Development Services

From Concept to Construction

Phone: 503-823-7300 Email: bds@portlandoregon.gov 1900 SW 4th Ave, Portland, OR 97201
More Contact Info (http://www.portlandoregon.gov//bds/article/519984)

APPEAL SUMMARY

Status: Decision Rendered

Appeal ID: 14749	**Project Address:** 1332 N Skidmore St
Hearing Date: 3/8/17	**Appellant Name:** Jeff Shoemaker
Case No.: P-006	**Appellant Phone:** 971-280-8646
Appeal Type: Plumbing	**Plans Examiner/Inspector:** preliminary
Project Type: commercial	**Stories:** 6 **Occupancy:** A-3, B, M, R-2, S-2, U **Construction Type:** III-A over I-A
Building/Business Name:	**Fire Sprinklers:** Yes - Throughout, NFPA 13
Appeal Involves: other: Drywell Location	**LUR or Permit Application No.:** 16-290087-LU
Plan Submitted Option: pdf [File 1]	**Proposed use:** Mixed Use

APPEAL INFORMATION SHEET

Appeal item 1

Code Section	OPCS 2014, 1101.5.3.2
Requires	No drywell shall be located closer than 5 feet to a property line nor closer than 10 feet to a building unless approved by the building official.
Proposed Design	The proposal is to manage stormwater by installing 3 drywells on the western lot of the proposed Overlook project, these drywells would be located under the basement with access through a locking manhole lid from the on grade parking area. The drywells will be placed approximately 24' west from the alley right of way line. The appeal request is for the drywells to be located under the basement/parking garage. No request is required from the property line as the drywells are being proposed to meet the minimum setback requirement from the property line. See attached site plan for drywell locations and setbacks from property line.
Reason for alternative	The site is located in a Central Employment zoning area with zero setback requirements. The developer would like to maximize commercial, residential density (per the City's goals to alleviate the housing crises), and underground parking while responsibly managing the stormwater on-site. GeoDesign Incorporated is the geotechnical firm on record for the project and has prepared the accompanying report stating (see attached GeoDesign Memo), "Based on our review of the information provided, the proposed dry wells will not significantly affect bearing capacity of the foots. The dry well structures should have suffiecient structural capacity to resist surcharge loads from the adjacent footings."

Froelich Engineers is the structural engineering firm on record for the project and has also prepared the accompanying report stating (see attached Froelich Memo),
"If the drywells are placed midway between foundation elements, we do not anticipate any issues with the drywells impacting the performance of the foundations as indicated by the memorandum prepared for this site by GeoDesign."

APPEAL DECISION

Drywell System located beneath the building: Granted as proposed

The Administrative Appeal Board finds that the information submitted by the appellant demonstrates that the approved modifications or alternate methods are consistent with the intent of the code; do not lessen health, safety, accessibility, life, fire safety or structural requirements; and that special conditions unique to this project make strict application of those code sections impractical.

Pursuant to City Code Chapter 25.07, you may appeal this decision to the Plumbing Code Board of Appeal within 180 calendar days of the date this decision is published. For information on the appeals process and costs, including forms, appeal fee, payment methods and fee waivers, go to www.portlandoregon.gov/bds/appealsinfo, call (503) 823-7300 or come in to the Development Services Center.

GEODESIGN INC

Memorandum

To:	Lee Novak	From:	Joe T. Westergreen, P.E. Brett A. Shipton, P.E., G.E.
Company:	Fore Green Development, LLC	Date:	February 23, 2017
Address:	1741 Village Center Circle Las Vegas, NV 89134		

cc:	Jeff Shoemaker, DOWL (via email only) Korey Derrick, DOWL (via email only) Henry Miller, Fore Construction, LLC (via email only)

GDI Project:	ForeProp-6-01
RE:	Dry Well Review Overlook & Skidmore N Skidmore Street Portland, Oregon

This memorandum documents our review of the proposed location and design of dry wells for the proposed development located southeast of the intersection of N Skidmore Street and N Maryland Avenue in Portland, Oregon. We prepared a geotechnical report[1] for the site development.

We understand the proposed development consists of a new six-story apartment building with one level of below-grade parking that is supported on shallow foundations. We reviewed a dry well location plan prepared by DOWL that shows the locations of the proposed dry wells. Three dry wells are proposed in the west basement within the drive aisle approximately 24 feet west of the basement wall and 2 dry wells are proposed in the parking area approximately 16 feet east of the public right-of-way. We recommend that the top of the dry well perforated section be located a minimum of 5 feet below the elevation of adjacent basement footings.

Based on our review of the information provided, the proposed dry wells will not significantly affect bearing capacity of the footings. The dry well structures should have sufficient structural capacity to resist surcharge loads from the adjacent footings.

