granular material. The placement of the imported granular fill should be done in conformance with the specifications provided in OSSC 00331 (Subgrade Stabilization). The geotextile fabric should meet the specifications provided below for subgrade geotextiles. Geotextile is not required where stabilization material is used at the base of utility trenches.

6.4.1.5 Trench Backfill

Trench backfill placed beneath, adjacent to, and for at least 12 inches above utility lines (i.e., the pipe zone) should consist of well-graded granular material with a maximum particle size of 1½ inches and less than 7 percent by dry weight passing the U.S. Standard No. 200 sieve and should meet the specifications provided in OSSC 00405.13 (Pipe Zone Material). The pipe zone backfill should be compacted to at least 90 percent of the maximum dry density, as determined by ASTM D 1557, or as required by the pipe manufacturer or local building department.

Within roadway alignments, the remainder of the trench backfill up to the subgrade elevation should consist of well-graded granular material with a maximum particle size of 2½ inches and less than 7 percent by dry weight passing the U.S. Standard No. 200 sieve and should meet the specifications provided in OSSC 00405.14 (Trench Backfill; Class B, C, or D). This material should be compacted to at least 92 percent of the maximum dry density, as determined by ASTM D 1557, or as required by the pipe manufacturer or local building department. The upper 3 feet of the trench backfill should be compacted to at least 95 percent of the maximum dry density, as determined by ASTM D 1557.

Outside of structural improvement areas (e.g., roadway alignments or building pads) trench backfill placed above the pipe zone may consist of general fill material that is free of organics and material over 6 inches in diameter and meets the specifications provided in OSSC 00405.14 (Trench Backfill; Class A, B, C, or D). This general trench backfill should be compacted to at least 90 percent of the maximum dry density, as determined by ASTM D 1557, or as required by the pipe manufacturer or local building department.

6.4.1.6 Floor Slab Aggregate Base

Imported granular material used as base rock for building floor slabs should consist of ¾- or 1½-inch-minus material (depending on the application) and meet the requirements in OSSC 00641 (Aggregate Subbase, Base, and Shoulders). In addition, the aggregate should have less than 5 percent by dry weight passing the U.S. Standard No. 200 sieve. The aggregate base should be compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D 1557.

6.4.1.7 Pavement Aggregate Base

Imported granular material used as base rock for building floor slabs should consist of ¾- or 1½-inch-minus material (depending on the application) and meet the requirements in OSSC 00641 (Aggregate Subbase, Base, and Shoulders). In addition, the aggregate should have less than 5 percent by dry weight passing the U.S. Standard No. 200 sieve. The aggregate base should be compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D 1557.

6.4.1.8 Retaining Wall Select Backfill

Backfill material placed behind retaining walls and extending a horizontal distance of ½H, where H is the height of the retaining wall, should consist of select granular material that meets the requirements provided in OSSC 00510.12 (Granular Wall Backfill). We recommend the select granular wall backfill be separated from general fill, native soil, and/or topsoil using a geotextile fabric that meets the specifications provided below for drainage geotextiles.

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

6.4.1.9 Drain Rock Material

Drain rock should consist of angular, granular material that meets the specifications provided in OSSC 00430.11 (Granular Drain Backfill Material) and the aggregate should have at least two fractured faces. The drain rock should be wrapped in a drainage geotextile that meets the specifications provided below for drainage geotextiles.

6.4.1.10 Retaining Wall Leveling Pad

Imported granular material placed at the base of retaining wall footings should consist of select granular material that meets the specifications provided in OSSC 00510.13 (Granular Structure Backfill). The granular material should meet either the 1”-0 or ¾”-0 aggregate size listed in OSSC Table 02630-1 – Grading Requirements for Dense-Graded Aggregate and have at least two mechanically fractured faces. The leveling pad material should be placed in a 6- to 12-inch lift and compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D 1557.

6.4.2 AC

6.4.2.1 ACP

The AC should be Level 2, ½-inch, dense ACP according to OSSC 00744 (Asphalt Concrete Pavement) and compacted to 91 percent of the theoretical maximum density of the mix, as determined by AASHTO T 209. The minimum and maximum lift thickness is 2.0 and 3.0 inches, respectively, for ½-inch ACP. Lift thicknesses desired outside these limits should be discussed with the design team prior to design or construction. Asphalt binder should be performance graded and conform to PG 64-22 or better.

6.4.2.2 Cold Weather Paving Considerations

In general, AC paving is not recommended during cold weather (temperatures less than 40 degrees Fahrenheit). Compacting under these conditions can result in low compaction and premature pavement distress.

Each AC mix design has a recommended compaction temperature range that is specific for the particular AC binder used. In colder temperatures, it is more difficult to maintain the temperature of the AC mix as it can lose heat while stored in the delivery truck, as it is placed, and in the time between placement and compaction. In Oregon, the AC surface temperature during paving should be at least 40 degrees Fahrenheit for lift thickness greater than 2.5 inches and at least 50 degrees Fahrenheit for lift thickness between 2.0 and 2.5 inches.

If paving activities must take place during cold-weather construction as defined above, the project team should be consulted and a site meeting should be held to discuss ways to lessen low compaction risks.

6.4.3 Geotextile Fabric

6.4.3.1 Subgrade Geotextile

The subgrade geotextile should meet the specifications provided in OSSC Table 02320-4 - Geotextile Property Values for Subgrade Geotextile (Separation). The geotextile should be installed in conformance with OSSC 00350 (Geosynthetic Installation). A minimum initial aggregate base lift of 6 inches is required over geotextiles. All drainage aggregate and stabilization material should be underlain by a subgrade geotextile. Geotextile is not required where stabilization material is used at the base of utility trenches.

6.4.3.2 Drainage Geotextile

Drainage geotextile should meet the specifications provided in OSSC Table 02320-1 - Geotextile Property Values for Drainage Geotextile. The geotextile should be installed in conformance with OSSC 00350 (Geosynthetic Installation). A minimum initial aggregate base lift of 6 inches is required over geotextiles.

6.5 EROSION CONTROL

The site soil is susceptible to erosion; therefore, erosion control measures should be carefully planned and in place before construction begins. 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.

7.0 OBSERVATION OF CONSTRUCTION

Satisfactory foundation and earthwork performance depends to a large degree on quality of construction. 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. Subsurface conditions observed during construction should be compared with those encountered during the subsurface exploration. Recognition of changed conditions often requires experience; therefore, qualified personnel should visit the site with sufficient frequency to detect if subsurface conditions change significantly from those anticipated.

We recommend that GeoDesign be retained to observe earthwork activities, including stripping, proof rolling of the subgrade and repair of soft areas, footing subgrade preparation, performing laboratory compaction and field moisture-density tests, observing final proof rolling of the pavement subgrade and base rock, and asphalt placement and compaction.

8.0 LIMITATIONS

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

Exploration observations 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 preliminary 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 for the buildings, and walls, the conclusions and recommendations presented may not be applicable. If design changes are made, we request that we be retained to review our conclusions and recommendations and to provide a written modification or verification.

The scope 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 generally accepted practices in this area at the time the report was prepared. No warranty, 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 T. 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: 6/30/18

REFERENCES

International Building Code, 2012.

ODOT, 2015. *Oregon Standard Specifications for Construction,* Oregon Department of Transportation, 2015 Edition.

State of Oregon, 2014. *Oregon Structural Specialty Code.*

USGS, 2014. *U.S. Seismic Design Maps.* Obtained from website: http://earthquake.usgs.gov/hazards/designmaps/. Last accessed August 22, 2017. Website last updated on June 12, 2014.

FIGURES

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

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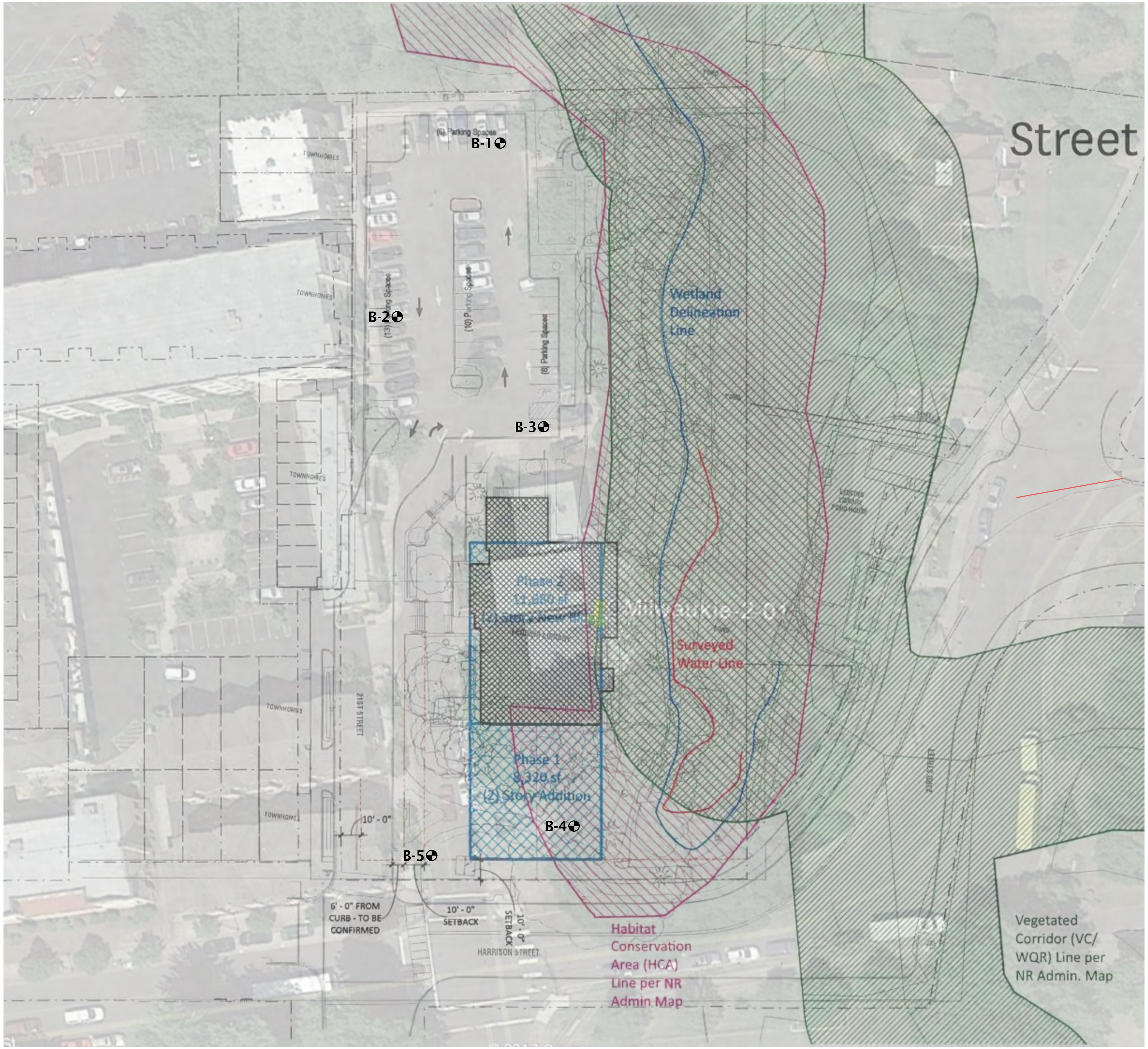
File Name: J:\A-D\CMilwaukie\CMilwaukie-2\CMilwaukie-2-01\Figures\CAD\CMilwaukie-2-01-VM01.dwg | Layout: FIGURE 1

GEODESIGN INC 9450 SW Commerce Circle - Suite 300 Wilsonville OR 97070 503.968.8787 www.geodesigninc.com	CMILWAUKIE-2-01	**VICINITY MAP**	
	AUGUST 2017	LEDDING LIBRARY OF MILWAUKIE MILWAUKIE, OR	**FIGURE 1**



LEGEND:

B-1● BORING

N

0 50 100

(SCALE IN FEET)

SITE PLAN BASED ON IMAGE OBTAINED FROM PLAN B CONSULTANCY

FIGURE 2

SITE PLAN

LEDDING LIBRARY OF MILWAUKIE
MILWAUKIE, OR

CMILWAUKIE-2-01

AUGUST 2017

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APPENDIX A

APPENDIX A

FIELD EXPLORATIONS

GENERAL

We explored the site by drilling five borings (B-1 through B-5) to depths ranging between 8.0 and 16.5 feet BGS. Drilling services were provided by Dan J. Fischer Excavating Inc. of Forest Grove, Oregon, using a trailer-mounted drill rig with solid-stem auger drilling methods. The exploration logs are presented in this appendix.

Approximate locations of our explorations are shown on Figure 2. The exploration locations were determined by pacing from existing site features and should be accurate implied by the methods used.

SOIL SAMPLING

A member of our geology 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 D 1586. 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. Sampling methods and intervals are shown on the exploration logs.

We understand that calibration of the SPT hammer used by Dan J. Fischer Excavating, Inc. has not been completed. The SPT blows completed by Dan J. Fischer Excavating, Inc. were conducted using two wraps around a cathead.

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

CLASSIFICATION

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

ATTERBERG LIMITS

The plastic limit and liquid limit (Atterberg limits) of a selected soil sample were determined in accordance with ASTM D 4318. 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.

MOISTURE CONTENT

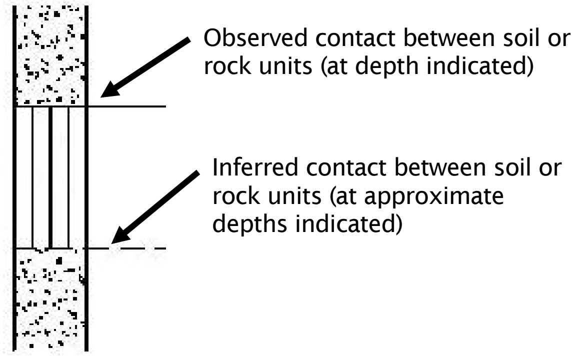
We tested the natural moisture content of selected soil samples in general accordance with ASTM D 2216. The natural moisture content is a ratio of the weight of the water to soil in a test sample and is expressed as a percentage. The test results are presented in this appendix.

PARTICLE-SIZE ANALYSES

Particle-size analyses were completed on selected soil samples in general accordance with ASTM C 117 and ASTM D 1140. 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	PP	Pocket Penetrometer
CBR	California Bearing Ratio	P200	Percent Passing U.S. Standard No. 200 Sieve
CON	Consolidation		
DD	Dry Density	RES	Resilient Modulus
DS	Direct Shear	SIEV	Sieve Gradation
HYD	Hydrometer Gradation	TOR	Torvane
MC	Moisture Content	UC	Unconfined Compressive Strength
MD	Moisture-Density Relationship	VS	Vane Shear
OC	Organic Content	kPa	Kilopascal
P	Pushed Sample		

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

EXPLORATION KEY

TABLE A-1

RELATIVE DENSITY - COARSE-GRAINED SOILS

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 SOILS

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 SOILS (more than 50% retained on No. 200 sieve)	GRAVEL (more than 50% of coarse fraction retained on No. 4 sieve)	CLEAN GRAVELS (< 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
		GRAVELS 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 SANDS (<5% fines)	SW or SP	SAND
		SANDS WITH FINES (≥ 5% and ≤ 12% fines)	SW-SM or SP-SM	SAND with silt
			SW-SC or SP-SC	SAND with clay
		SANDS WITH FINES (> 12% fines)	SM	silty SAND
			SC	clayey SAND
			SC-SM	silty, clayey SAND
FINE-GRAINED SOILS (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 SOILS			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

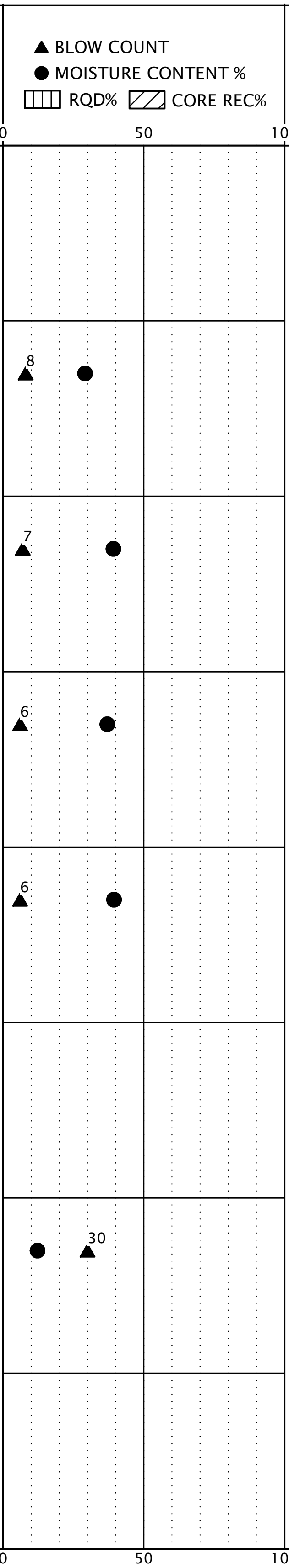
Secondary granular components or other materials such as organics, man-made debris, etc.

Percent	Silt and Clay In:		Percent	Sand and Gravel In:	
	Fine-Grained Soils	Coarse-Grained Soils		Fine-Grained Soils	Coarse-Grained Soils
< 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 %

SOIL CLASSIFICATION SYSTEM

TABLE A-2

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	INSTALLATION AND COMMENTS
0.0	ASPHALT CONCRETE (3.0 inches).			
	AGGREGATE BASE (11.0 inches).	0.3		
	Medium stiff, brown SILT (ML), trace sand; moist, sand is fine.	1.2		
2.5			PP	PP = 1.5 tsf
5.0			PP	PP = 1.75 tsf
7.5	minor sand at 7.5 feet		PP	PP = 1.0 tsf
	Loose, brown, silty SAND (SM); moist, fine.	9.5		
10.0			P200	P200 = 53%
12.5	Medium dense, brown GRAVEL with sand (GP), trace silt; moist, sand is medium. wet at 13.0 feet	12.5		Hard at 12.5 feet. ⊻ 13.0 feet, during drilling
15.0	dense at 15.0 feet			Hard at 15.0 feet.
	Exploration terminated at a depth of 16.5 feet due to refusal. Hammer efficiency factor is unknown. SPT completed using two wraps and a cathead.	16.5		Surface elevation was not measured at the time of exploration.
17.5				
20.0				



DRILLED BY: Dan J. Fischer Excavating, Inc.
LOGGED BY: CR
COMPLETED: 07/12/17

BORING METHOD: solid-stem auger (see document text)
BORING BIT DIAMETER: 4 inches

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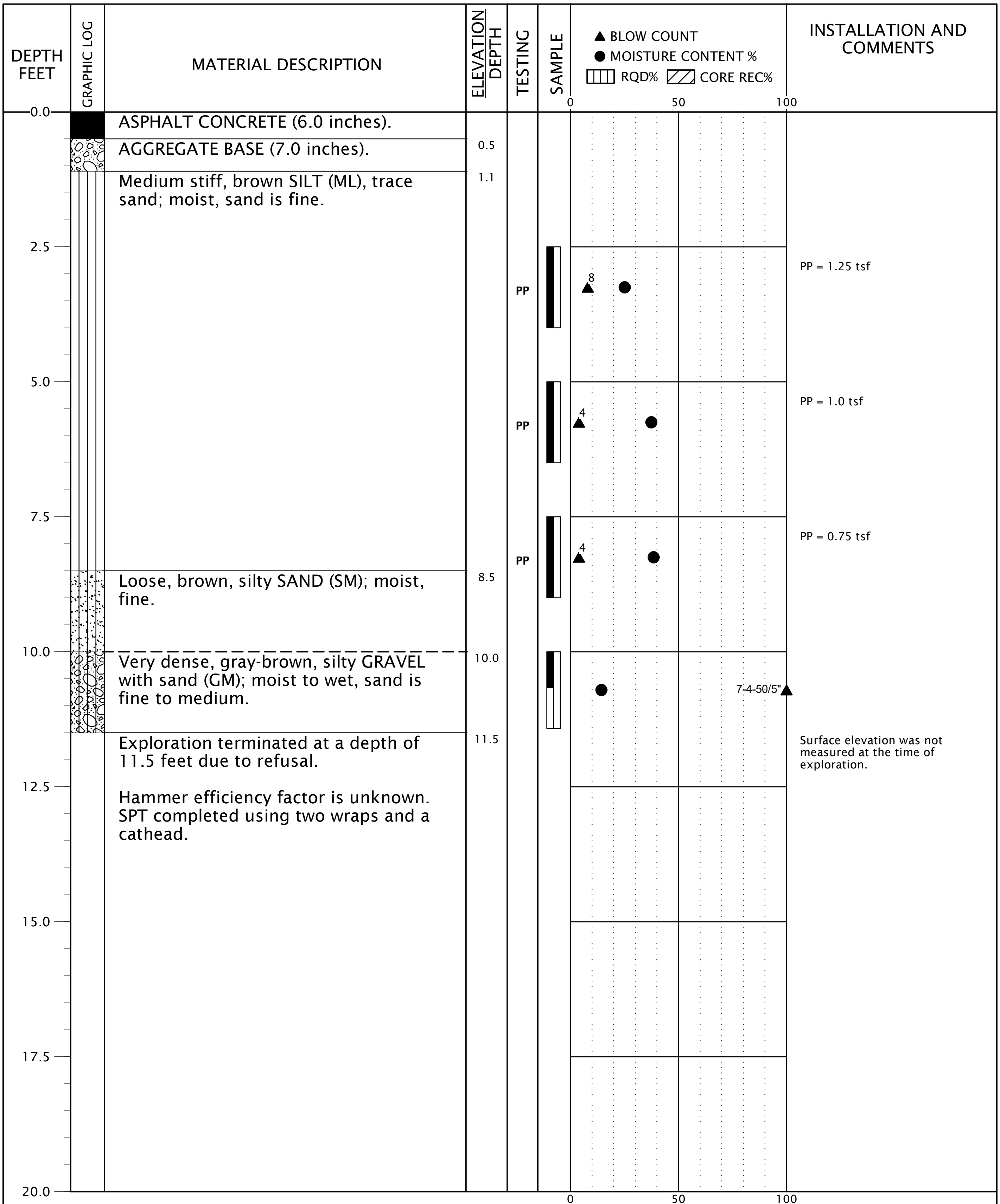
BORING B-1

LEDDING LIBRARY OF MILWAUKIE
MILWAUKIE, OR

FIGURE A-1



DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	BLOW COUNT	INSTALLATION AND COMMENTS
0.0	ASPHALT CONCRETE (6.0 inches).				
	AGGREGATE BASE (7.0 inches).	0.5			
	Medium stiff, brown SILT (ML), trace sand; moist, sand is fine.	1.1			
2.5			PP	8	PP = 1.25 tsf
5.0			PP	4	PP = 1.0 tsf
7.5			PP	4	PP = 0.75 tsf
	Loose, brown, silty SAND (SM); moist, fine.	8.5			
10.0	Very dense, gray-brown, silty GRAVEL with sand (GM); moist to wet, sand is fine to medium.	10.0		7-4-50/5"	
	Exploration terminated at a depth of 11.5 feet due to refusal. Hammer efficiency factor is unknown. SPT completed using two wraps and a cathead.	11.5			Surface elevation was not measured at the time of exploration.