REGISTERED PROFESSIONAL ENGINEER
56207
OREGON
July 11, 2000
BRETT A. SHIPTON
EXPIRES: 6/30/18

JTW:BAS:kt
One copy submitted (via email only)
Document ID: ForeProp-6-01-022317-geom.docx

[1] GeoDesign, Inc. *Report of Geotechnical Engineering Services; Overlook & Skidmore; N Skidmore Street; Portland, Oregon*, dated September 15, 2016. GeoDesign Project: ForeProp-6-01

FROELICH
ENGINEERS

Memorandum

Client: Holst Architecture
Job Name: **Overlook Project**
Southeast Corner of N. Skidmore & Overlook
Portland, Oregon

FCE Job #: 16-T171
Date: February 28, 2017

Comments:

Froelich Engineers understands that surface water generated by this project will be treated/stored on site. The Civil Engineer indicated that this will likely consist of drywells and other storage tanks within the areas of the foundations.

The structure for this building is anticipated to be conventional spread footings bearing on soils beneath the single-floor subterranean parking slab.

If the drywells are placed midway between foundation elements, we do not anticipate any issues with the drywells impacting the performance of the foundations as indicated by the memorandum prepared for this site by GeoDesign.

Please call our office if you have any questions.

STRUCTURAL
REGISTERED PROFESSIONAL
ENGINEER
51411PE
OREGON
SEPT. 22, 1998
TIMOTHY T. TERICH
EXPIRES: 6-30-18

Drainage Report

Overlook

14272-01

Prepared for

Fore Green Development
1741 Village Center Circle
Las Vegas, Nevada 89134

March 3, 2017

Revised from December 21, 2106

Prepared for	Fore Green Development
Project Name	Drainage Report
Job Number	14272-01
Date	March 3, 2017

DOWL
720 SW Washington Street, Suite 750
Portland, Oregon
97205

Telephone: 971-280-8641
Facsimile: 800-865-9847
jshoemaker@dowl.com

Name	Title	Date	Revision	Reviewer
Atalia Raskin	WR Project Manager	12/21/2016	1	Korey Derrick
Atalia Raskin	WR Project Manager	03/3/2017	2	Korey Derrick

Executive Summary

The Overlook project will consist of two six story mixed use buildings (five over one). The proposed building to the west of the public alley (Interstate/Montana Alley) will also include one floor of underground parking as well as a small at grade parking lot. The proposed building to the east will have a small at grade parking lot and a small roof terrace deck. Public improvements will occur on N Maryland Ave, N Skidmore St, N Montana Ave, and the public alley between Maryland and Montana.

The purpose of this report is to describe the stormwater strategy being proposed as part of the Overlook development and to show the design follows the standards and regulations developed by the City of Portland. These regulations are identified in the City of Portland's *Stormwater Management Manual*, Bureau of Environmental Services, dated August 2016.

Stormwater Management

The City of Portland has developed a stormwater discharge hierarchy that includes four stormwater disposal categories. The highest technically feasible category must be used prior to moving to a lower category. Infiltration through vegetated facilities, followed by infiltration through underground infiltration facilities are the preferred methods of disposal.

- Proposed apartment building rooftop will drain into proposed onsite drywells for infiltration. Per the geotechnical recommendations infiltration rates in excess of 14" per hour can be expected below 10'. Borings did not encounter groundwater during field investigations. Groundwater is generally greater than 50' in the site vicinity.
- Sedimentation manholes followed by a public sump (drywell) is proposed for the public alley (Interstate/Montana Alley). See N Interstate/Montana Alley –Public Stormwater Analysis Memo dated March 3, 2017 by DOWL.
- The stormwater management for the Overlook development falls under stormwater discharge hierarchy 2, however per section 1.3.3, rooftops or pedestrian-only plaza runoff can drain directly into underground or subsurface infiltration systems (drywells) without requiring additional pollution reduction. Therefore, the proposed project will fall under stormwater hierarchy two.

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

Atalia Raskin, PE

WR Project Manager

Table of Contents

Tables

Figures

A. Project Overview

A.1 Project Overview

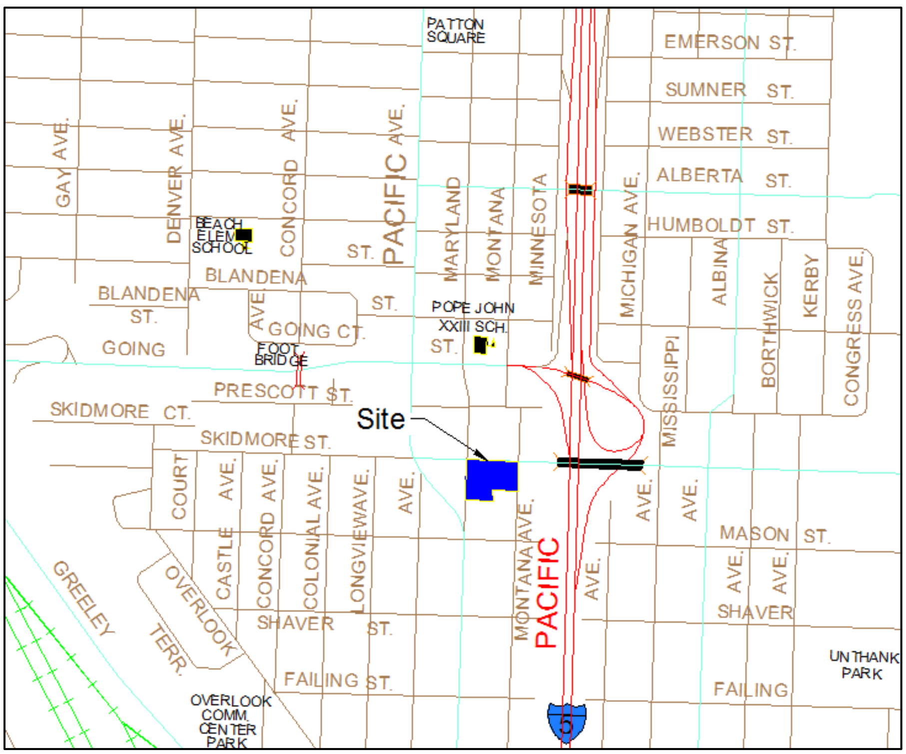
The Overlook project will consist of two six story mixed use buildings (five over one). The proposed building to the west of the public alley will also include one floor of underground parking as well as a small at grade parking lot. Three drywells are proposed within the underground parking lot. The project is applying for a pumping code appeal for the drywells located within the proposed basement. The proposed building to the east will have a small at grade parking lot and a small roof terrace deck. Two drywells will be located within the parking lot. Public improvements will occur on N Maryland Ave, N Skidmore St, N Montana Ave, and the public alley between Maryland and Montana.

The purpose of this report is to describe the stormwater strategy being proposed as part of the Overlook development and to show the design follows the standards and regulations developed by the City of Portland. These regulations are identified in the City of Portland's *Stormwater Management Manual*, Bureau of Environmental Services, dated August 2016.

A.2 Location

The proposed Overlook development is located between N Maryland Ave and N Montana Ave, just south of N Skidmore Street, in Portland, Oregon (See Vicinity Map).

Figure A-1 Vicinity Map



A.3 Stormwater Hierarchy

The disposal hierarchy found on page 1-25 in the City of Portland's *Stormwater Management Manual* was used to evaluate flow control options at the site.

Per City of Portland's Manual, Section 1.3.1 – Infiltration and Discharge Hierarch:

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

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

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

The proposed project will send site improvement area to proposed onsite drywells.

A.4 Topography

The existing site is currently a restaurant with associated parking and landscaping. Site topography slopes to the southwest corner of the site. The highest elevation of 188 feet is located at the center of the project area. The lowest elevation of 184 feet is located in the southwestern property corner. Site slopes are approximately 2.5% across the site.

A.5 Climate

The site is located in Portland, Oregon approximately 50 miles inland from the Pacific Ocean. There is a gradual change in seasons with defined seasonal characteristics. Average daily temperatures range from 44°F to 82°F. Record temperatures recorded for this region of the state are -18°F and 108°F. Average annual rainfall recorded in this area is 41 inches. Average annual snowfall is approximately 3 inches between December and February.

A.6 Site Geology

The underlying soil type on the existing site as classified by the United States Department of Agriculture Soil Survey of Multnomah County, Oregon is Urban Land-Latourell Complex, with 0 to 3 percent slopes (See Appendix A: USGS Soils Map - Multnomah County). A hydrologic soil group is not assigned to this soil type.

A.7 Existing Hydrology

Catch basins are located within the existing parking lot. The survey was unable to determine where the exiting catch basins drain. The sewer pipe surrounding the project is a combined public sewer system. No water quality treatment and water quantity control is provided onsite.

A.8 Existing Basin Areas

Table A-1 lists the basin area in existing conditions. The existing basin is 82.7% pervious prior to the right-of-way dedication. See Exhibit #1 – Existing Basin Areas in the appendix.

Table A-1 Basin Areas

Development Condition	Impervious Basin Area (ac)	Pervious Basin Area (ac)	Total Basin Area (ac)
Prior to Dedication			
Basin 1	0.424	0.035	0.459
Basin 2	0.240	0.104	0.344
After Dedication			
Basin 1	0.417	0.031	0.448
Basin 2	0.237	0.099	0.336

B. Proposed Conditions

B.1 Hydrology

Proposed building and canopy runoff will be conveyed into roof drains that will connect into the onsite drywells for infiltration. A small parking lot will drain to onsite drywells. The parking lot has less than 50 stalls and fewer than 1,000 trips per day. Therefore, additional water quality treatment is not required. The proposed Interstate/Montana Alley will drain to a public sump.