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COMPLETED: 07/12/17

BORING METHOD: solid-stem auger (see document text)
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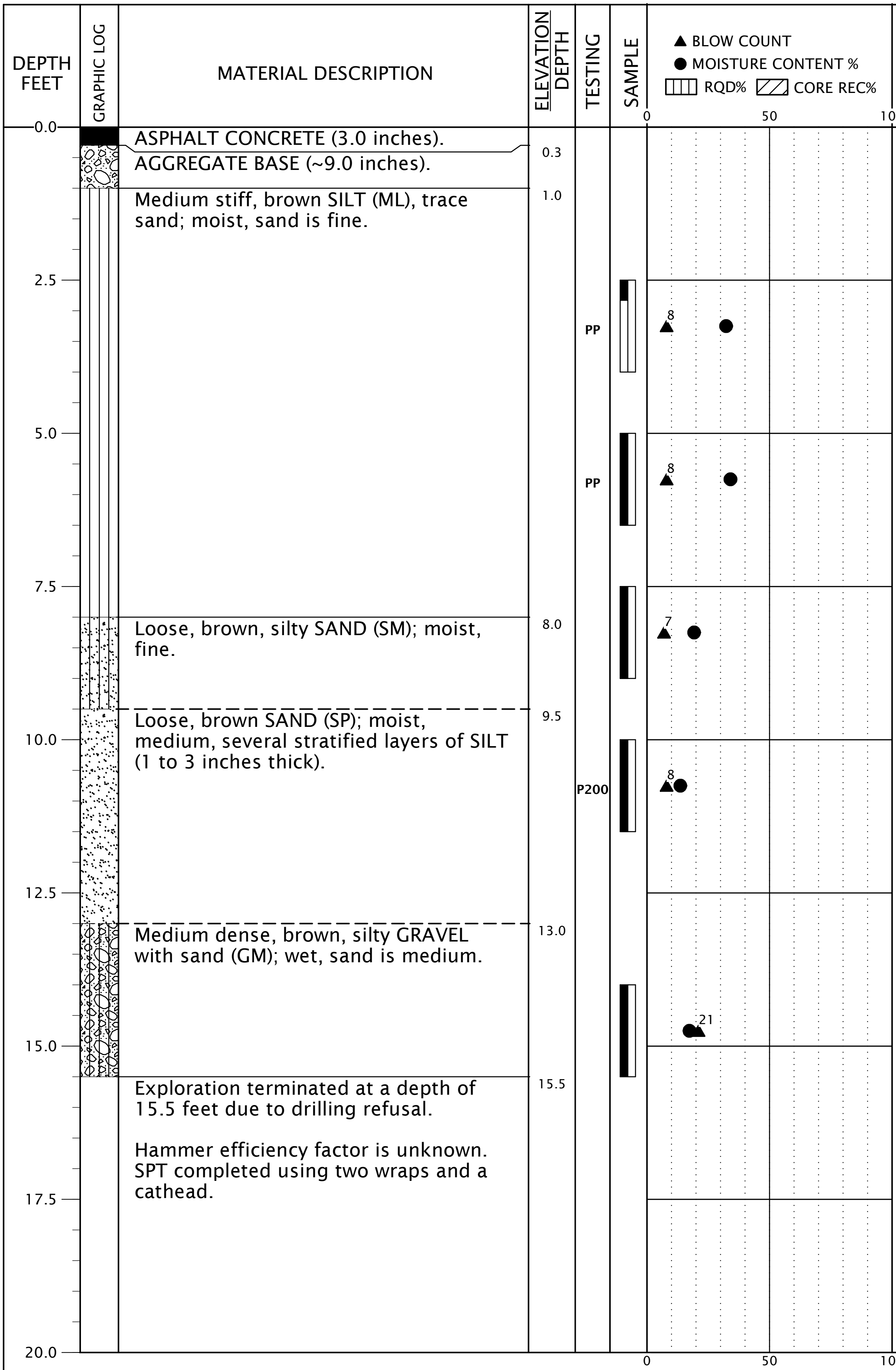
FIGURE A-2

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	INSTALLATION AND COMMENTS
0.0	ASPHALT CONCRETE (3.0 inches).	0.3		
	AGGREGATE BASE (~9.0 inches).	1.0		
	Medium stiff, brown SILT (ML), trace sand; moist, sand is fine.	8.0	PP PP	PP = 1.0 tsf PP = 1.5 tsf
	Loose, brown, silty SAND (SM); moist, fine.	9.5		
	Loose, brown SAND (SP); moist, medium, several stratified layers of SILT (1 to 3 inches thick).	13.0	P200	P200 = 16%
	Medium dense, brown, silty GRAVEL with sand (GM); wet, sand is medium.	15.5		Hard drilling at 13.0 feet. 14.3 feet, during drilling
	Exploration terminated at a depth of 15.5 feet due to drilling refusal. Hammer efficiency factor is unknown. SPT completed using two wraps and a cathead.			Surface elevation was not measured at the time of exploration.



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COMPLETED: 07/12/17

BORING METHOD: solid-stem auger (see document text)
BORING BIT DIAMETER: 4 inches

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BORING B-3

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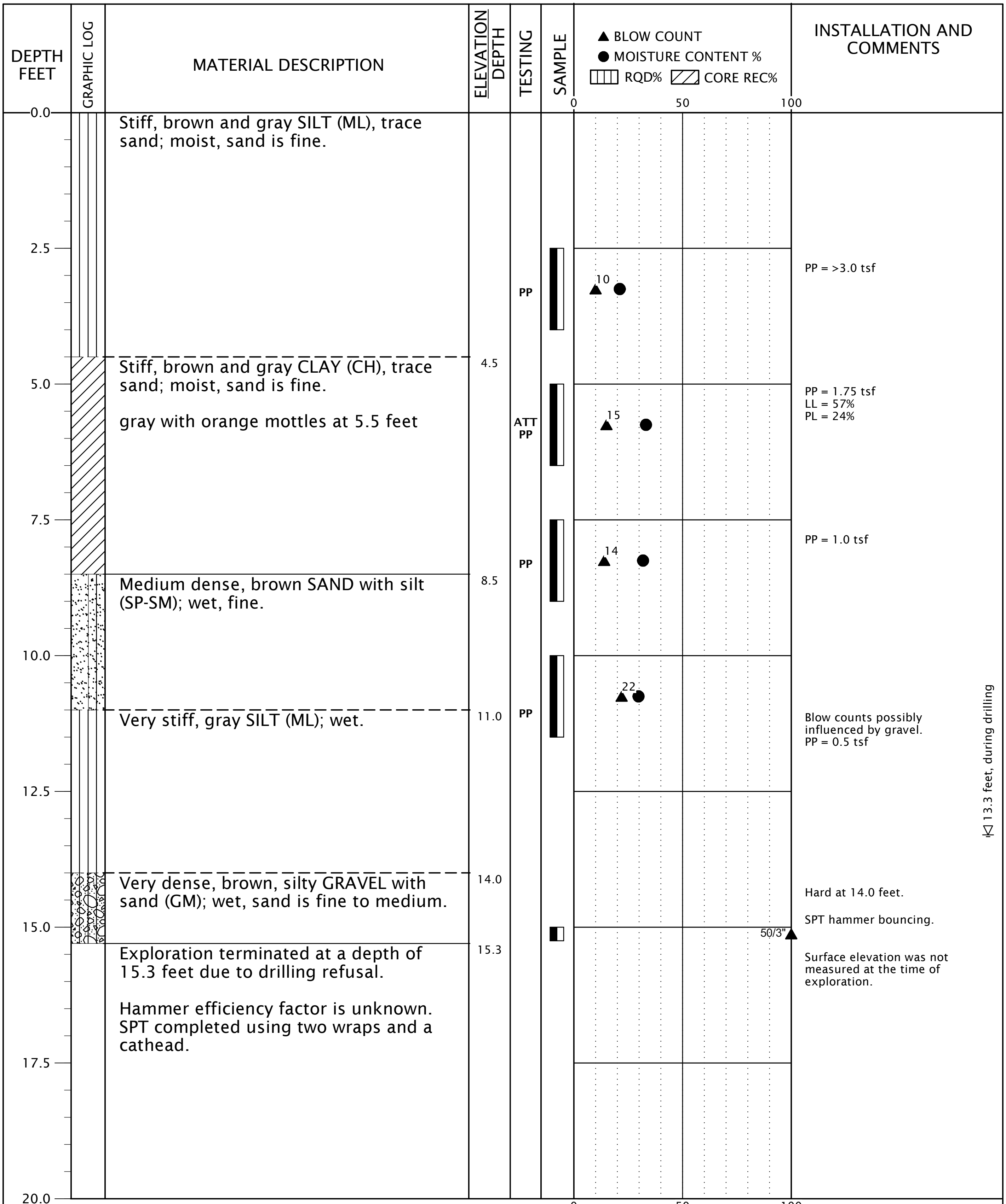
FIGURE A-3

BORING LOG CMILWAUKIE-2-01-B1_5.GPJ GEODESIGN.GDT PRINT DATE: 8/25/17:RC:KT



ELEVATION DEPTH	MATERIAL DESCRIPTION
	Stiff, brown and gray SILT (ML), trace sand; moist, sand is fine.
4.5	Stiff, brown and gray CLAY (CH), trace sand; moist, sand is fine. gray with orange mottles at 5.5 feet
8.5	Medium dense, brown SAND with silt (SP-SM); wet, fine.
11.0	Very stiff, gray SILT (ML); wet.
14.0	Very dense, brown, silty GRAVEL with sand (GM); wet, sand is fine to medium.
15.3	Exploration terminated at a depth of 15.3 feet due to drilling refusal. Hammer efficiency factor is unknown. SPT completed using two wraps and a cathead.

DEPTH FEET	TESTING	BLOW COUNT	INSTALLATION AND COMMENTS
2.5	PP	10	PP = >3.0 tsf
5.0	ATT PP	15	PP = 1.75 tsf LL = 57% PL = 24%
7.5	PP	14	PP = 1.0 tsf
10.0	PP	22	Blow counts possibly influenced by gravel. PP = 0.5 tsf
14.0			Hard at 14.0 feet.
15.0		50/3"	SPT hammer bouncing. Surface elevation was not measured at the time of exploration.

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BORING BIT DIAMETER: 4 inches

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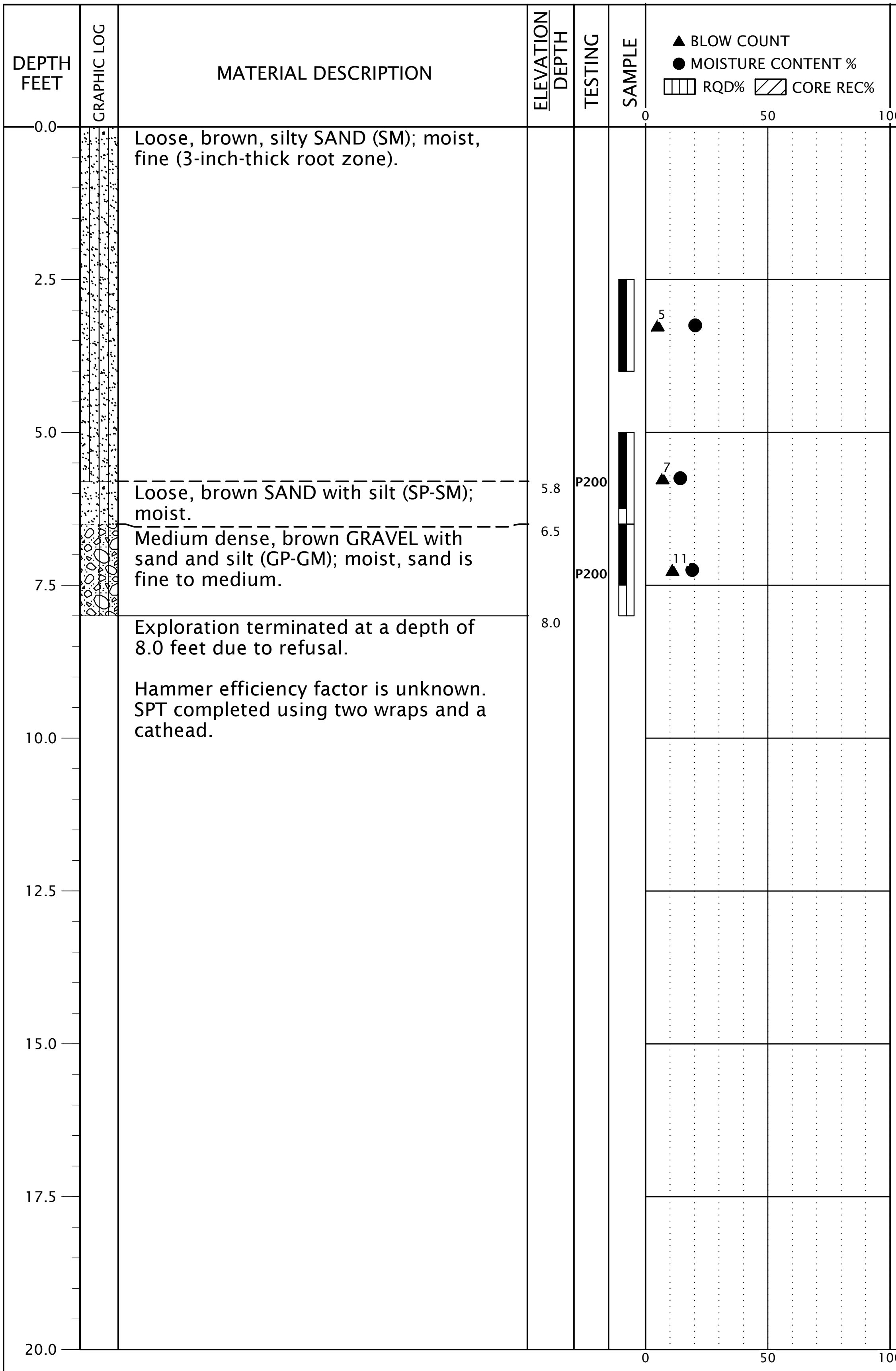
FIGURE A-4

BORING LOG CMILWAUKIE-2-01-B1_5.GPJ GEODESIGN.GDT PRINT DATE: 8/25/17:RC:KT

MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	INSTALLATION AND COMMENTS
Loose, brown, silty SAND (SM); moist, fine (3-inch-thick root zone).			
Loose, brown SAND with silt (SP-SM); moist.	5.8	P200	P200 = 24% Infiltration test: 0.3 inch per hour at 6.0 feet.
Medium dense, brown GRAVEL with sand and silt (GP-GM); moist, sand is fine to medium.	6.5	P200	Hard at 6.5 feet. P200 = 27%
Exploration terminated at a depth of 8.0 feet due to refusal. Hammer efficiency factor is unknown. SPT completed using two wraps and a cathead.	8.0		Surface elevation was not measured at the time of exploration.



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BORING METHOD: solid-stem auger (see document text)
BORING BIT DIAMETER: 4 inches

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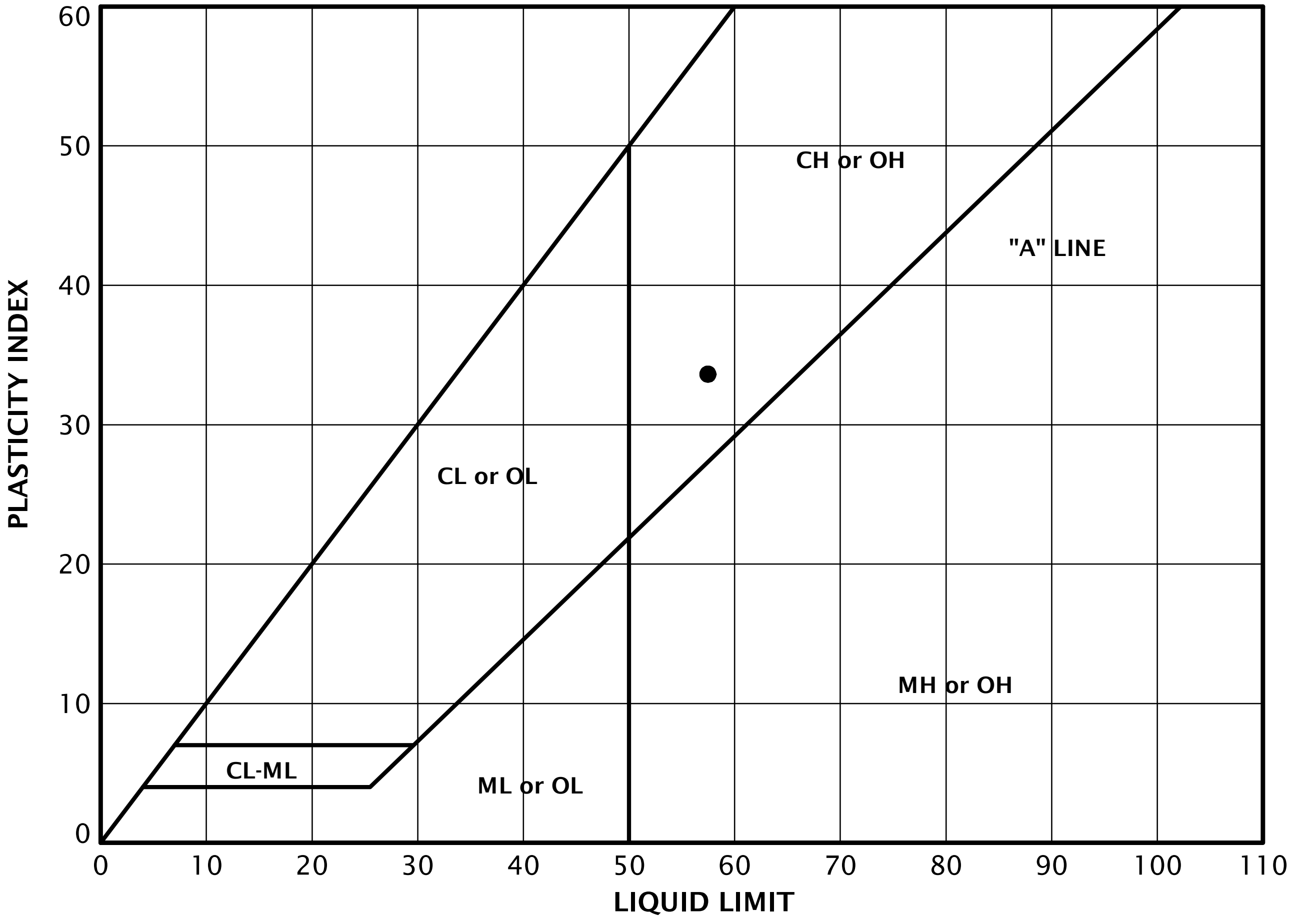
FIGURE A-5



KEY	EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	MOISTURE CONTENT (PERCENT)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
●	B-4	5.0	43	57	24	33

ATTERBERG_LIMITS 7 CMILWAUKIE-2-01-B1_5.GPJ GEODESIGN.GDT PRINT DATE: 8/4/17:KT

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ATTERBERG LIMITS TEST RESULTS

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FIGURE A-6

SAMPLE INFORMATION			MOISTURE CONTENT (PERCENT)	DRY DENSITY (PCF)	SIEVE			ATTERBERG LIMITS		
EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	ELEVATION (FEET)			GRAVEL (PERCENT)	SAND (PERCENT)	P200 (PERCENT)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
B-1	2.5		29							
B-1	5.0		39							
B-1	7.5		37							
B-1	10.0		39				53			
B-1	15.0		12							
B-2	2.5		25							
B-2	5.0		37							
B-2	7.5		39							
B-2	10.0		14							
B-3	2.5		32							
B-3	5.0		34							
B-3	7.5		19							
B-3	10.0		14				16			
B-3	14.0		17							
B-4	2.5		21							
B-4	5.0		33					57	24	33
B-4	7.5		32							
B-4	10.0		30							
B-5	2.5		20							
B-5	5.0		14				24			
B-5	6.5		19				27			

LAB SUMMARY CMILWAUKIE-2-01-B1_5.GPJ GEODESIGN.GDT PRINT DATE: 8/4/17:KT

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SUMMARY OF LABORATORY DATA

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FIGURE A-7

APPENDIX B

APPENDIX B

SITE-SPECIFIC SEISMIC HAZARD EVALUATION

INTRODUCTION

The information in this appendix summarizes the results of a site-specific seismic hazard evaluation for the proposed improvements at Ledding Library in Milwaukie, Oregon. This seismic hazard evaluation was performed to meet the requirement of the 2014 SOSSC.