The proposed drywells in Basin 1 (west building) will be located under the lower level parking garage. An escape route is not possible for these drywells; therefore the drywells are designed to infiltration the entire 100-year storm event. The proposed drywells in Basin 2 (east building) will be located within the small parking lot area. These drywells are sized to infiltration the entire 10-year storm event with the escape route flowing down the alley.

B.2 Coefficients

A Rational Method coefficient of 0.88 will be assumed for impervious surfaces that has a ground slope less than 5 percent per The City of Portland *Drainage & Sewer Manual*.

B.3 Time of Concentration

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

The minimum time of concentration is 5 minutes in highly developed urban areas (i.e. parking lots, roof tops) and the maximum is 100 minutes in rural areas. The time of concentration value used for proposed conditions is 5 minutes.

B.4 Proposed Basin Areas

Table B-1 lists the basin area in proposed conditions. See Exhibit #2 Proposed Basin Areas for proposed basin delineation within the Appendix.

Table B-1 Proposed Basin Areas

Basin Area	Impervious Basin Area (ac)	Pervious Basin Area (ac)	Total Basin Area (ac)
Basin 1	0.448	0.000	0.448
Basin 2	0.327	0.009	0.336
Total	0.775	0.009	0.784

C. Hydrologic Design Analysis

C.1 Design Guidelines

The site is located within the jurisdiction of the City of Portland. The analysis and design criteria used for stormwater management described in this section will follow the City of Portland's *Sewer and Drainage Facilities Design Manual*, revised in June 2007.

C.2 Hydrograph Method (SBUH)

Rainstorms occur naturally over long periods of time. The most effective way of estimating storm rainfall is by using the hydrograph method. The hydrograph method generates storm runoff based on physical characteristics of the site. The Santa Barbara Urban Hydrograph (SBUH) was used for this analysis. The SBUH method is based on the curve number (CN) approach, and uses the Natural Resources Conservation Service's (NRCS) equations for computing soil absorption and precipitation excess. The SBUH method converts the incremental runoff depths into instantaneous hydrographs, which are then routed through an imaginary reservoir with a time delay equal to the basin time of concentration.

The rainfall distribution to be used within the City of Portland's jurisdiction is the design storm of 24-hour duration based on the standard NRCS Type 1A rainfall distribution. Table C-1 shows total precipitation depths for different storm events which were used for the type 1A 24-hour rainfall distribution in xpswmm. A typical NRCS Type 1A 24-hour rainfall distribution for a 10-year storm event is shown in Figure C-1.

Figure C-1 10-year Type 1A Rainfall Distribution

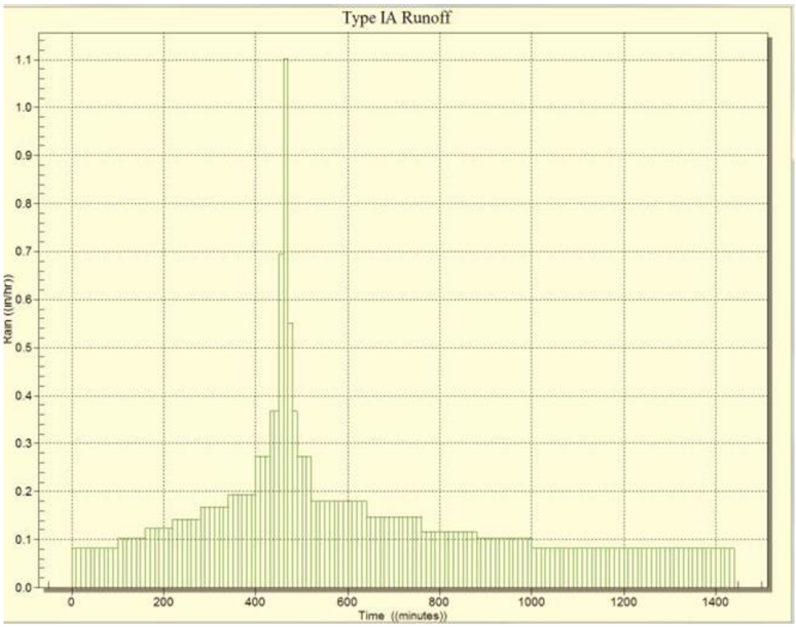


Table C-1 Precipitation Depth

Recurrence Interval (years)	Total Precipitation Depth (in)
2	2.4
10	3.4
25	3.9
100	4.4

C.3 Rational Method

The rational method was also used for this site to generate peak runoff rates for the conveyance analysis.

The Rational Formula:

$$Q = C * I * A$$

Where;

Q = Peak Runoff, cfs
C = Runoff coefficient representing a ratio between runoff to rainfall, dimensionless

I = Average rainfall intensity, inches/hour, for a design storm duration equal to Tc
A = Drainage area contributing to the point of interest, acres

D. Hydraulic Design Analysis

D.1 Design Guidelines

The analysis and design criteria described in this section will follow the City of Portland's *Sewer and Drainage Facilities Design Manual*, revised in June 2007. The manual requires storm drainage facilities be designed to pass the 10-year storm event without surcharge and have a means to pass the 25-year storm event without damage to property or endangering human life or public health, or significant environmental damage.

D.2 System Capacities

All conveyance is internal to the building. Conveyance will be completed by the mechanical engineer.

E. Water Quality

E.1 Water Quality Guidelines

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

E.2 Water Quality Facility

All roof and canopy areas will drain into roof laterals that will collect stormwater and be conveyed through the building before connecting into the proposed drywells for infiltration; therefore water quality treatment is not required for roof runoff. The small parking lot area outside of the building will drain directly to onsite drywells. The parking lot has less than 50 stalls and fewer than 1,000 trips per day. Therefore, additional water quality treatment is not required.

F. Water Quantity

F.1 Water Quality Requirements and Guidelines

Water quantity facilities were designed in accordance with the City of Portland's *Stormwater Management Manual*, issued in August 2016.

F.2 Drywells

The performance approach was used to size the proposed private drywells, per The City of Portland's *Stormwater Management Manual* for sites with greater than 10,000 square feet of impervious area.

Drywells will be located under the proposed western building and adjacent to the proposed eastern building. The drywells will maintain 5' distance off of the property line. A plumbing code exception is being applied for due to the proximity to the building. Basin 1 will be conveyed to drywells 1, 2, and 3. The proposed drywells are set to a depth below poor draining soils and into well-draining soils. Basin 2 will be conveyed to drywells 4 and 5. The proposed drywells are set to a depth below poor draining soils and into well-draining soils.

Infiltration testing at the site was measured by GeoDesign Inc. The infiltration rate of 14 inches/hour at the one boring location was measured at the site. The test was measured at depths of approximately 10.2 feet and represent un-factored rate. A safety factor of 2 was applied to the infiltration rate to establish the long-term design infiltration rate of 7 inches/hour.

Drywells located within building setbacks or under the building require designing them to the 100-year storm, unless an approved escape route as defined by section 2.3.2 – Stormwater Facility and Configuration is available for the drywells.

As shown on Exhibit #2 Proposed Basin Areas are outlined below, an approved escape route is provided for Drywells 4 and 5 within Basin 2. Therefore, these drywells were sized for the 10-year storm. An escape route is not available for Drywells 1, 2, and 3. Therefore, these drywells were sized for the 100-year storm. Table F-1 shows basin size and drywell dimensions. The five drywell depths are approximately 28 feet, with a minimum of 20 feet of perforations.

Table F-1 Drywell Sizing

Drywell #	Drywell Depth (ft)	Max Drywell Volume (cu.-ft)		Max Water Depth (ft)	
		10-yr	100-yr	10-yr	100-yr
1	28.0	-	409	-	23.7
2	28.0	-	409	-	23.7
3	28.0	-	409	-	23.7
4	28.0	401	-	23.3	-
5	28.0	317	-	18.4	-

Sizing was completed in xpswmm using the SBUH method. A rating curve was created for the amount of flow leaving the drywell based on the depth of the drywell. Outfall rates ranged from 0.004 cfs to 0.065 cfs. The draw-down time for the drywells is approximately 8 hours once full. (See Appendix: xpswmm Runoff and Conveyance Tables, and Drywell Stage Hydrographs).

F.3 Escape Route

For Drywells 4 and 5, in the event the capacity of the drywells is exceeded, an escape route has been designed to maintain public safety and prevent property damage. Stormwater runoff will sheet flow west and offsite onto Interstate/Montana Alley. The escape route is shown on Exhibit #2 Proposed Basin Areas. Drywells 1, 2, and 3 are designed to contain the 100-year storm event.

G. Operation & Maintenance

An Operation and Maintenance (O&M) Plan will be recorded with Multnomah County. The proposed stormwater facilities will be operated and maintained per this plan.

H. Summary

The proposed stormwater management system will meet or exceed the requirements of the City of Portland. Stormwater quality and quantity designs followed the City of Portland’s *Stormwater Management Manual*, issued in August 2016.