SITE CONDITIONS

REGIONAL GEOLOGY

The site is located within the Portland Basin, which is separated from by the Tualatin Basin by the Tualatin Mountains (Portland Hills) to the west. Geologic mapping by Ma et al. (2012) and Beeson et al. (1989) shows the near-surface geology mapped as catastrophic Missoula flood deposits (channel facies). The Missoula flood deposits generally consists of a varying mix of unconsolidated deposits of sand, silt, and gravel sediment, which were deposited in major flood events. Since being deposited, the deposits have been modified by recent alluvium (Beeson et al., 1989). The Missoula flood deposits are underlain by undifferentiated sediments, which are commonly fine-grained sediments that overlay basalt bedrock in the site vicinity. The thickness is highly variably and ranges from less than 15 feet to greater than 200 feet (Beeson et al., 1989). The undifferentiated sediments are underlain by Eocene (54 million to 33 million years old) Basalt of Waverly Heights, a sequence of subaerial basaltic lava flows and associated undifferentiated sedimentary rocks (Beeson et al., 1989).

SUBSURFACE CONDITIONS

A detailed description of site subsurface conditions is presented in the main report.

SEISMIC SETTING

Earthquake Source Zones

Three scenario earthquakes were considered for this study consistent with the local seismic setting. Two of the possible earthquake sources are associated with the CSZ, and the third event is a shallow local crustal earthquake that could occur in the North American plate. The three earthquake scenarios are discussed below.

Regional Events

The CSZ is the region where the Juan de Fuca Plate is being subducted beneath the North American Plate. This subduction is occurring in the coastal region between Vancouver Island and northern California. Evidence has accumulated suggesting that this subduction zone has generated eight great earthquakes in the last 4,000 years, with the most recent event occurring approximately 300 years ago (Weaver and Shedlock, 1991). The fault trace is mapped approximately 50 to 120 km off the Oregon Coast.

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

1. An interface event earthquake on the seismogenic part of the interface between the Juan de Fuca Plate and the North American Plate on the CSZ. This source is reportedly capable of generating earthquakes with a moment magnitude of between 8.5 and 9.0.
2. A deep intraplate earthquake on the seismogenic part of the subducting Juan de Fuca Plate. These events typically occur at depths of between 30 and 60 km. This source is capable of generating an event with a moment magnitude of up to 7.5.

Local Events

A significant earthquake could occur on a local fault near the site within the design life of the facility. Such an event would cause ground shaking at the site that could be more intense than the CSZ events, though the duration would be shorter. Figure B-1 shows the locations of faults with potential Quaternary movement within a 20-mile radius of the site (USGS, 2014a; PNSN, 2014). Figure B-2 shows the interpreted locations of seismic events that occurred between 1833 and 2014 (USGS, 2014b). The most significant faults in the site vicinity are the Oatfield fault and Portland Hills fault. Table B-1 presents the closest mapped distance and mapped length of these faults.

Table B-1. Closest Crustal Faults

Source	Closest Mapped Distance[1] (km)	Mapped Length[1] (km)
Oatfield fault	1.0	24
Portland Hills fault	2.3	49

1. Reported by USGS (USGS, 2014a)

Oatfield Fault

The northwest-striking Oatfield fault forms northeast-facing escarpments in volcanic rocks of the Miocene CRBG in the Tualatin Mountains and northern Willamette Valley. The fault may be part of the Portland Hills-Clackamas River structural zone. The Oatfield fault is primarily mapped as a very high-angle, reverse fault with apparent down-to-the-southwest displacement, but a few kilometer-long reach of the fault with down-to-the-northeast displacement is mapped in the vicinity of the Willamette River. This apparent change in displacement direction along strike may reflect a discontinuity in the fault trace or could reflect the right-lateral, strike-slip displacement that characterizes other parts of the Portland Hills-Clackamas River structural zone. The fault has also been modeled as a 70-degree, east-dipping reverse fault. Reverse displacement with a right-lateral, strike-slip component is consistent with the tectonic setting, mapped geologic relations, and microseismicity in the area. Fault scarps on surficial deposits have not been described, but exposures in a light rail tunnel showing offset of approximately 1 M_a Boring Lava across the fault indicate Quaternary displacement (Personius, 2002a).

Portland Hills Fault

The northwest-striking Portland Hills fault forms the prominent linear northeastern margin of the Tualatin Mountains (Portland Hills) and the southwestern margin of the Portland Basin; this basin may be a right-lateral, pull-apart basin in the forearc of the CSZ or a piggyback synclinal basin formed between antiformal uplifts of the Portland fold belt. The fault is part of the Portland Hills-Clackamas River structural zone, which controlled the deposition of Miocene CRBG lavas in the region. The crest of the Portland Hills is defined by the northwest-striking Portland Hills anticline. Sense of displacement on the Portland Hills fault is poorly known and controversial. The fault was originally mapped as a down-to-the-northeast normal fault. The fault has also been mapped as part of a regional-scale zone of right-lateral oblique slip faults and as a steep escarpment caused by asymmetrical folding above a southwest-dipping blind thrust. Reverse displacement with a right-lateral, strike-slip component may be most consistent with the tectonic setting, mapped geologic relations, aeromagnetic data, and microseismicity in the area. Fault scarps on surficial Quaternary deposits have not been described along the fault trace, but some geomorphic (steep, linear escarpment, triangular facets, over-steepened, and knick-pointed tributaries) and geophysical (aeromagnetic, seismic reflection, and ground penetrating radar) evidence suggest Quaternary displacement (Personius, 2012b).

DESIGN EARTHQUAKE

We determined acceleration response spectra for the three postulated scenarios discussed above by using the USGS Interactive Mapping Project that provides a probabilistic site response spectrum for the site assuming bedrock conditions. We assumed an MCE that has a 2 percent probability of exceedance in a 50-year period, as required by the 2014 SOSSC. Some of the major contributing sources to the PGA reported by USGS are presented in Table 2.

Table 2. Partial List of Faults Considered

Source	Magnitude[1] (M_w)	Distance[1] (km)
Cascadia Megathrust (Deep Interface)	9.10	82.70
Portland Hills	6.75	2.93
Cascadia Megathrust (Middle Interface)	8.92	132.72
Grant Butte 50	6.19	8.23

1. Reported by USGS (USGS, 2014)

Figure B-3 shows the site-specific bedrock spectrum as reported by USGS. The soil profile at the site is classified as a Site Class D as prescribed by Section 1613 of SOSSC. Accordingly, the bedrock response spectrum has been amplified using the factors prescribed by SOSSC for Site Class D. Table 3 presents the factors.

Table 3. SOSSC 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 = 0.984 g	S_1 = 0.421 g
Site Coefficient, F	F_a = 1.107	F_v = 1.579
Adjusted Spectral Acceleration, S_M	S_{MS} = 1.088 g	S_{M1} = 0.665 g

Figure B-3 shows adjusted spectrum appropriate for use in design of structures at the site.

GEOLOGIC HAZARDS

In addition to ground shaking, site-specific geologic conditions can influence the potential for earthquake damage. Deep deposits of loose or soft alluvium can amplify ground motions, resulting in increased seismic loads on structures. Other geologic hazards are related to soil failure and permanent ground deformation. Permanent ground deformation could result from liquefaction, lateral spreading, landsliding, and fault rupture. The following sections provide additional discussion regarding potential seismic hazards that could affect the planned development.

FAULT SURFACE RUPTURE

The Oatfield fault is mapped 0.6 mile northeast of the site and the Portland Hills fault is mapped 1.4 miles southwest of the site. Consequently, it is our opinion that the probability of surface fault rupture beneath the site is low.

LIQUEFACTION

Liquefaction is caused by a rapid increase in pore water pressure that reduces the effective stress between soil particles to near zero. Granular soil, which relies on interparticle friction for strength, is susceptible to liquefaction until the excess pore pressures can dissipate. In general, loose, saturated sand soil with low silt and clay content is the most susceptible to liquefaction. Silty soil with low plasticity is moderately susceptible to liquefaction under relatively higher levels of ground shaking

Based on a review of the available information, soil types encountered, and groundwater depth, it is our opinion that liquefaction is not considered a hazard under design levels of ground shaking.

LATERAL SPREAD

Lateral spread is a liquefaction-related seismic hazard. Development areas subject to lateral spreading are typically gently sloping or flat sites underlain by liquefiable sediments adjacent to an open face, such as riverbanks. Liquefied soil adjacent to open faces may “flow” in that direction, resulting in surface cracking and lateral displacement towards the open face (i.e., riverbank). Since the site has low susceptibility to liquefaction, lateral spreading is expected to be negligible at this site.

GROUND MOTION AMPLIFICATION

The soil profile at the site is classified as a Site Class D as prescribed by Section 1613.5.5 of SOSSC. Accordingly, the bedrock response spectrum has been appropriately amplified using the factors prescribed by the code for Site Class D.

LANDSLIDE

Earthquake-induced landsliding generally occurs in steeper slopes comprised of relatively weak soil deposits. The site and surrounding area are relatively flat, and seismically induced landslides are not considered a site hazard.

SETTLEMENT

Settlement due to earthquakes is most prevalent in relatively deep deposits of dry, clean sand. We do not anticipate that seismic-induced settlement in addition to liquefaction-induced settlement will occur during design levels of ground shaking.

SUBSIDENCE/UPLIFT

Subduction zone earthquakes can cause vertical tectonic movements. The movements reflect coseismic strain release accumulation associated with interplate coupling in the subduction zone.

Based on our review of the literature, the locked zone of the CSZ is located in excess of 90 miles from the site. Consequently, we do not anticipate that subsidence or uplift is a significant design concern.

LURCHING

Lurching is a phenomenon generally associated with very high levels of ground shaking, which cause localized failures and distortion of the soil. The anticipated ground accelerations shown on Figure C-3 are below the threshold required to induce lurching of the site soil.

SEICHE AND TSUNAMI

The site is inland and elevated away from tsunami inundation zones and away from large bodies of water that may develop seiches. Seiches and tsunamis are not considered a hazard in the site vicinity.

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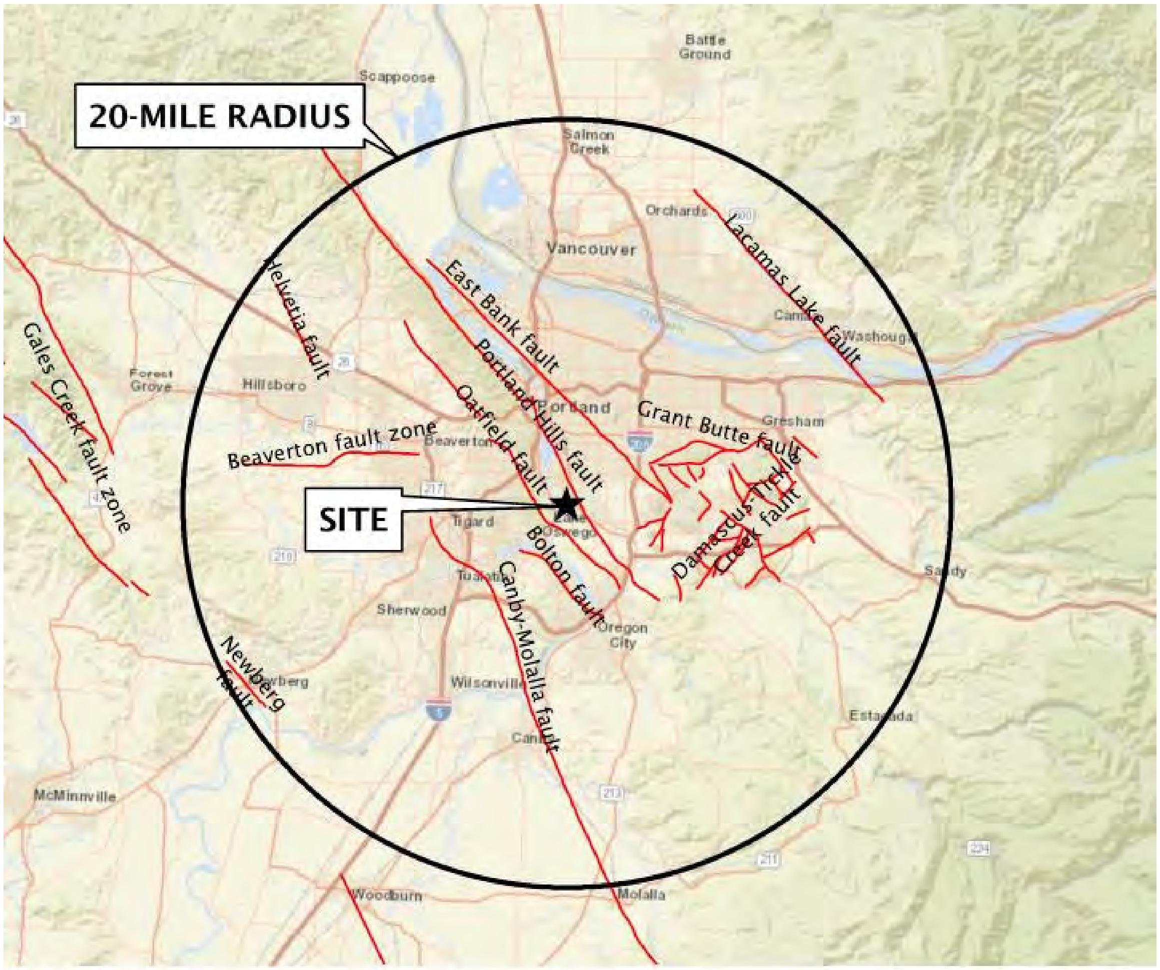

LEGEND

—— QUATERNARY FAULT

FAULTS PROVIDED BY THE USGS FAULT AND FOLD DATABASE (2006)

N

0 5 10

(SCALE IN MILES)

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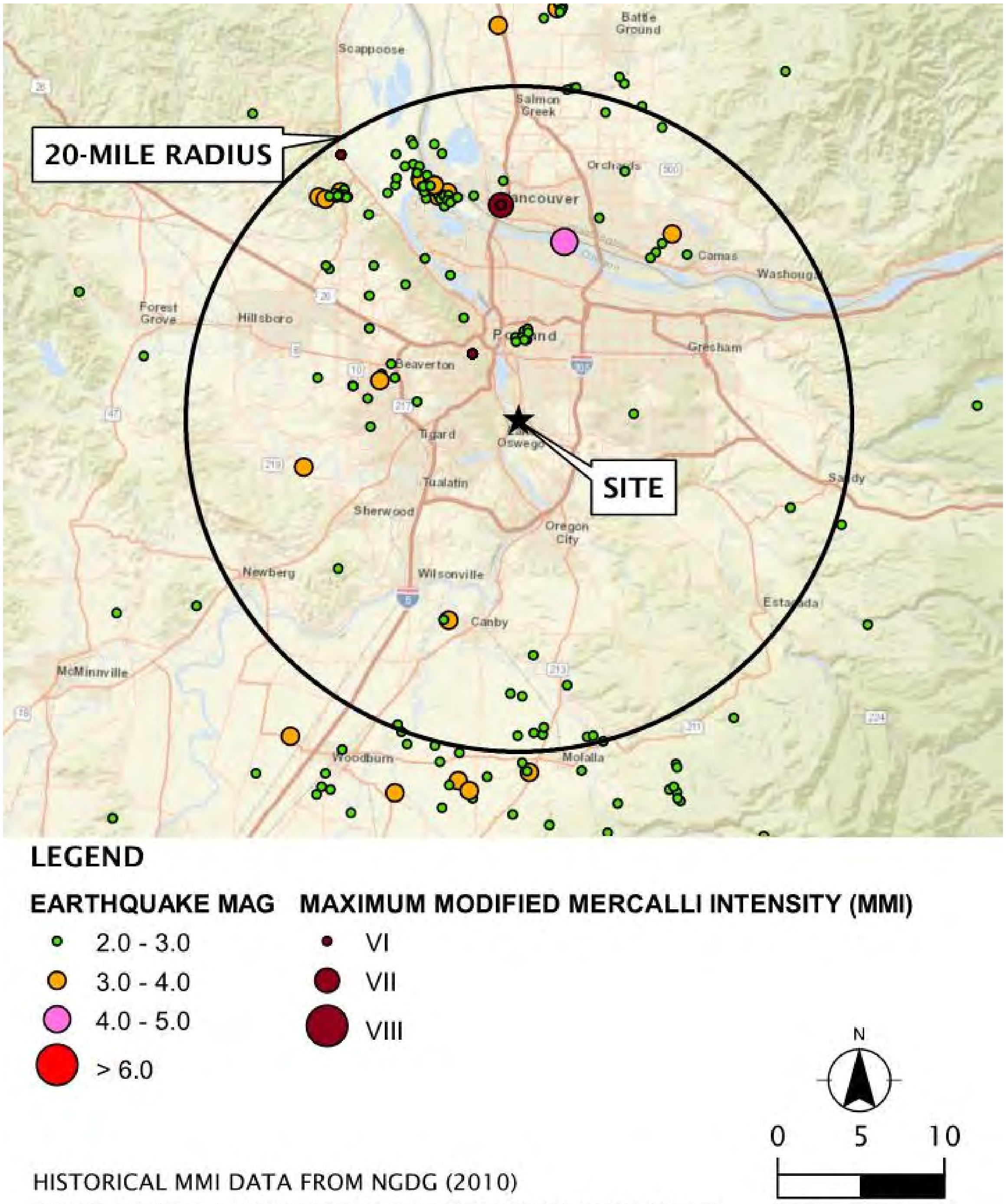
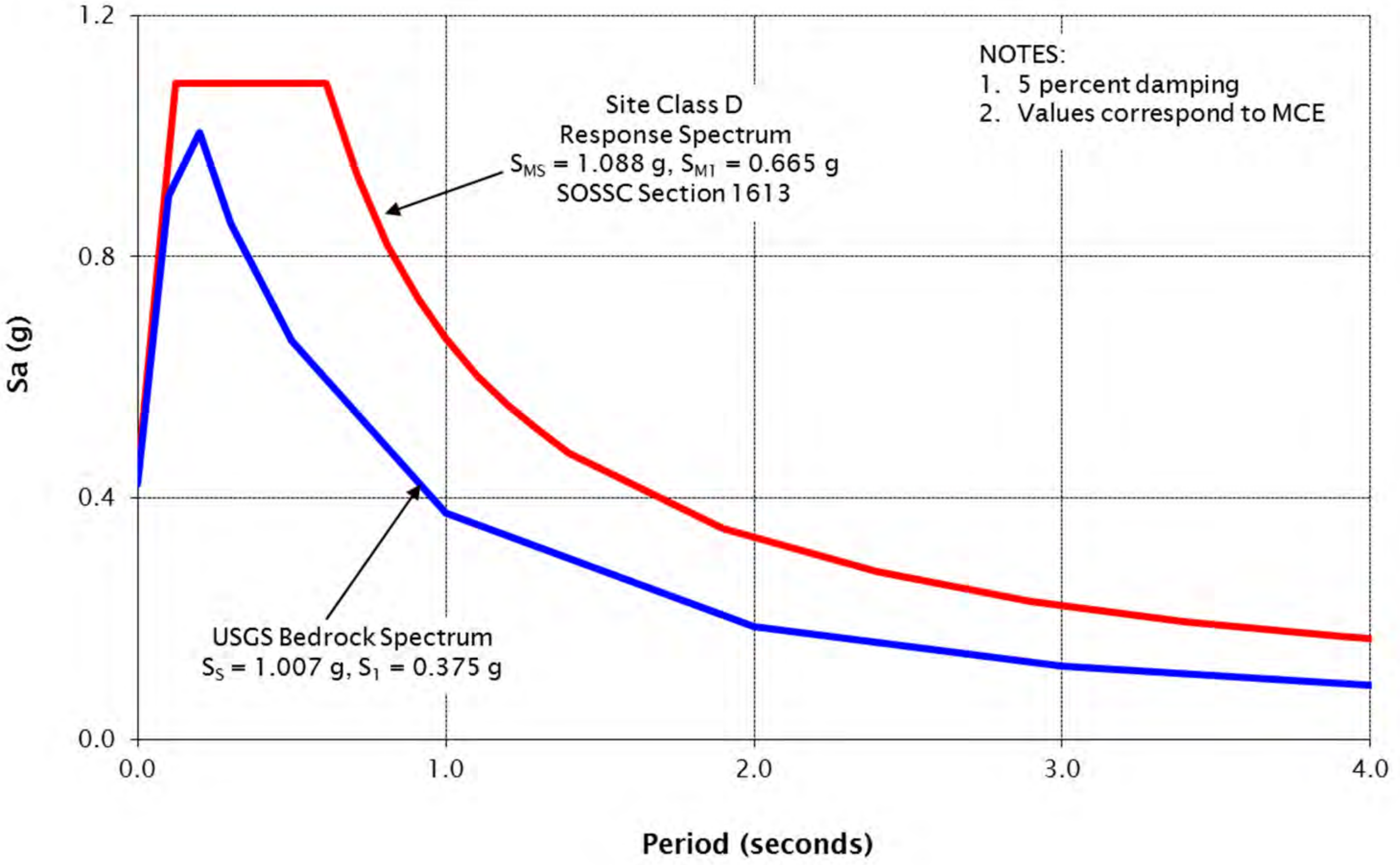
	AUGUST 2017	LEDDING LIBRARY OF MILWAUKIE MILWAUKIE, OR	FIGURE B-1