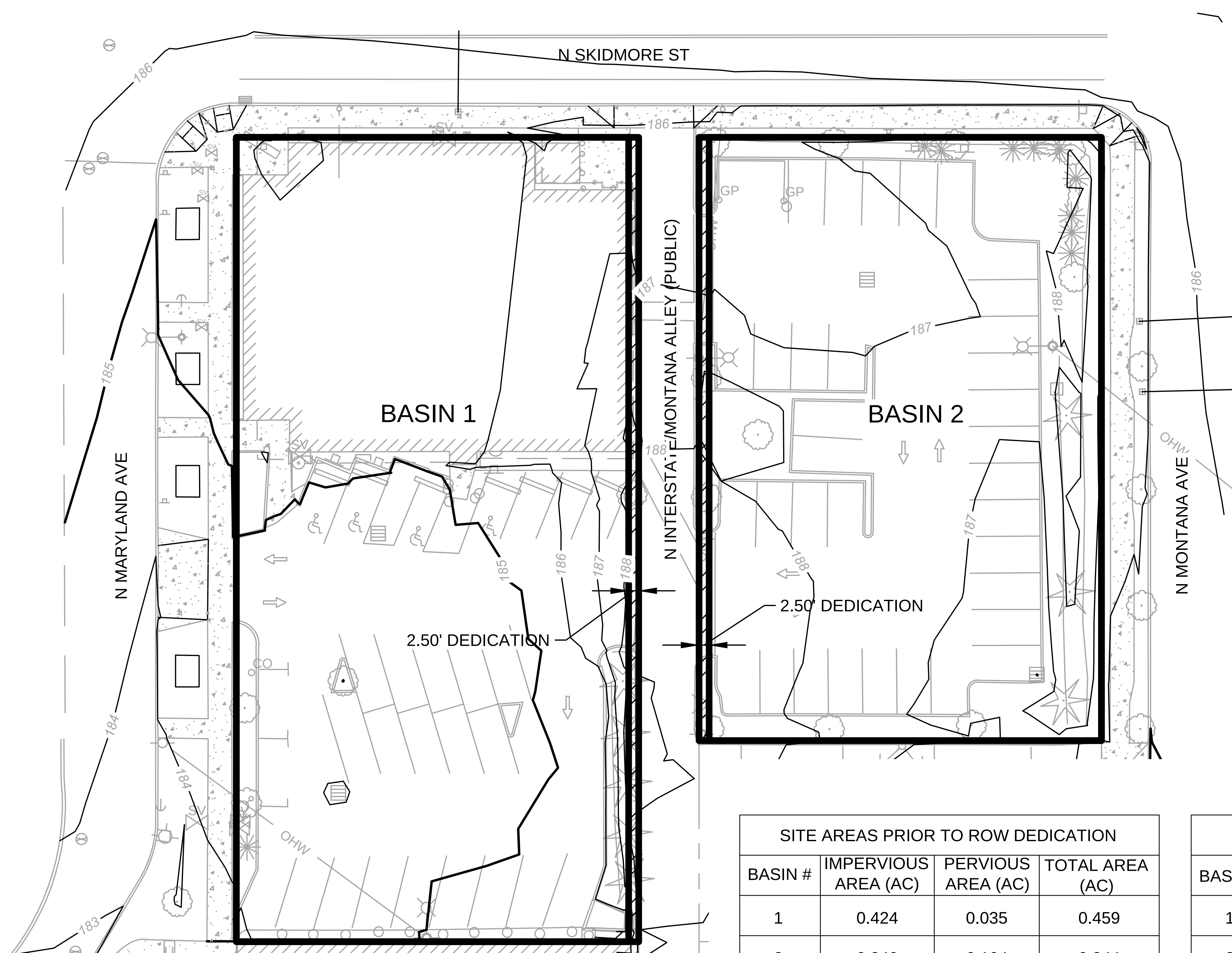
Technical Appendix

TECHNICAL APPENDIX

- Exhibit #1 – Existing Basin Areas
- Exhibit #2 – Proposed Basin Areas
- USGS Soils Map - Multnomah County
- Drywell Stage-Discharge Table
- xpswmm
    - Schematic Layout
    - Runoff Table
    - Conveyance Table
    - Drywell Stage Hydrographs

REFERENCES

1. *USDA Soil Survey of Multnomah County, Oregon Area*
2. City of Portland *Stormwater Management Manual*
   August, 2016
3. City of Portland Sewer and *Drainage Facilities Design Manual*
   Revised June 2007



SITE AREAS PRIOR TO ROW DEDICATION			
BASIN #	IMPERVIOUS AREA (AC)	PERVIOUS AREA (AC)	TOTAL AREA (AC)
1	0.424	0.035	0.459
2	0.240	0.104	0.344
TOTAL SITE AREA (AC):			0.803

SITE AREAS AFTER ROW DEDICATION			
BASIN #	IMPERVIOUS AREA (AC)	PERVIOUS AREA (AC)	TOTAL AREA (AC)
1	0.417	0.031	0.448
2	0.237	0.099	0.336
TOTAL SITE AREA (AC):			0.784

N

30 0 30
SCALE IN FEET

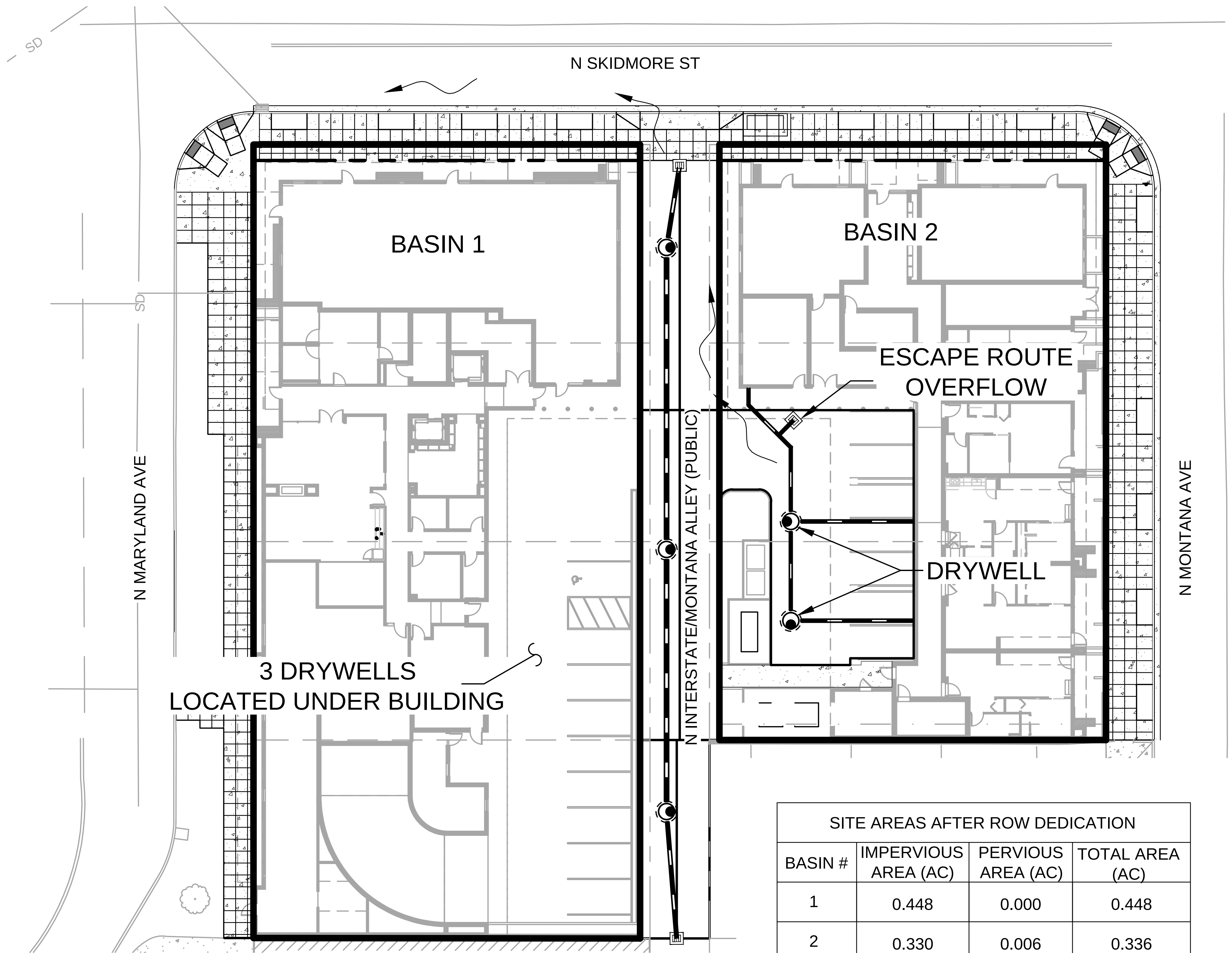
DOWL
WWW.DOWL.COM
720 SW Washington Street, #750
Portland, Oregon 97205
971-280-8641

EXISTING BASIN AREAS
OVERLOOK
FORE GREEN DEVELOPMENT

PROJECT 14272-01
DATE 03/03/2017
BY BCF

EXHIBIT #1

Q:\22\14272-01\40Study_DD Submittal\Exhibits\Existing Basins.dwg PLOT DATE 2017-3-2 17:55 SAVED DATE 2016-12-21 11:43 USER: araskin



SITE AREAS AFTER ROW DEDICATION			
BASIN #	IMPERVIOUS AREA (AC)	PERVIOUS AREA (AC)	TOTAL AREA (AC)
1	0.448	0.000	0.448
2	0.330	0.006	0.336
TOTAL SITE AREA (AC):			0.784

ESCAPE ROUTE

N

30 0 30

SCALE IN FEET

DOWL

PROPOSED BASIN AREA
OVERLOOK
FORE GREEN DEVELOPMENT

PROJECT	14272-01
DATE	03/03/2017
BY	BCF

EXHIBIT #2

Q:\22\14272-01\40Study_DD Submittal\Exhibits\Proposed Basins.dwg PLOT DATE 2017-3-2 17:48 SAVED DATE 2017-03-02 15:05 USER: araskin

Soil Map—Multnomah County Area, Oregon

Map Scale: 1:544 if printed on A landscape (11" x 8.5") sheet.