HISTORICAL MMI DATA FROM NGDG (2010)
INSTRUMENTAL MAGNITUDE FROM USGS (2009), PNSN (2017)

CMilwaukie-2-01-FB1_B2.docx Print Date: 8/2/17

GEODESIGN INC 9450 SW Commerce Circle - Suite 300 Wilsonville OR 97070 503.968.8787 www.geodesigninc.com	CMILWAUKIE-2-01	**HISTORICAL SEISMICITY MAP**
	AUGUST 2017	LEDDING LIBRARY OF MILWAUKIE MILWAUKIE, OR — **FIGURE B-2**



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SITE RESPONSE SPECTRA

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FIGURE B-3

ACRONYMS AND ABBREVIATIONS

ACRONYMS AND ABBREVIATIONS

AASHTO	American Association of State Highway and Transportation Officials
AC	asphalt concrete
ACP	asphalt concrete pavement
ASTM	American Society for Testing and Materials
BGS	below ground surface
CRBG	Columbia River Basalt Group
CSZ	Cascadia Subduction Zone
ESAL	equivalent single-axle load
FHWA	Federal Highway Administration
g	gravitational acceleration (32.2 feet/second^2)
H:V	horizontal to vertical
IBC	International Building Code
km	kilometers
MCE	maximum considered earthquake
MSL	mean sea level
OSHA	Occupational Safety and Health Administration
OSSC	Oregon Standard Specifications for Construction (2015)
pcf	pounds per cubic foot
PG	performance grade
PGA	peak ground acceleration
psf	pounds per square foot
psi	pounds per square inch
SOSSC	State of Oregon Structural Specialty Code
SPT	standard penetration test
USGS	U.S. Geological Survey

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Appendix C

Detention & Water Quality Calculations

Appendix C
PAC - Scott
Existing Planter

PAC Report

Project Details

Project Name	Permit No	Created
Scott Park		6/6/2023 7:37:43 PM
Project Address	**Designer**	**Last Modified**
10660 SE 21st Ave		6/6/2023 10:37:46 PM
	Company	**Report Generated**
	KPFF	6/6/2023 3:40:11 PM

Project Summary

Catchment Name	Impervious Area (sq ft)	Native Soil Design Infiltration Rate (in/hr)	Level	Category	Config	Facility Area (excl. free board) (sq ft)	Facility Sizing Ratio (%)	PR Results	Infiltration Results	Flow Control Results
SP-EX	10166	0	2C	FlatPlanter	D	280.00	2.75	Pass	NA	Pass

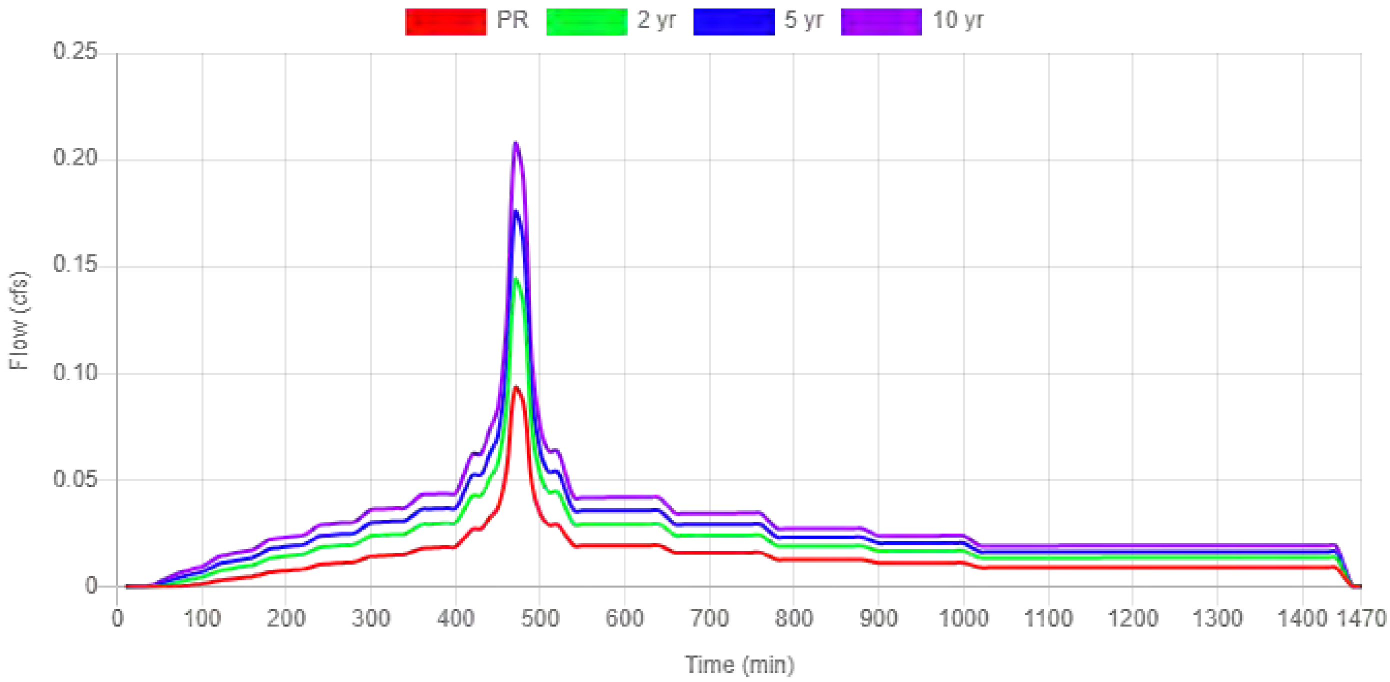
Appendix C
PAC - Scott
Existing Planter

SP-EX

Site Soils & Infiltration Testing	Infiltration Testing Procedure OpenPit Tested Native Soil Infiltration Rate 0 in/hr
Correction Factor	CF_{test} 2
Design Infiltration Rates	Native Soil 0 in/hr Imported Blended Soil 6 in/hr
Catchment Information	Hierarchy Level 2C Hierarchy Description Base requirement for all other discharge points Pollution Reduction Requirement Filter the post-development stormwater runoff from the water quality storm event through the blended soil. Infiltration Requirement N/A Flow Control Requirement Limit the 2-yr, the 5-yr, and the 10-yr post-development peak flows to their respective pre-development peak flows. Impervious Area 10166 sq ft 0.233 acre Pre-Development Time of Concentration (Tc_{pre}) 5 min Post-Development Time of Concentration (Tc_{post}) 5 min Pre-Development Curve Number (CN_{pre}) 97 Post-Development Curve Number (CN_{post}) 98

SBUH Results

Post-Development Runoff



	Pre - Development Rate and Volume		Post - Development Rate and Volume	
	Peak Rate (cfs)	**Total Volume (cf)**	**Peak Rate (cfs)**	**Total Volume (cf)**
PR	0.0867	1093.2	0.0929	1176.4
2-Year	0.1381	1749.4	0.1435	1839.5
5-Year	0.1704	2168.1	0.1753	2260.8
10-Year	0.2024	2588.2	0.2069	2682.8

	Overflow		Underdrain Outflow		Infiltration	
	Peak Rate (cfs)	**Total Volume (cf)**	**Peak Rate (cfs)**	**Total Volume (cf)**	**Peak Rate (cfs)**	**Total Volume (cf)**
PR	0	0	0.029	1173.6	0	0
2-Year	0	0	0.029	1836.7	0	0
5-Year	0.064	183.8	0.029	2074.3	0	0
10-Year	0.162	428	0.029	2252.1	0	0

Flat Planter

Site Soils & Infiltration Testing

Category
Flat Planter

Shape
Null

Location
Parcel

Configuration
D: Lined Facility with RS and Ud

Above Grade Storage Data

Bottom Area
280 sq ft

Bottom Width
10 ft

Overflow Height
12 in

Total Depth of Blended Soil plus Rock
28 in

Surface Storage Capacity at Overflow
280 cu ft

Design Infiltration Rate to Soil Underlying the Facility
0.000 cfs

Design Infiltration Rate for Imported Blended Soil in the Facility
0.039 cfs

Below Grade Storage Data

Catchment is too small for flow control?
No

Rock Area
25.00 sq ft

Rock Width
3.00 ft

Rock Storage Depth
10.0 in

Rock Porosity
0.3

Underdrain Height

<table>
<tr><td></td><td>1.0 in
Percent of Facility Base that Allows Infiltration
0 %
Orifice (Y/N)?
Yes
Orifice Diameter
1.000 in</td></tr>
<tr><td>Facility Facts</td><td>Total Facility Area (excluding freeboard)
280.00 sq ft
Sizing Ratio
2.75 %</td></tr>
<tr><td>Pollution Reduction Results</td><td>Pollution Reduction Score
Pass
Overflow Volume
0.00 cf
Surface Capacity Used
36.26 %</td></tr>
<tr><td>Flow Control Results</td><td>Flow Control Score
Pass
<table>
<tr><th></th><th>STORMWATER FACILITY OUTFLOW (CFS)</th><th></th><th>PRE-DEVELOPMENT RUNOFF (CFS)</th></tr>
<tr><td>2 year</td><td>0.0288</td><td><=</td><td>0.1381</td></tr>
<tr><td>5 year</td><td>0.0928</td><td><=</td><td>0.1704</td></tr>
<tr><td>10 year</td><td>0.1910</td><td><=</td><td>0.2024</td></tr>
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</td></tr>
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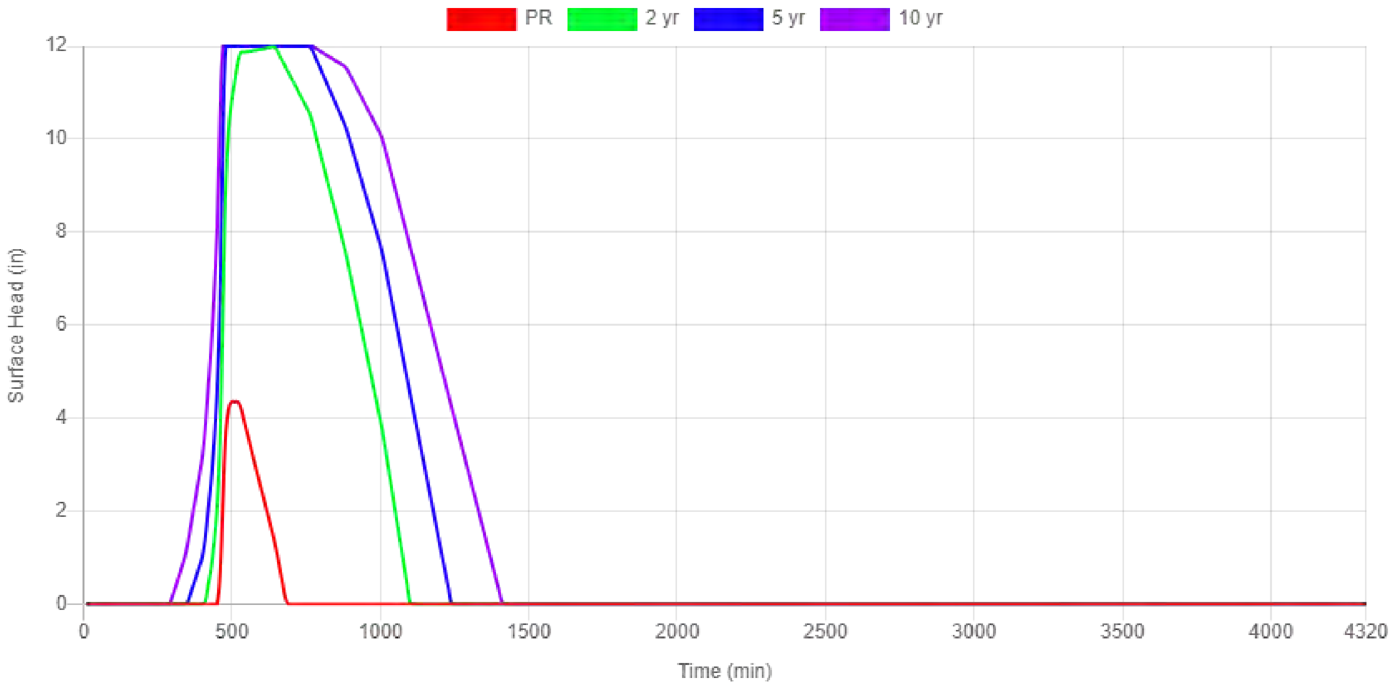
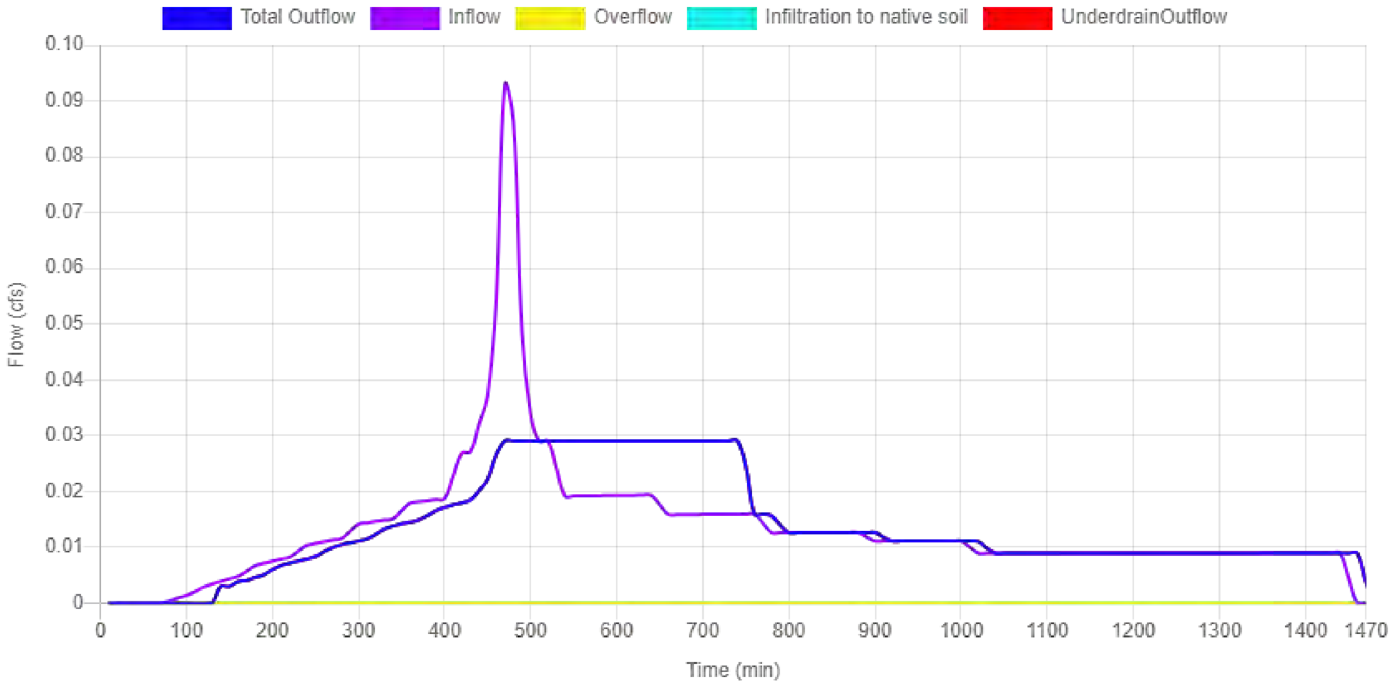
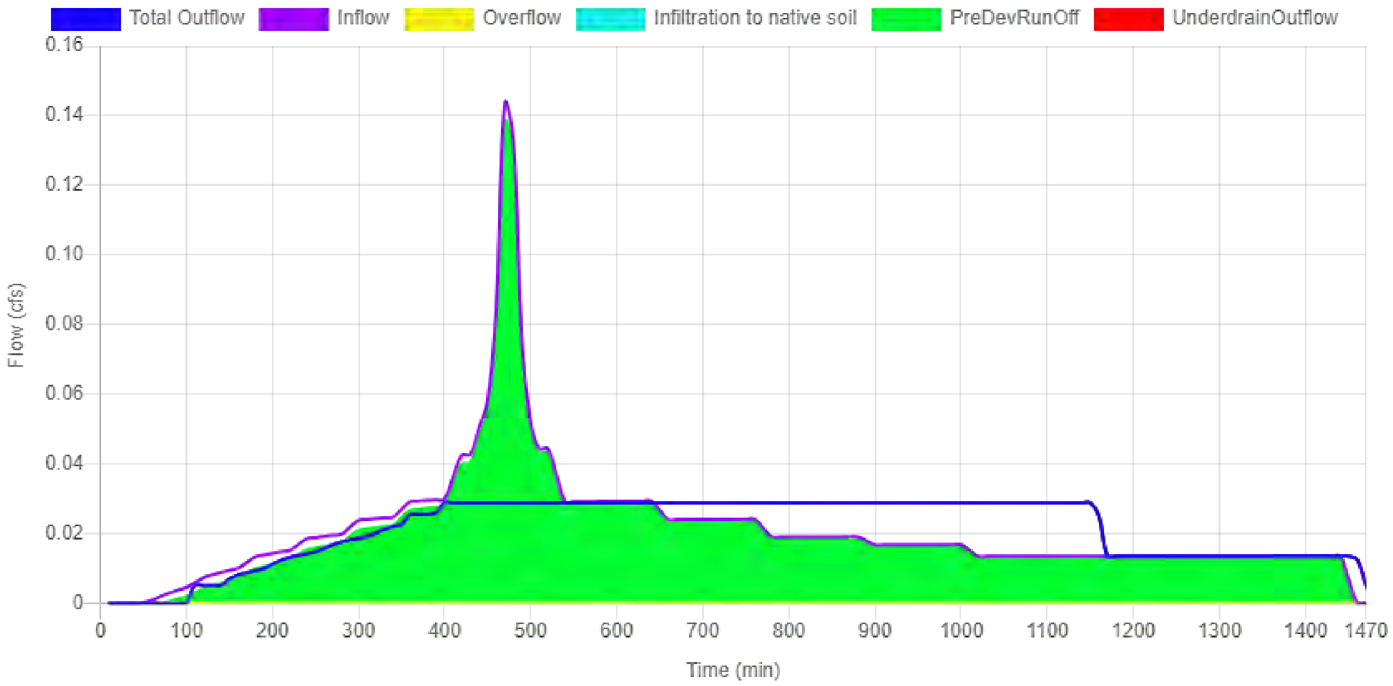
Surface Head



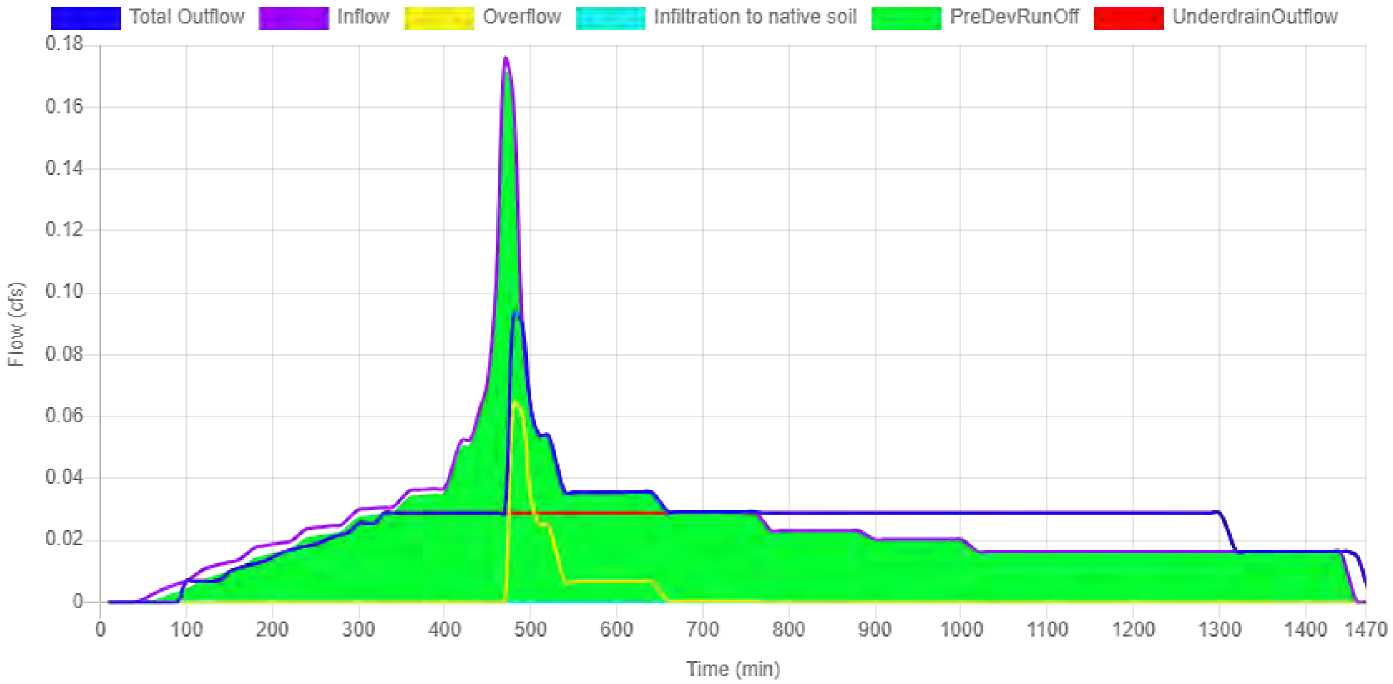
Water Quality



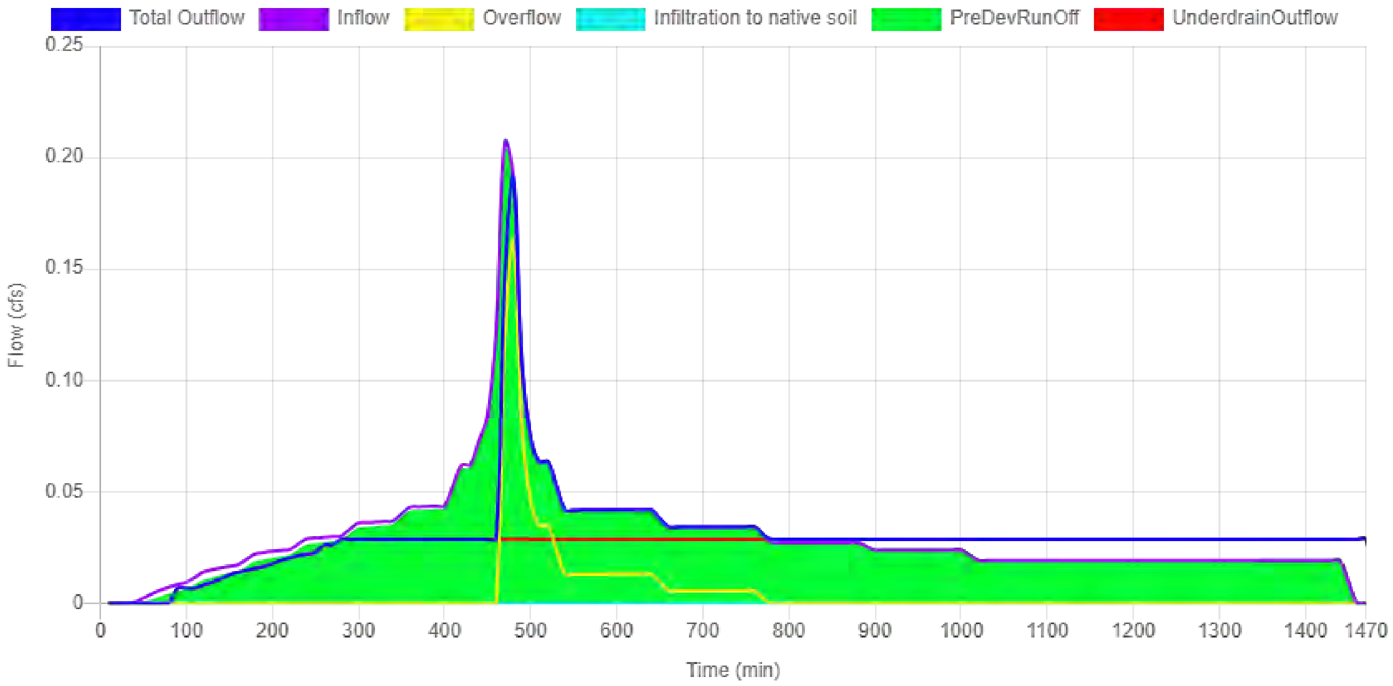
2-Year


Total Outflow
Inflow
Overflow
Infiltration to native soil
PreDevRunOff
UnderdrainOutflow
Flow (cfs)
0.16
0.14
0.12
0.10
0.08
0.06
0.04
0.02
0
0
100
200
300
400
500
600
700
800
900
1000
1100
1200
1300
1400
1470
Time (min)

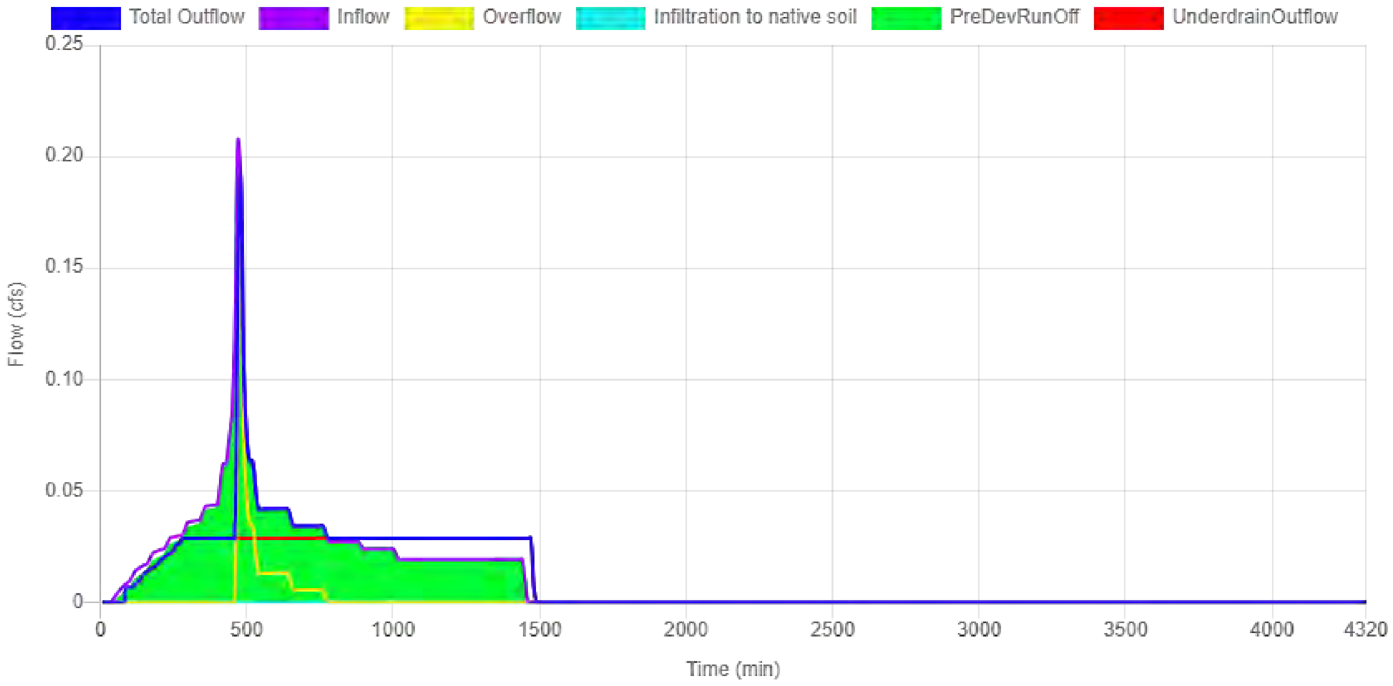
5-Year


Total Outflow
Inflow
Overflow
Infiltration to native soil
PreDevRunOff
UnderdrainOutflow
Flow (cfs)
0.18
0.16
0.14
0.12
0.10
0.08
0.06
0.04
0.02
0
0
100
200
300
400
500
600
700
800
900
1000
1100
1200
1300
1400
1470
Time (min)

10-Year



10-Year



Appendix C
PAC - Scott
Proposed Planter

PAC Report

Project Details

Project Name	Permit No	Created
Scott Park		6/6/2023 7:37:43 PM
Project Address	**Designer**	**Last Modified**
10660 SE 21st Ave		6/6/2023 10:40:44 PM
	Company	**Report Generated**
	KPFF	6/6/2023 3:47:11 PM

Project Summary

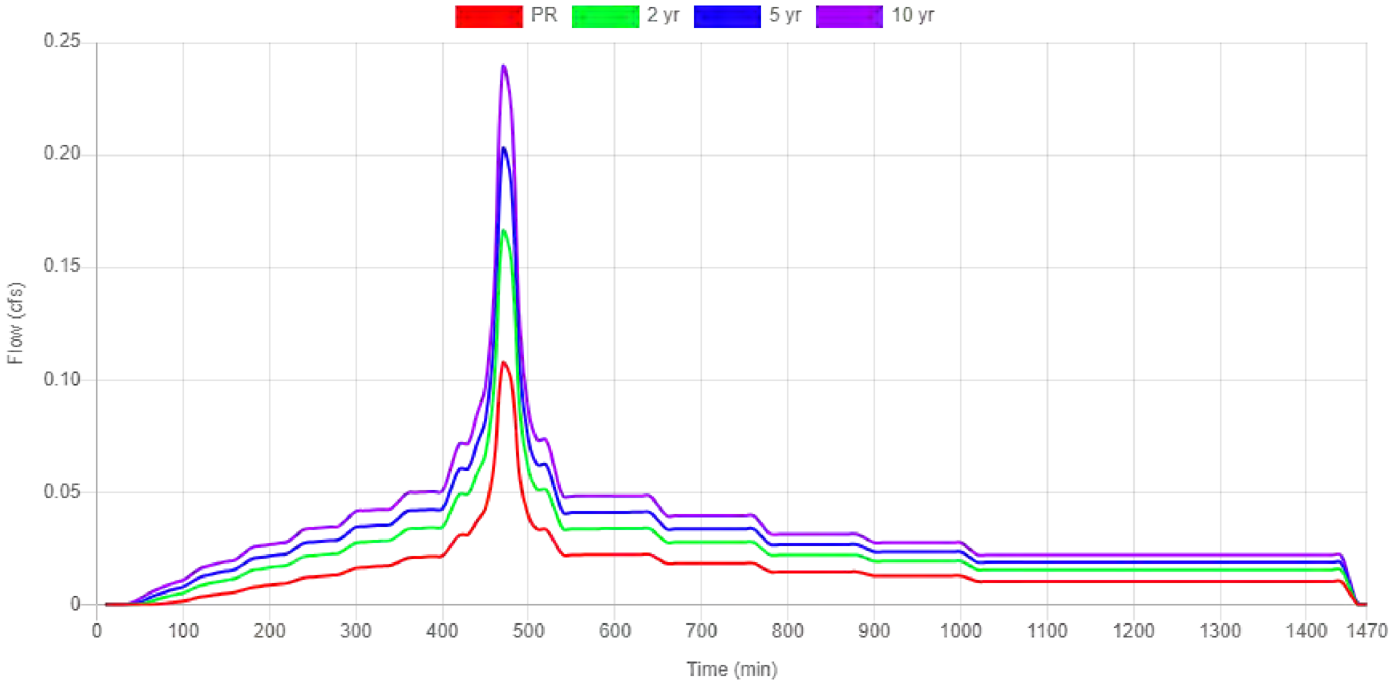
Catchment Name	Imper-vious Area (sq ft)	Native Soil Design Infilt-ration Rate (in/hr)	Level	Category	Config	Facility Area (excl. free board) (sq ft)	Facility Sizing Ratio (%)	PR Results	Infilt-ration Results	Flow Control Results
SP-1	11729	0	2C	FlatPlanter	D	350.00	2.98	Pass	NA	Pass

SP-1

Site Soils & Infiltration Testing	Infiltration Testing Procedure OpenPit Tested Native Soil Infiltration Rate 0 in/hr
Correction Factor	CF test 2
Design Infiltration Rates	Native Soil 0 in/hr Imported Blended Soil 6 in/hr
Catchment Information	Hierarchy Level 2C Hierarchy Description Base requirement for all other discharge points Pollution Reduction Requirement Filter the post-development stormwater runoff from the water quality storm event through the blended soil. Infiltration Requirement N/A Flow Control Requirement Limit the 2-yr, the 5-yr, and the 10-yr post-development peak flows to their respective pre-development peak flows. Impervious Area 11729 sq ft 0.269 acre Pre-Development Time of Concentration (Tc pre) 5 min Post-Development Time of Concentration (Tc post) 5 min Pre-Development Curve Number (CN pre) 97 Post-Development Curve Number (CN post) 98

SBUH Results

Post-Development Runoff



	Pre - Development Rate and Volume		Post - Development Rate and Volume	
	Peak Rate (cfs)	**Total Volume (cf)**	**Peak Rate (cfs)**	**Total Volume (cf)**
PR	0.1	1261.2	0.1071	1357.2
2-Year	0.1594	2018.4	0.1656	2122.3
5-Year	0.1966	2501.5	0.2023	2608.4
10-Year	0.2335	2986.1	0.2387	3095.3

	Overflow		Underdrain Outflow		Infiltration	
	Peak Rate (cfs)	**Total Volume (cf)**	**Peak Rate (cfs)**	**Total Volume (cf)**	**Peak Rate (cfs)**	**Total Volume (cf)**
PR	0	0	0.031	1353.9	0	0
2-Year	0.003	17.3	0.03	2101.6	0	0
5-Year	0.076	257.8	0.031	2347.3	0	0
10-Year	0.19	551.1	0.031	2540.8	0	0

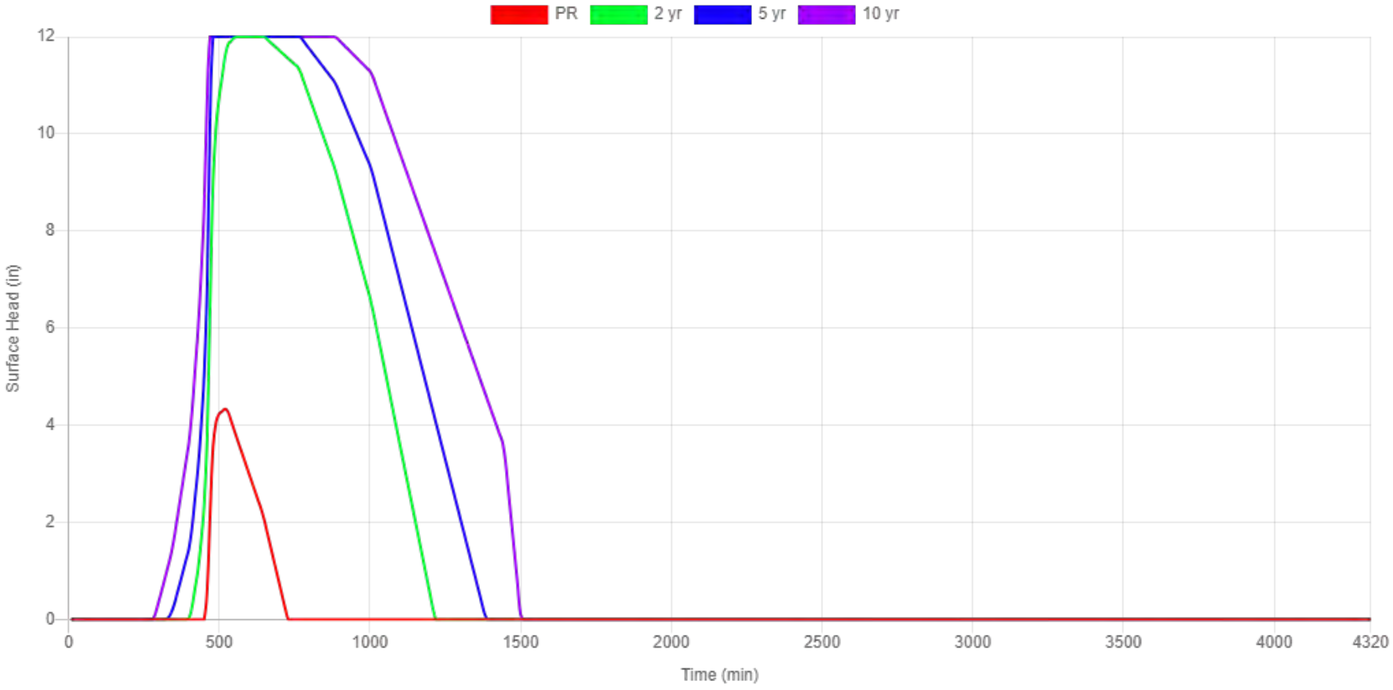
Flat Planter

Site Soils & Infiltration Testing	Category Flat Planter Shape Null Location Parcel Configuration D: Lined Facility with RS and Ud **Above Grade Storage Data** Bottom Area 350 sq ft Bottom Width 10 ft Overflow Height 12 in Total Depth of Blended Soil plus Rock 28 in Surface Storage Capacity at Overflow 350 cu ft Design Infiltration Rate to Soil Underlying the Facility 0.000 cfs Design Infiltration Rate for Imported Blended Soil in the Facility 0.049 cfs **Below Grade Storage Data** Catchment is too small for flow control? No Rock Area 25.00 sq ft Rock Width 3.00 ft Rock Storage Depth 10.0 in Rock Porosity 0.3 Underdrain Height

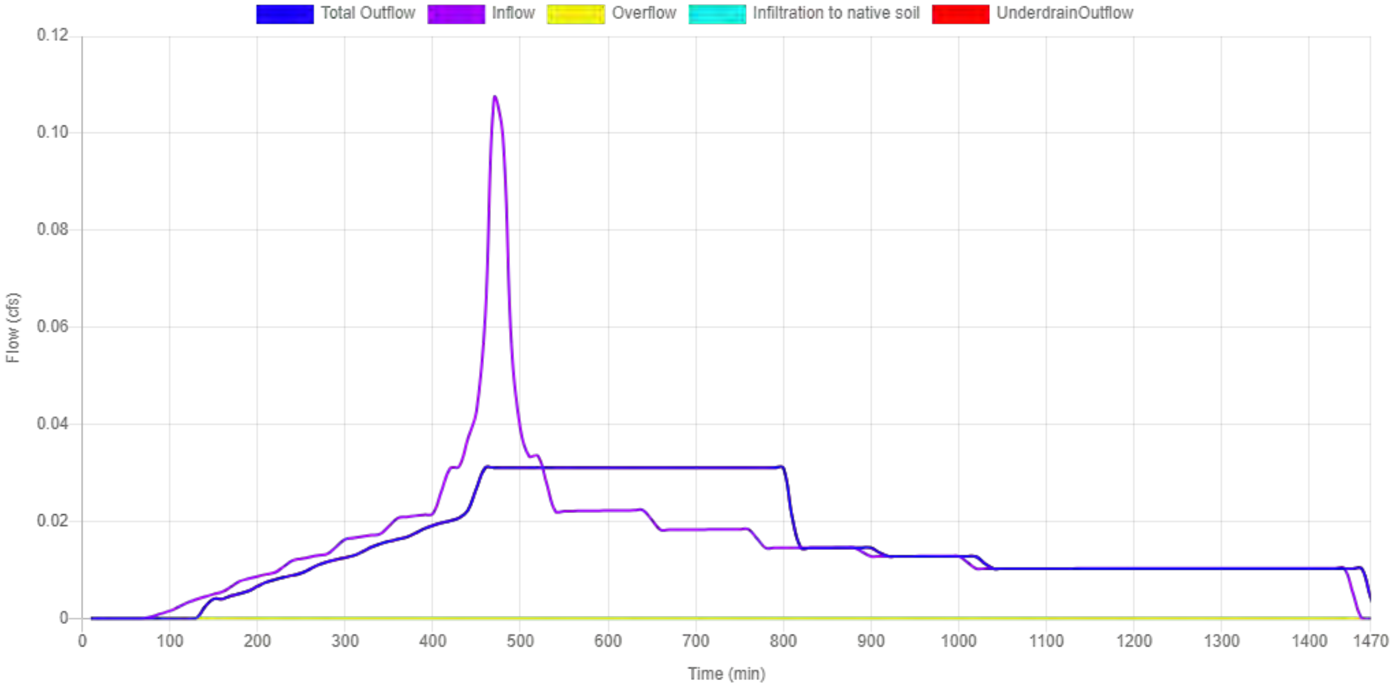
	1.0 in Percent of Facility Base that Allows Infiltration 0 % Orifice (Y/N)? Yes Orifice Diameter 1.000 in
Facility Facts	Total Facility Area (excluding freeboard) 350.00 sq ft Sizing Ratio 2.98 %
Pollution Reduction Results	Pollution Reduction Score Pass Overflow Volume 0.00 cf Surface Capacity Used 36.13 %
Flow Control Results	Flow Control Score Pass

	STORMWATER FACILITY OUTFLOW (CFS)		PRE-DEVELOPMENT RUNOFF (CFS)
2 year	0.0338	<=	0.1594
5 year	0.1066	<=	0.1966
10 year	0.2203	<=	0.2335

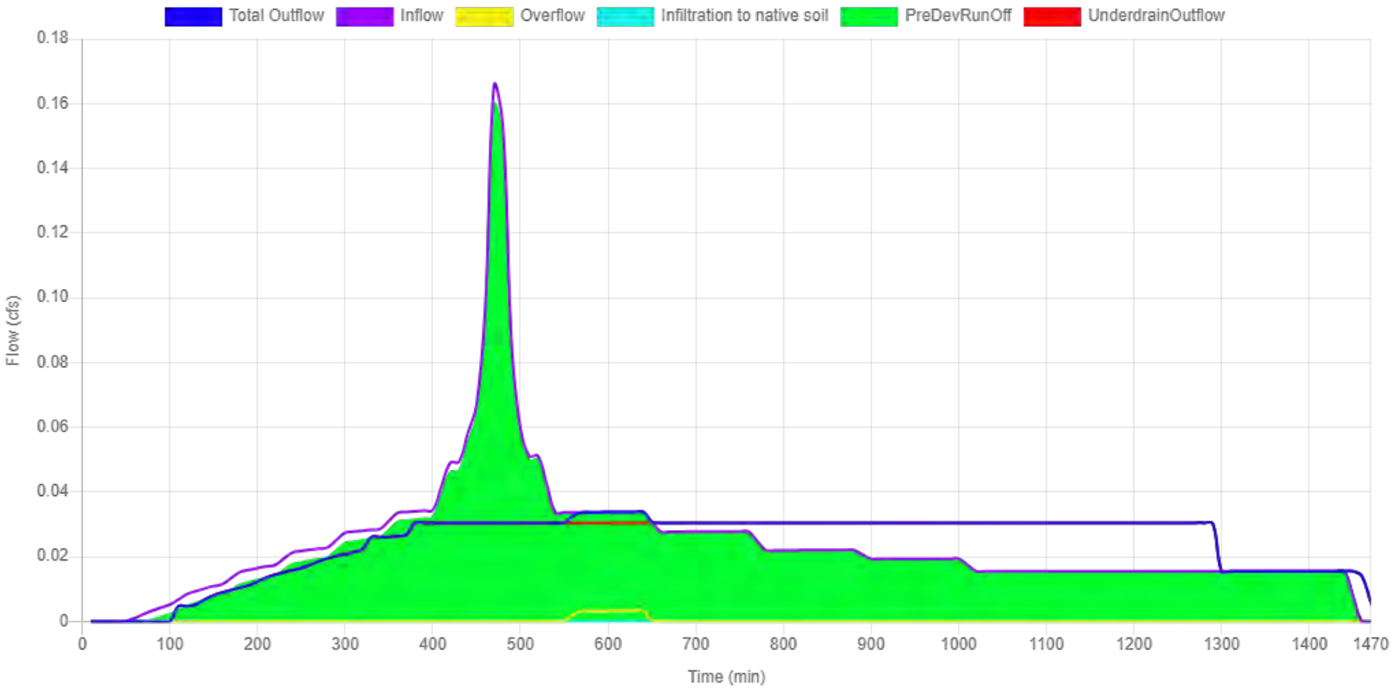
Surface Head



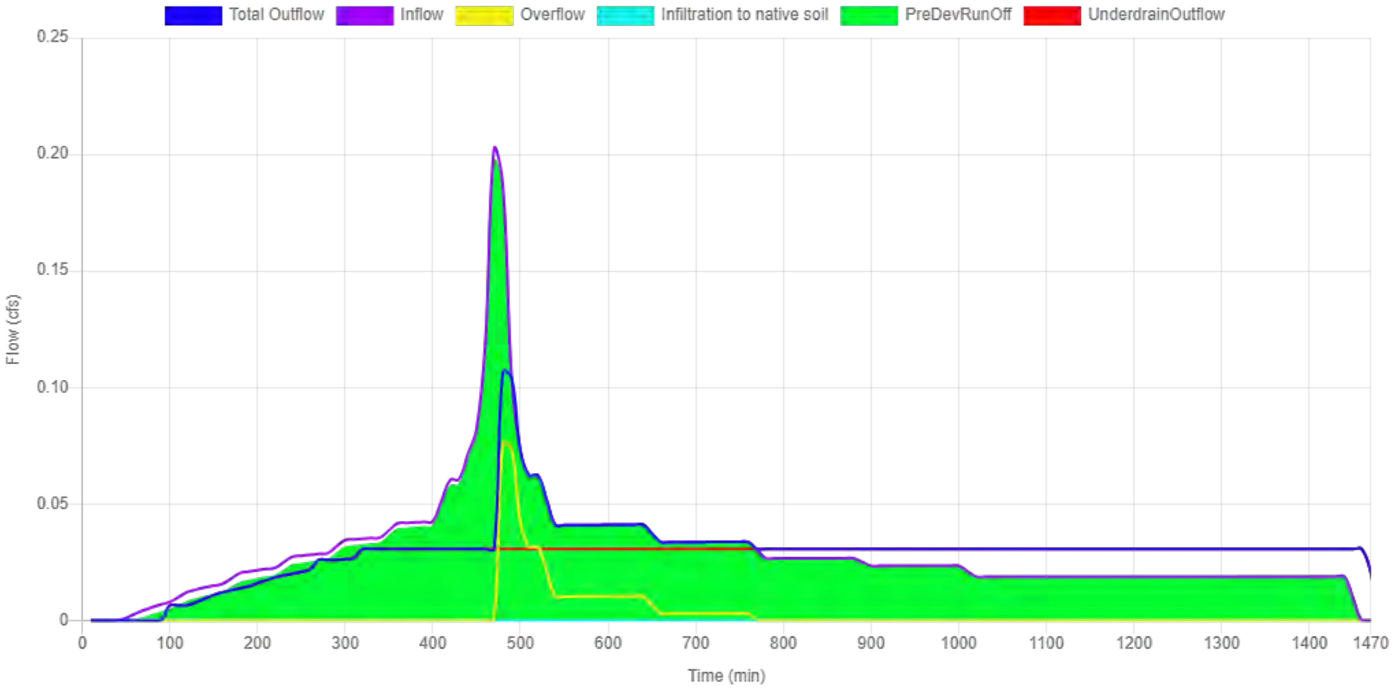
Water Quality



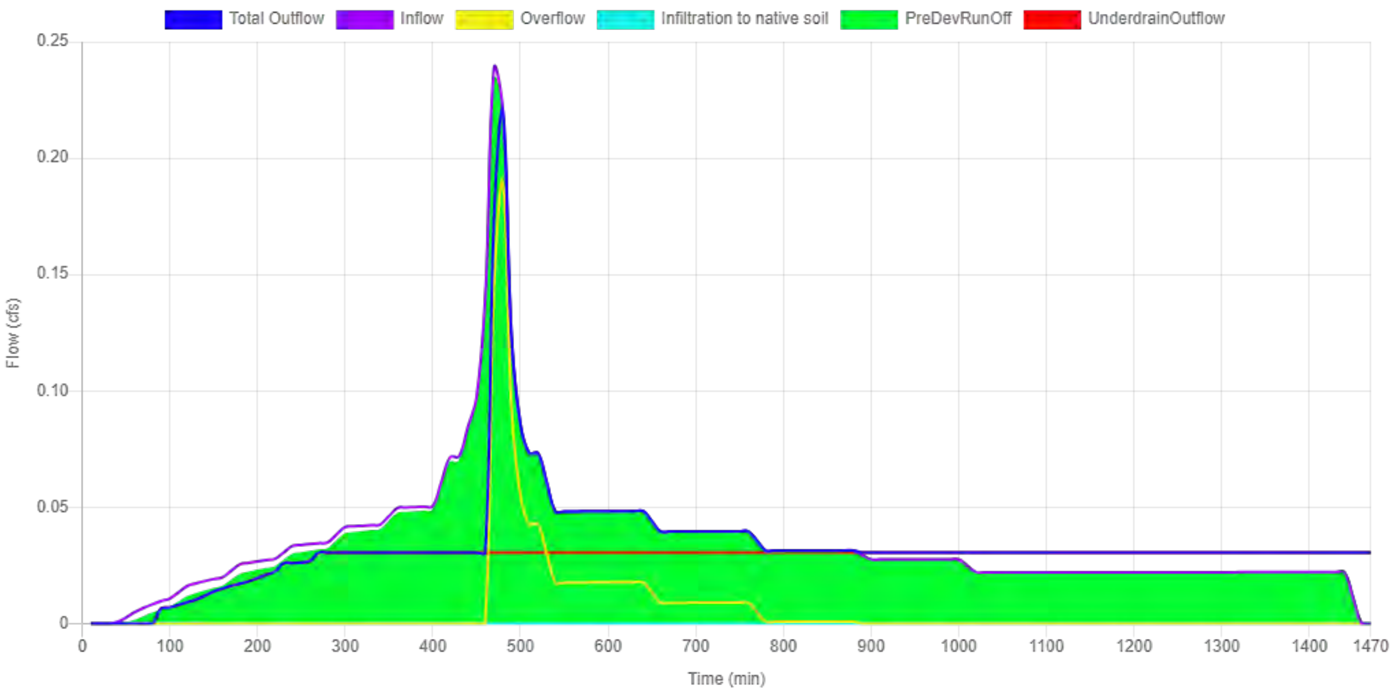
2-Year


Total Outflow
Inflow
Overflow
Infiltration to native soil
PreDevRunOff
UnderdrainOutflow
0.18
0.16
0.14
0.12
0.10
0.08
0.06
0.04
0.02
0
Flow (cfs)
0
100
200
300
400
500
600
700
800
900
1000
1100
1200
1300
1400
1470
Time (min)

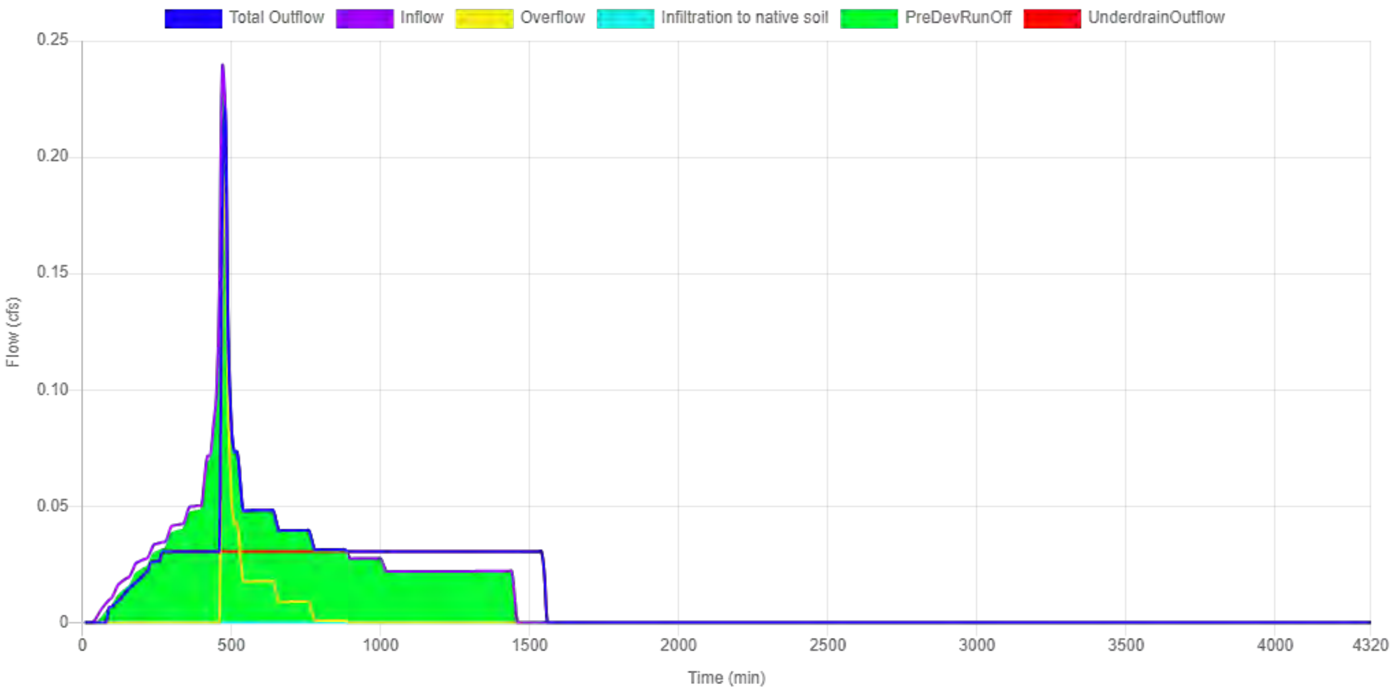
5-Year


Total Outflow
Inflow
Overflow
Infiltration to native soil
PreDevRunOff
UnderdrainOutflow
0.25
0.20
0.15
0.10
0.05
0
Flow (cfs)
0
100
200
300
400
500
600
700
800
900
1000
1100
1200
1300
1400
1470
Time (min)

10-Year



10-Year



PAC Report

Project Details

Project Name	Permit No	Created
Bowman Brae Park		6/5/2023 11:02:08 PM
Project Address	**Designer**	**Last Modified**
4267 SE Bowman St		6/6/2023 9:52:39 PM
	Company	**Report Generated**
	KPFF	6/6/2023 2:59:07 PM

Project Summary

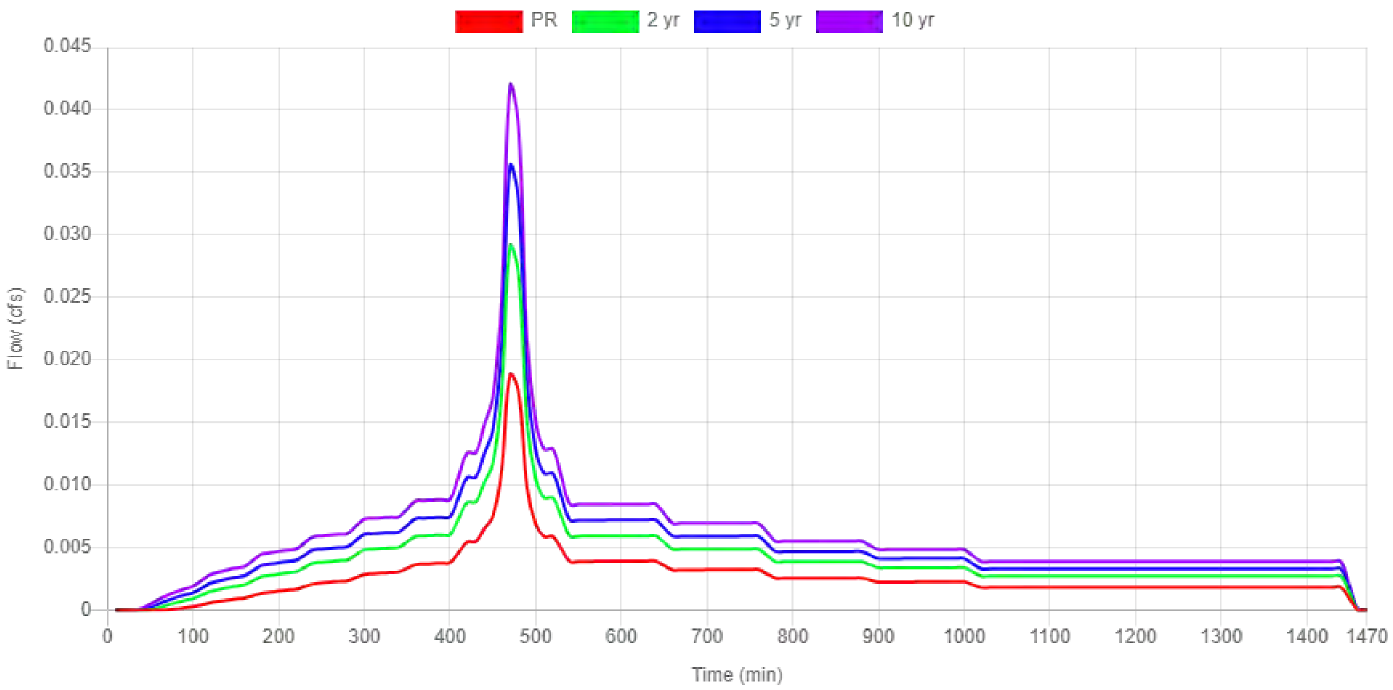
Catchment Name	Impervious Area (sq ft)	Native Soil Design Infiltration Rate (in/hr)	Level	Category	Config	Facility Area (excl. free board) (sq ft)	Facility Sizing Ratio (%)	PR Results	Infiltration Results	Flow Control Results
SP-2	2059	0	2C	Basin	D	277.50	13.48	Pass	NA	Fail

SP-2

Site Soils & Infiltration Testing	Infiltration Testing Procedure OpenPit Tested Native Soil Infiltration Rate 0 in/hr
Correction Factor	CF_{test} 2
Design Infiltration Rates	Native Soil 0 in/hr Imported Blended Soil 6 in/hr
Catchment Information	Hierarchy Level 2C Hierarchy Description Base requirement for all other discharge points Pollution Reduction Requirement Filter the post-development stormwater runoff from the water quality storm event through the blended soil. Infiltration Requirement N/A Flow Control Requirement Limit the 2-yr, the 5-yr, and the 10-yr post-development peak flows to their respective pre-development peak flows. Impervious Area 2059 sq ft 0.047 acre Pre-Development Time of Concentration (Tc_{pre}) 5 min Post-Development Time of Concentration (Tc_{post}) 5 min Pre-Development Curve Number (CN_{pre}) 74 Post-Development Curve Number (CN_{post}) 98

SBUH Results

Post-Development Runoff



	Pre - Development Rate and Volume		**Post - Development Rate and Volume**	
	Peak Rate (cfs)	**Total Volume (cf)**	**Peak Rate (cfs)**	**Total Volume (cf)**
PR	0.0007	32	0.0188	238.3
2-Year	0.0038	94.9	0.0291	372.6
5-Year	0.0075	145.1	0.0355	457.9
10-Year	0.0116	201	0.0419	543.4

	Overflow		**Underdrain Outflow**		**Infiltration**	
	Peak Rate (cfs)	**Total Volume (cf)**	**Peak Rate (cfs)**	**Total Volume (cf)**	**Peak Rate (cfs)**	**Total Volume (cf)**
PR	0	0	0.015	233.3	0	0
2-Year	0	0	0.026	367.6	0	0
5-Year	0	0	0.03	452.9	0	0
10-Year	0	0	0.029	538.4	0	0

Appendix C
PAC - Bowman-Brae
1-inch Orifice

Amoeba Basin

Site Soils & Infiltration Testing	Category Amoeba Basin Shape Amoeba Location Parcel Configuration D: Lined Facility with RS and Ud **Above Grade Storage Data** Bottom Area 165 sq ft Bottom Perimeter Length 75.00 ft Side Slope 3.0 h:1v Freeboard Depth 6.0 in Overflow Height 6.0 in Total Depth of Blended Soil plus Rock 24 in Surface Storage Capacity at Overflow 110.62 cu ft Design Infiltration Rate to Soil Underlying the Facility 0.000 cfs Design Infiltration Rate for Imported Blended Soil in the Facility 0.035 cfs **Below Grade Storage Data** Catchment is too small for flow control? No Rock Area 10.00 sq ft Rock Width 3.00 ft Rock Storage Depth

8.0 in

Rock Porosity

0.3

Underdrain Height

2.0 in

Percent of Facility Base that Allows Infiltration

0 %

Orifice (Y/N)?

Yes

Orifice Diameter

1.000 in

Facility Facts

Total Facility Area (excluding freeboard)

277.50 sq ft

Sizing Ratio

13.48 %

Pollution Reduction Results

Pollution Reduction Score

Pass

Overflow Volume

0.00 cf

Surface Capacity Used

0.00 %

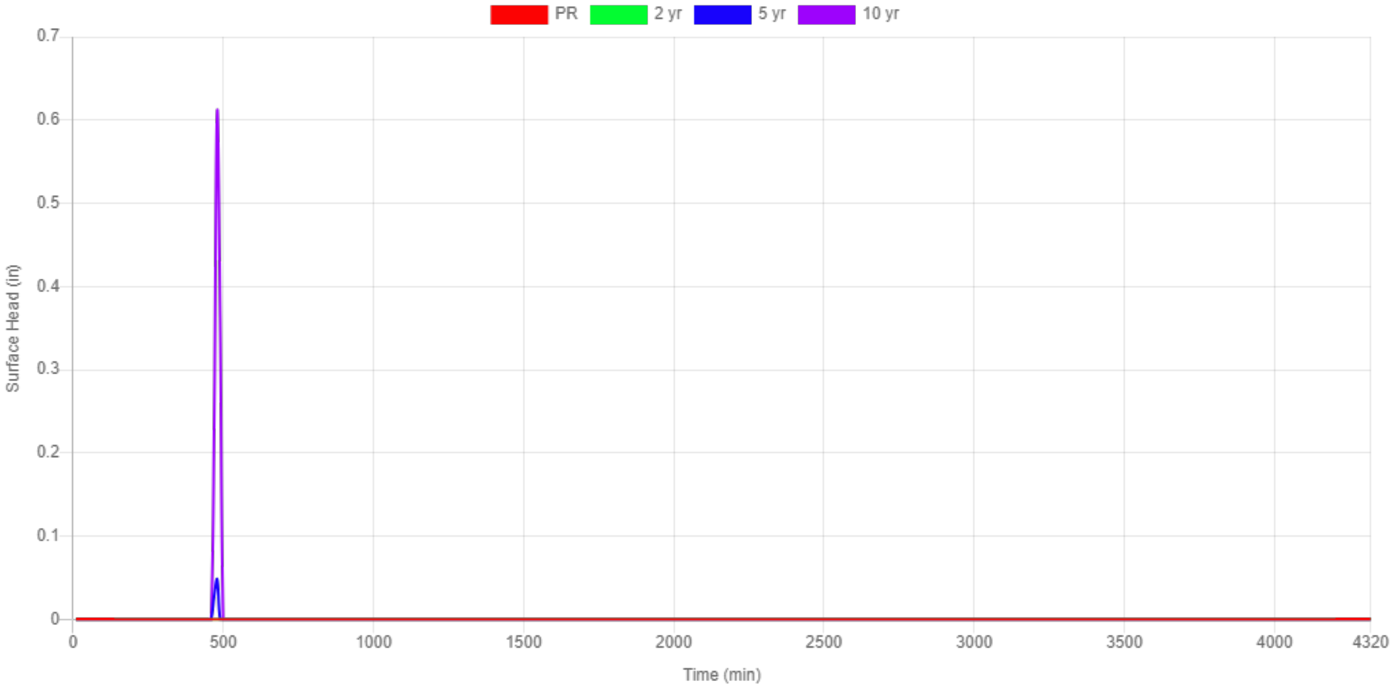
Flow Control Results

Flow Control Score

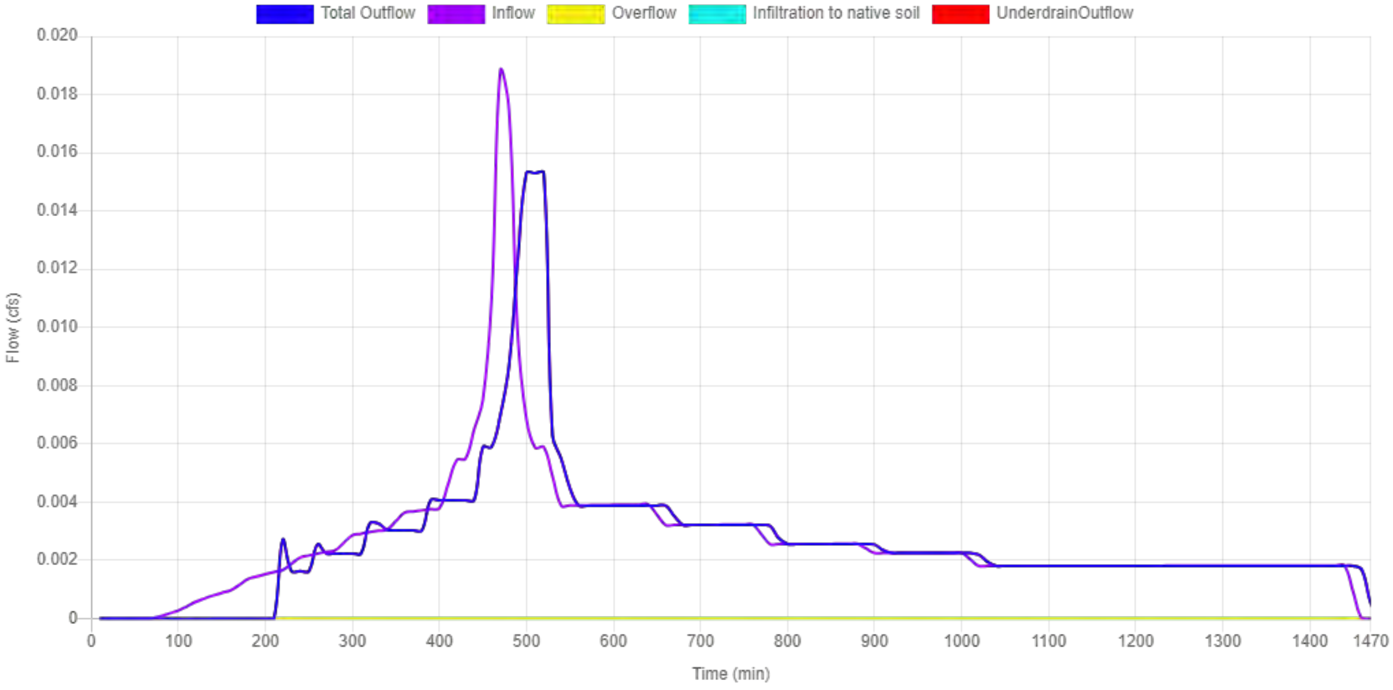
Fail

	STORMWATER FACILITY OUTFLOW (CFS)		PRE-DEVELOPMENT RUNOFF (CFS)
2 year	0.0259	<=	0.0038
5 year	0.0296	<=	0.0075
10 year	0.0292	<=	0.0116

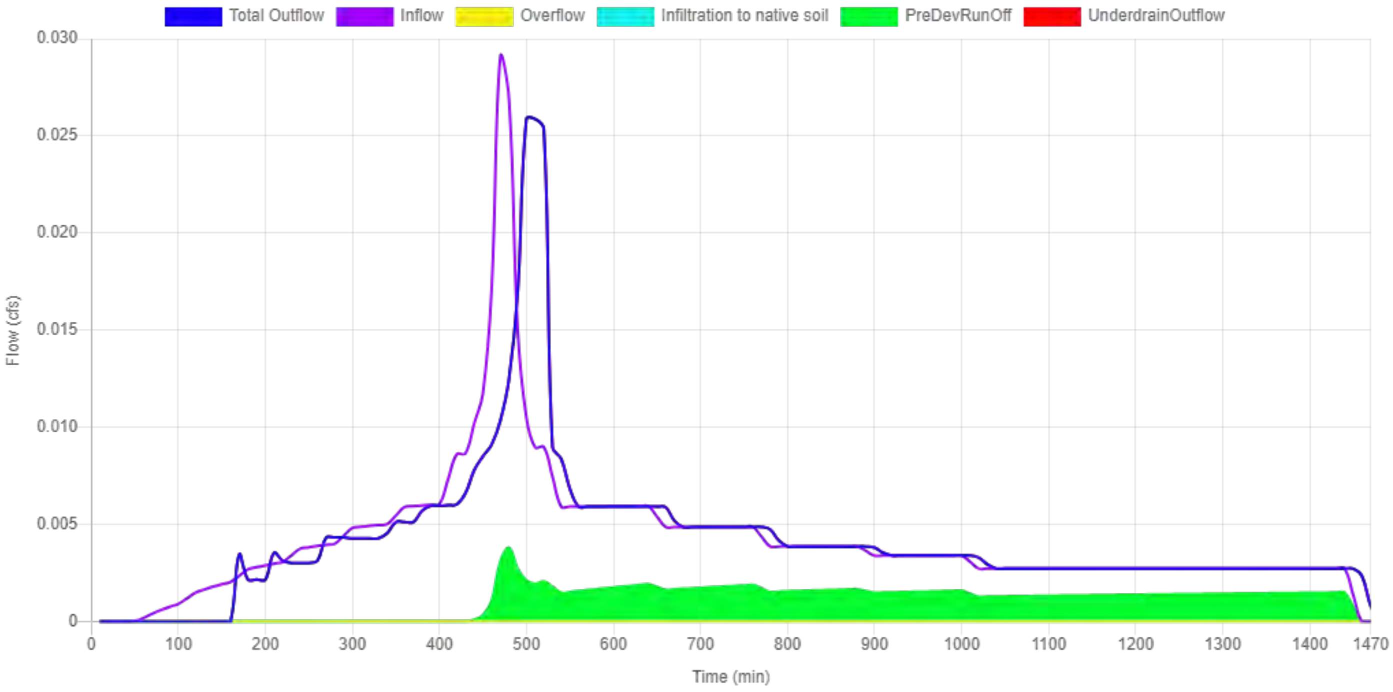
Surface Head



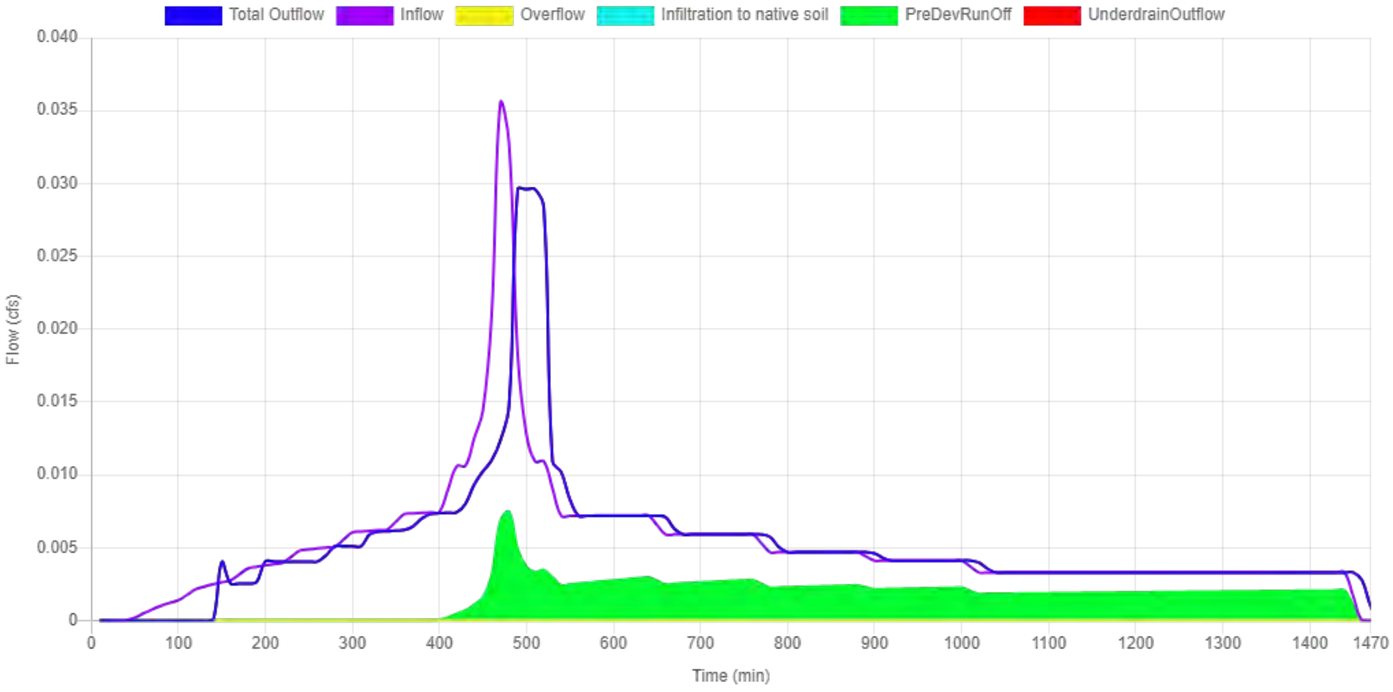
Water Quality



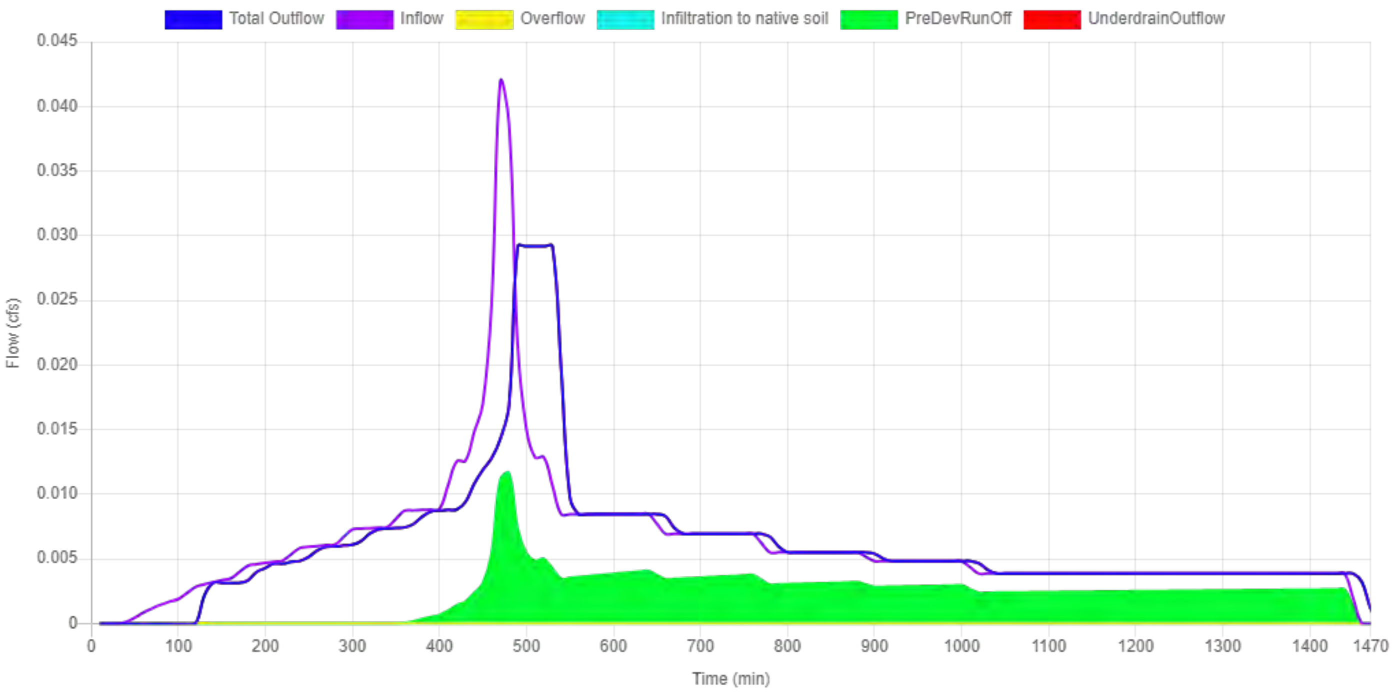
2-Year


Total Outflow
Inflow
Overflow
Infiltration to native soil
PreDevRunOff
UnderdrainOutflow
Flow (cfs)
0.030
0.025
0.020
0.015
0.010
0.005
0
0
100
200
300
400
500
600
700
800
900
1000
1100
1200
1300
1400
1470
Time (min)

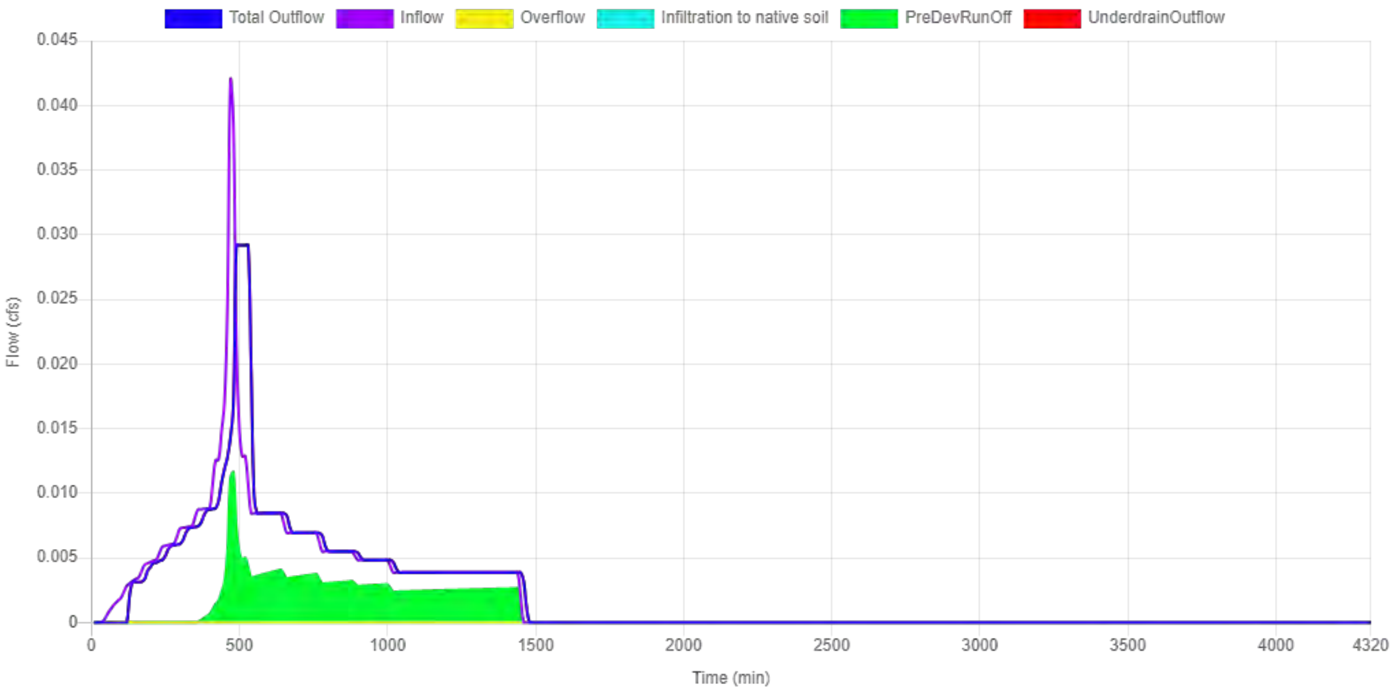
5-Year


Total Outflow
Inflow
Overflow
Infiltration to native soil
PreDevRunOff
UnderdrainOutflow
Flow (cfs)
0.040
0.035
0.030
0.025
0.020
0.015
0.010
0.005
0
0
100
200
300
400
500
600
700
800
900
1000
1100
1200
1300
1400
1470
Time (min)

10-Year



10-Year



PAC Report

Project Details

Project Name	Permit No	Created
Bowman Brae Park		6/5/2023 11:02:08 PM
Project Address	**Designer**	**Last Modified**
4267 SE Bowman St		6/6/2023 7:14:35 PM
	Company	**Report Generated**
	KPFF	6/6/2023 2:52:51 PM

Project Summary

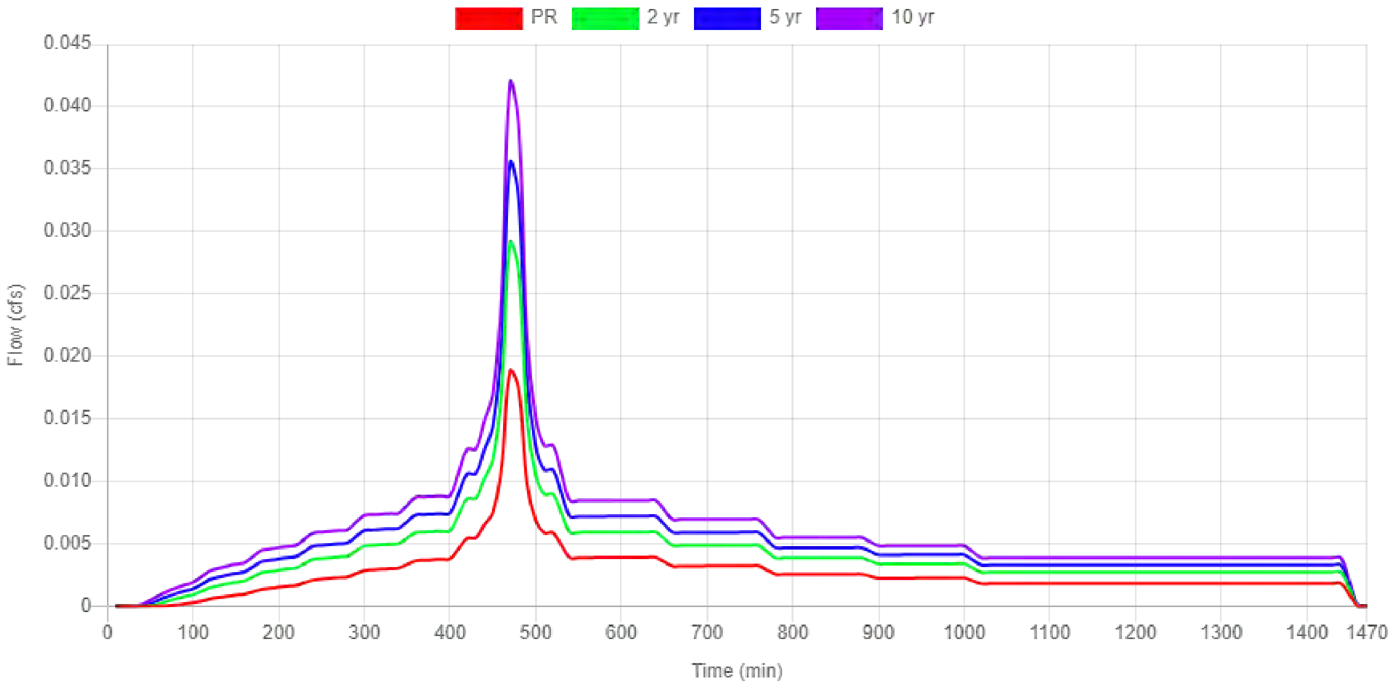
Catchment Name	Impervious Area (sq ft)	Native Soil Design Infiltration Rate (in/hr)	Level	Category	Config	Facility Area (excl. free board) (sq ft)	Facility Sizing Ratio (%)	PR Results	Infiltration Results	Flow Control Results
SP-1	2059	0	2C	Basin	D	277.50	13.48	Pass	NA	Fail

SP-1

Site Soils & Infiltration Testing	Infiltration Testing Procedure OpenPit Tested Native Soil Infiltration Rate 0 in/hr
Correction Factor	CF test 2
Design Infiltration Rates	Native Soil 0 in/hr Imported Blended Soil 6 in/hr
Catchment Information	Hierarchy Level 2C Hierarchy Description Base requirement for all other discharge points Pollution Reduction Requirement Filter the post-development stormwater runoff from the water quality storm event through the blended soil. Infiltration Requirement N/A Flow Control Requirement Limit the 2-yr, the 5-yr, and the 10-yr post-development peak flows to their respective pre-development peak flows. Impervious Area 2059 sq ft 0.047 acre Pre-Development Time of Concentration (Tc pre) 5 min Post-Development Time of Concentration (Tc post) 5 min Pre-Development Curve Number (CN pre) 74 Post-Development Curve Number (CN post) 98

SBUH Results

Post-Development Runoff



	Pre - Development Rate and Volume		**Post - Development Rate and Volume**	
	Peak Rate (cfs)	**Total Volume (cf)**	**Peak Rate (cfs)**	**Total Volume (cf)**
PR	0.0007	32	0.0188	238.3
2-Year	0.0038	94.9	0.0291	372.6
5-Year	0.0075	145.1	0.0355	457.9
10-Year	0.0116	201	0.0419	543.4

	Overflow		**Underdrain Outflow**		**Infiltration**	
	Peak Rate (cfs)	**Total Volume (cf)**	**Peak Rate (cfs)**	**Total Volume (cf)**	**Peak Rate (cfs)**	**Total Volume (cf)**
PR	0	0	0.005	233.3	0	0
2-Year	0	0	0.005	367.6	0	0
5-Year	0	0	0.005	452.9	0	0
10-Year	0.005	37.8	0.005	500.6	0	0

Amoeba Basin

Site Soils & Infiltration Testing

Category
Amoeba Basin

Shape
Amoeba

Location
Parcel

Configuration
D: Lined Facility with RS and Ud

Above Grade Storage Data

Bottom Area
165 sq ft

Bottom Perimeter Length
75.00 ft

Side Slope
3.0 h:1v

Freeboard Depth
6.0 in

Overflow Height
6.0 in

Total Depth of Blended Soil plus Rock
24 in

Surface Storage Capacity at Overflow
110.62 cu ft

Design Infiltration Rate to Soil Underlying the Facility
0.000 cfs

Design Infiltration Rate for Imported Blended Soil in the Facility
0.035 cfs

Below Grade Storage Data

Catchment is too small for flow control?
No

Rock Area
10.00 sq ft

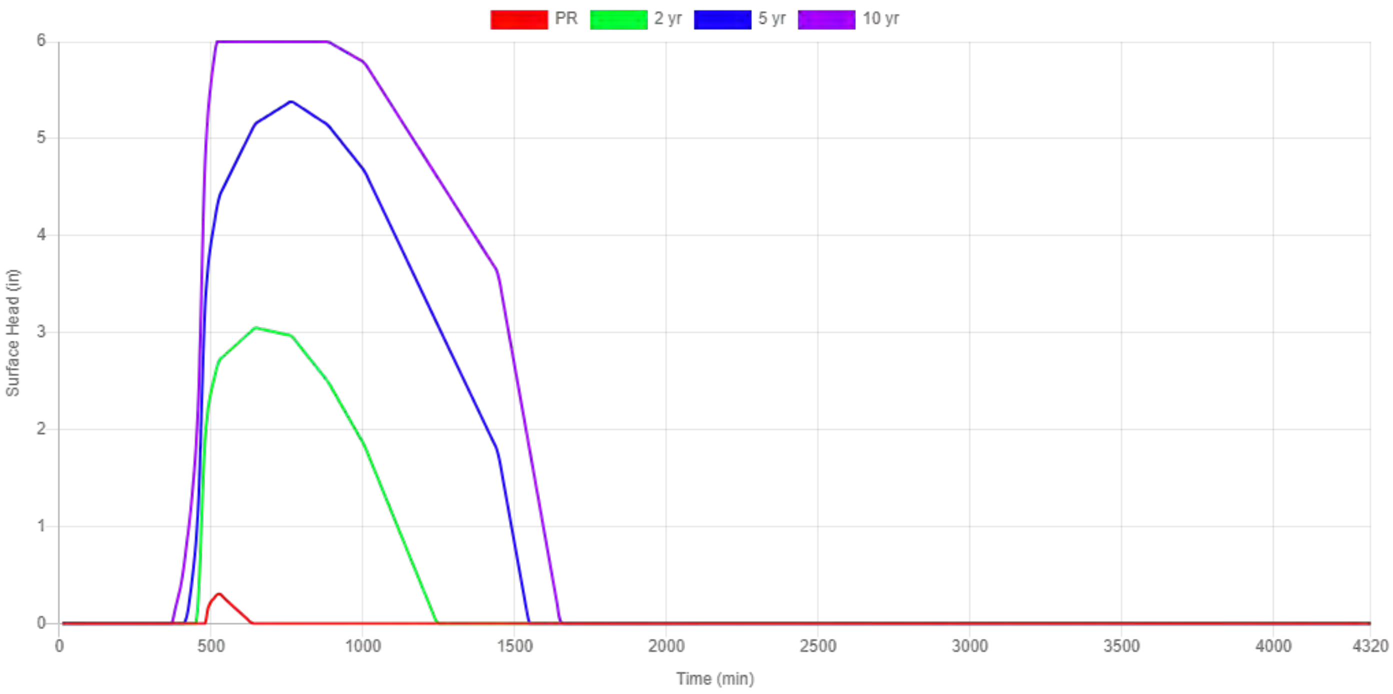
Rock Width
3.00 ft

Rock Storage Depth

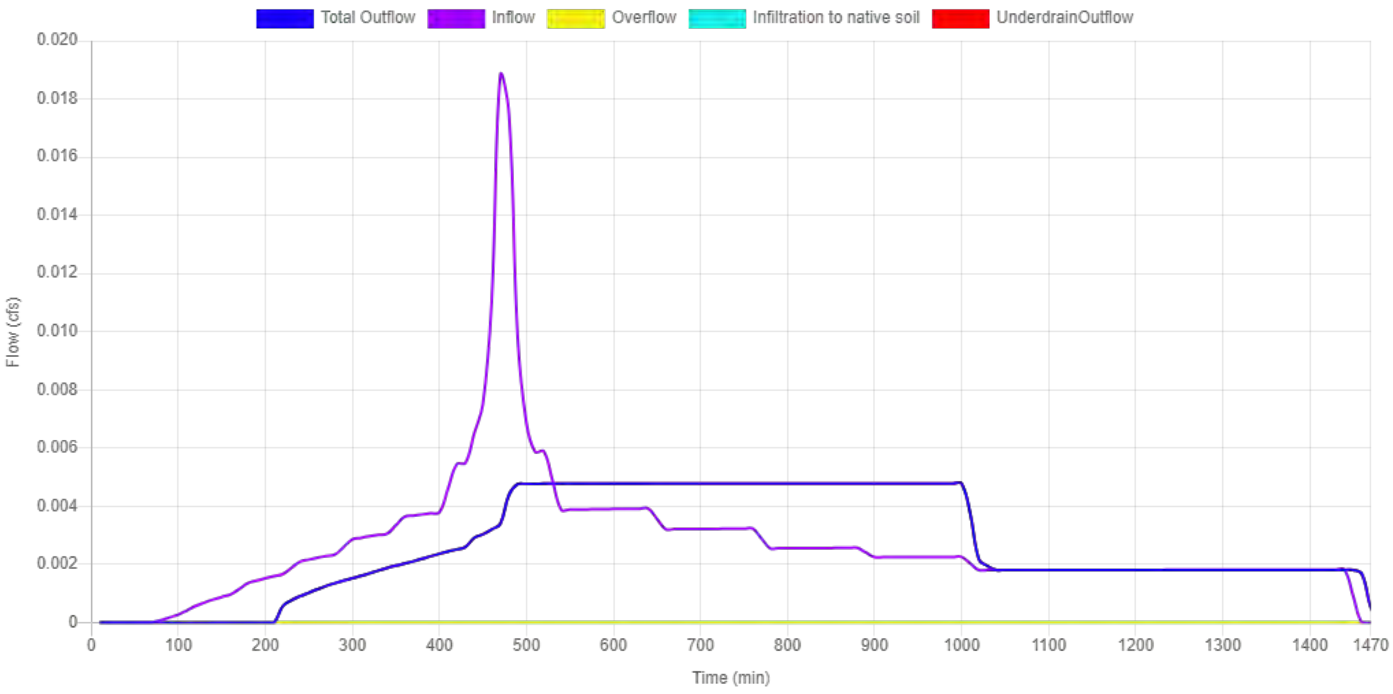
	8.0 in Rock Porosity 0.3 Underdrain Height 2.0 in Percent of Facility Base that Allows Infiltration 0 % Orifice (Y/N)? Yes Orifice Diameter 0.375 in
Facility Facts	Total Facility Area (excluding freeboard) 277.50 sq ft Sizing Ratio 13.48 %
Pollution Reduction Results	Pollution Reduction Score Pass Overflow Volume 0.00 cf Surface Capacity Used 5.03 %
Flow Control Results	Flow Control Score Fail

	STORMWATER FACILITY OUTFLOW (CFS)		PRE-DEVELOPMENT RUNOFF (CFS)
2 year	0.0051	<=	0.0038
5 year	0.0053	<=	0.0075
10 year	0.0107	<=	0.0116

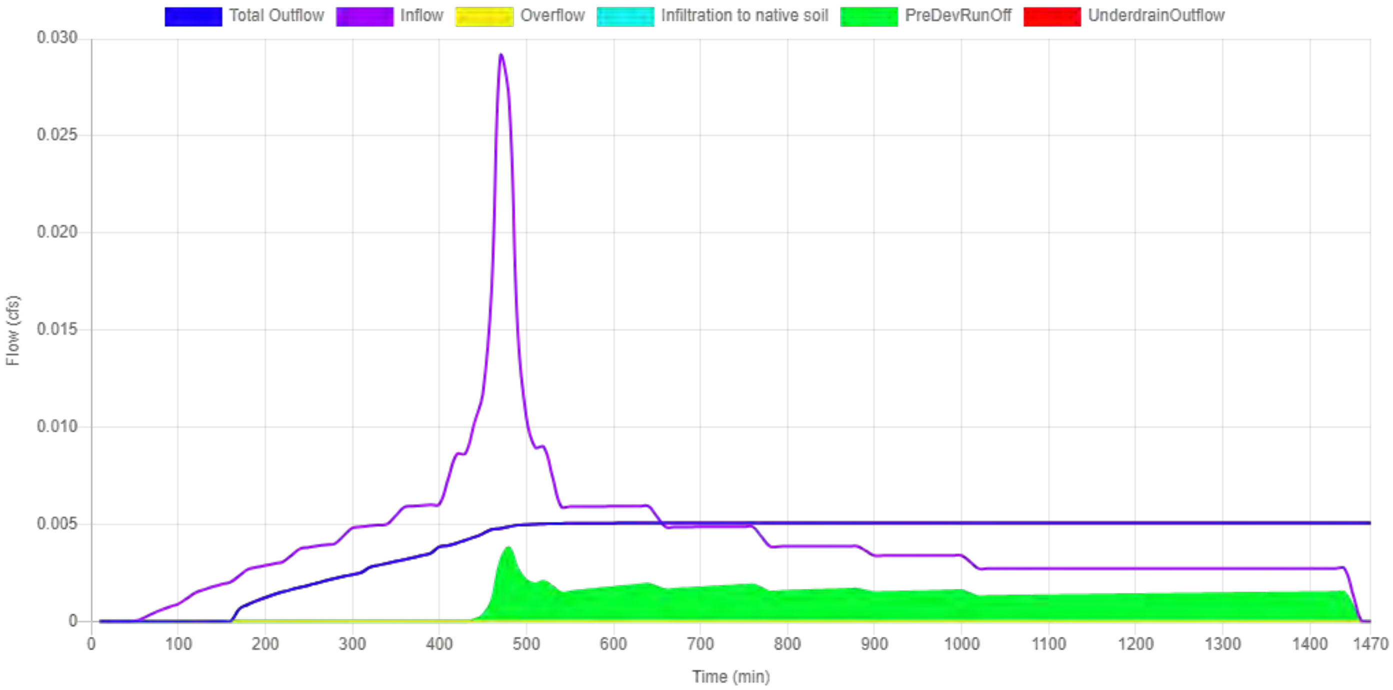
Surface Head



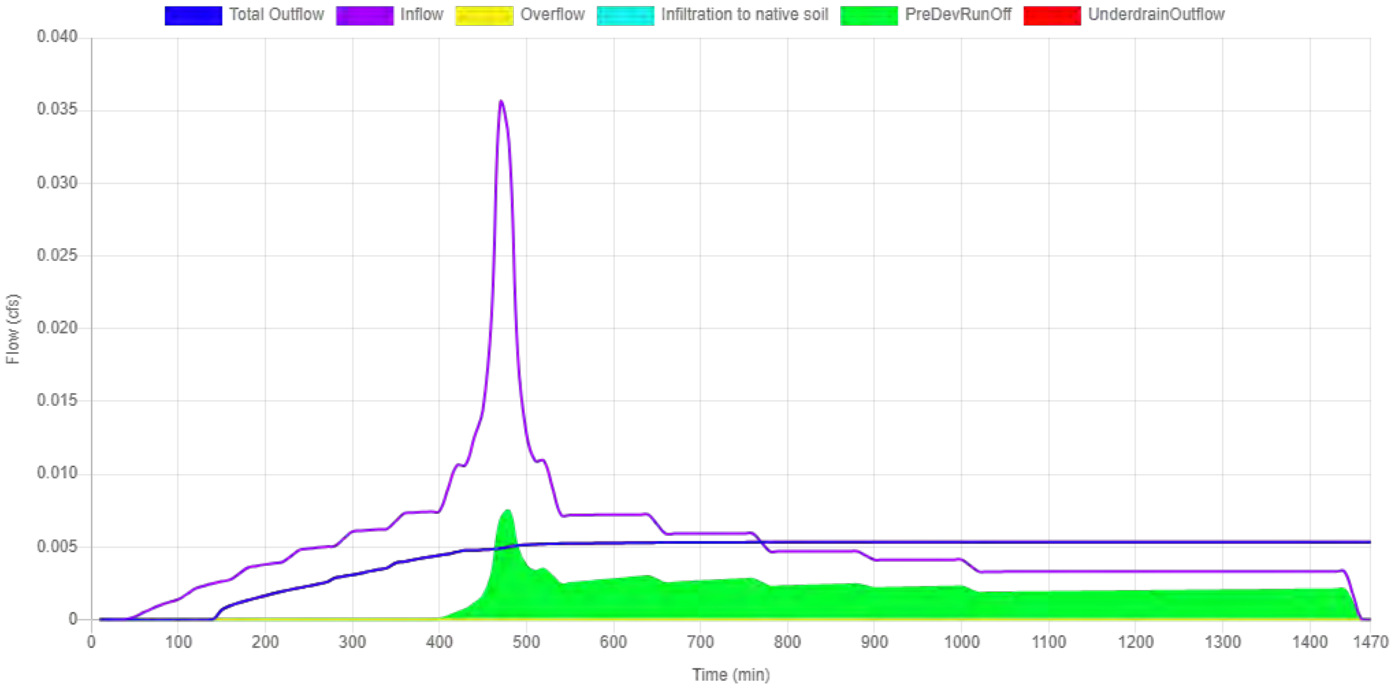
Water Quality



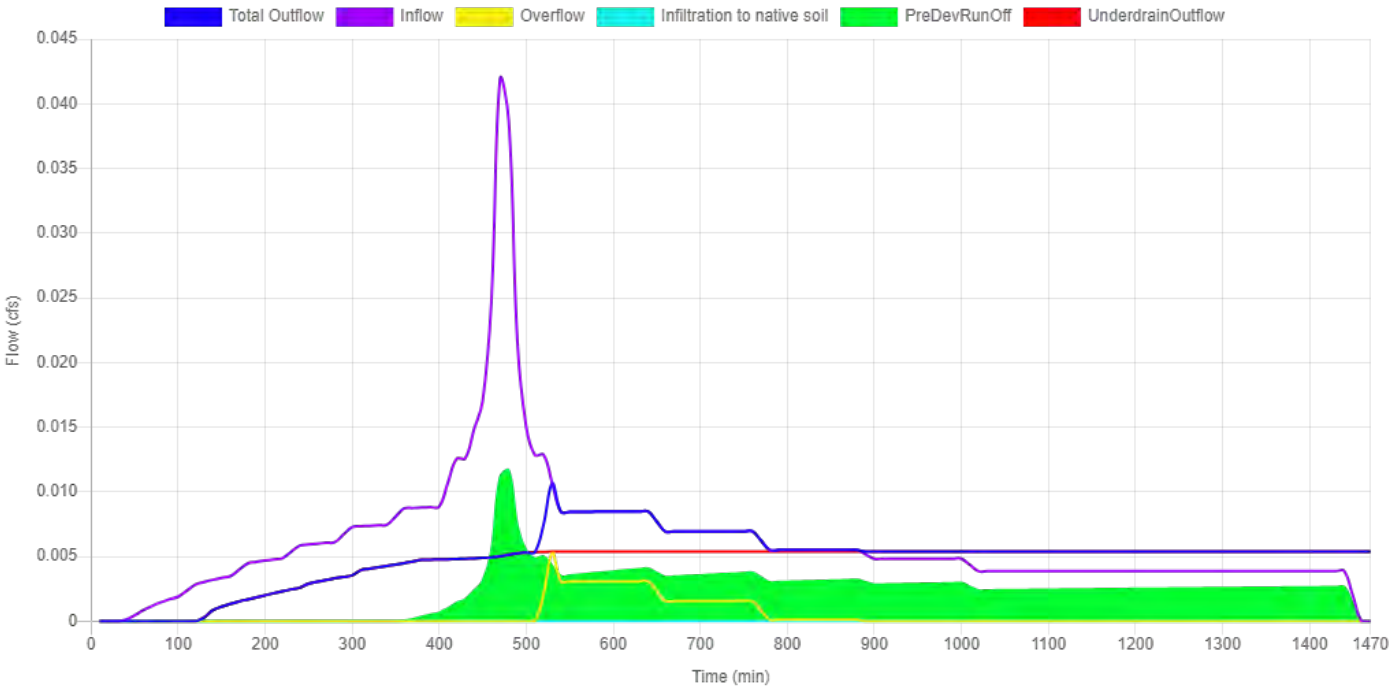
2-Year


Total Outflow
Inflow
Overflow
Infiltration to native soil
PreDevRunOff
UnderdrainOutflow
Flow (cfs)
0.030
0.025
0.020
0.015
0.010
0.005
0
0
100
200
300
400
500
600
700
800
900
1000
1100
1200
1300
1400
1470
Time (min)

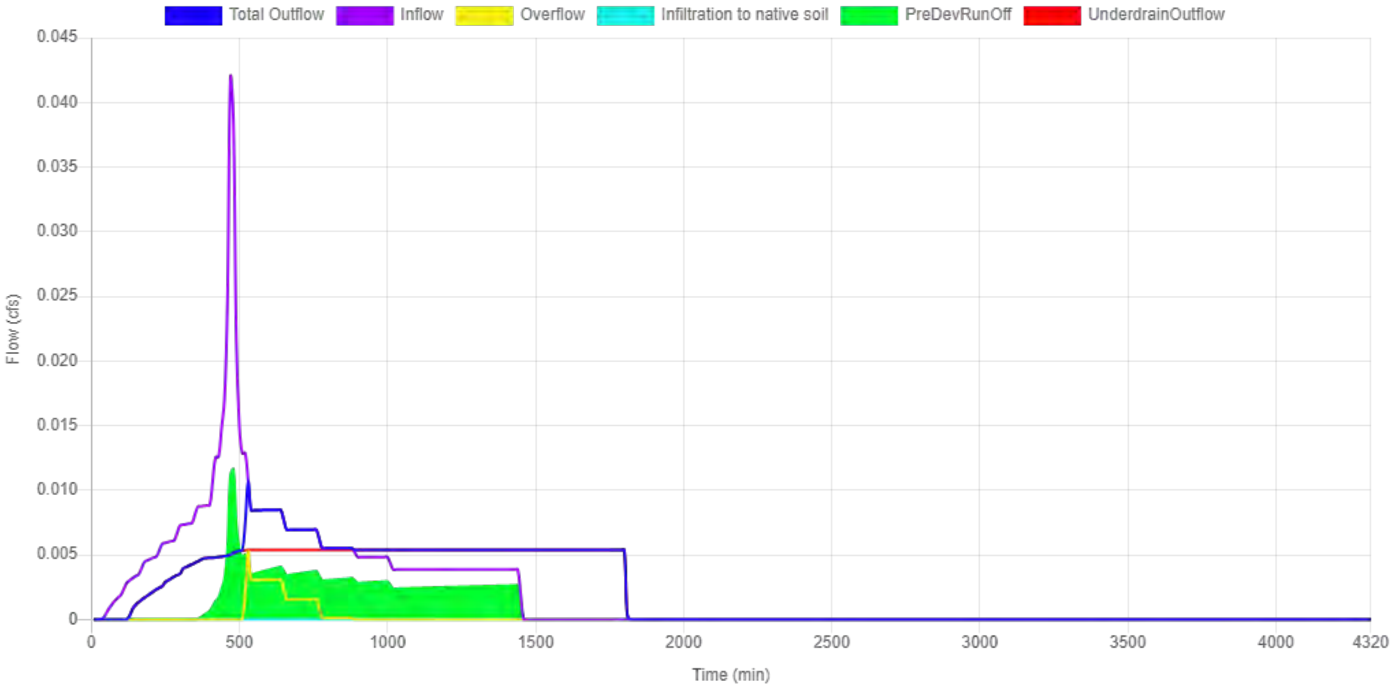
5-Year


Total Outflow
Inflow
Overflow
Infiltration to native soil
PreDevRunOff
UnderdrainOutflow
Flow (cfs)
0.040
0.035
0.030
0.025
0.020
0.015
0.010
0.005
0
0
100
200
300
400
500
600
700
800
900
1000
1100
1200
1300
1400
1470
Time (min)

10-Year



10-Year



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Appendix D

Conveyance Calculations (will be included in final report)

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