N

Meters: 0 5 10 20 30

Feet: 0 25 50 100 150

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

MAP LEGEND

Area of Interest (AOI)

- Area of Interest (AOI)

Soils

- Soil Map Unit Polygons
- Soil Map Unit Lines
- Soil Map Unit Points

Special Point Features

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

Water Features

- Streams and Canals

Transportation

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

Background

- Aerial Photography

MAP INFORMATION

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

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

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

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

Source of Map: Natural Resources Conservation Service
Web Soil Survey URL: http://websoilsurvey.nrcs.usda.gov
Coordinate System: Web Mercator (EPSG:3857)

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

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

Soil Survey Area: Multnomah County Area, Oregon
Survey Area Data: Version 14, Sep 16, 2016

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

Date(s) aerial images were photographed: Jul 26, 2014—Sep 5, 2014

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

Map Unit Legend

Multnomah County Area, Oregon (OR051)			
Map Unit Symbol	**Map Unit Name**	**Acres in AOI**	**Percent of AOI**
51A	Urban land-Latourell complex, 0 to 3 percent slopes	1.3	100.0%
Totals for Area of Interest		**1.3**	**100.0%**

Drywell Stage-Discharge Table

SUBJECT	Overlook	**DATE**	3/3/2017
PROJECT NO.	14272-01		

Rock Diameter	6	ft
Design Infiltration Rate	7	in/hr

Depth, ft	Drywell Volume, sf	Rock Volume, sf	Total Volume, sf	Volume, ac	Q out, cfs
0	12.56	4.71	17.27	0.000396	0.004579
1	12.56	4.71	17.27	0.000396	0.004579
2	12.56	4.71	17.27	0.000396	0.004579
3	12.56	4.71	17.27	0.000396	0.004579
4	12.56	4.71	17.27	0.000396	0.007632
5	12.56	4.71	17.27	0.000396	0.010685
6	12.56	4.71	17.27	0.000396	0.013738
7	12.56	4.71	17.27	0.000396	0.01679
8	12.56	4.71	17.27	0.000396	0.019843
9	12.56	4.71	17.27	0.000396	0.022896
10	12.56	4.71	17.27	0.000396	0.025949
11	12.56	4.71	17.27	0.000396	0.029001
12	12.56	4.71	17.27	0.000396	0.032054
13	12.56	4.71	17.27	0.000396	0.035107
14	12.56	4.71	17.27	0.000396	0.03816
15	12.56	4.71	17.27	0.000396	0.041213
16	12.56	4.71	17.27	0.000396	0.044265
17	12.56	4.71	17.27	0.000396	0.047318
18	12.56	4.71	17.27	0.000396	0.050371
19	12.56	4.71	17.27	0.000396	0.053424
20	12.56	4.71	17.27	0.000396	0.056476
21	12.56	4.71	17.27	0.000396	0.059529
22	12.56	4.71	17.27	0.000396	0.062582
23	12.56	4.71	17.27	0.000396	0.065635
24	12.56	4.71	17.27	0.000396	0.065635
25	12.56	4.71	17.27	0.000396	0.065635
26	12.56	4.71	17.27	0.000396	0.065635
27	12.56	4.71	17.27	0.000396	0.065635
28	12.56	4.71	17.27	0.000396	0.065635

xpswmm RUNOFF DATA (10-YR STORM EVENT)

Freemont & Mississippi - City of Portland, Oregon

Node Information					Runoff Information			
Node Name	Area	Impervious	SBUH Curve Number	Tc	Rainfall	Infiltration	Surface Runoff	
	acre	%		min.	in	in	in	cfs
Drywell#1	0.448	100	98	5	3.40	0.00	3.06	0.35
Drywell#4	0.336	100	98	5	3.40	0	3.061	0.265

xpswmm RUNOFF DATA (100-YR STORM EVENT)

Freemont & Mississippi - City of Portland, Oregon

Node Information					Runoff Information			
Node Name	Area	Impervious	SBUH Curve Number	Tc	Rainfall	Infiltration	Surface Runoff	
	acre	%		min.	in	in	in	cfs
Drywell#1	0.448	100	98	5	4.40	0.00	4.06	0.46
Drywell#4	0.336	100	98	5	4.40	0	4.056	0.347

xpswmm CONVEYANCE DATA (10-YEAR STORM EVENT)

Freemont & Mississippi - City of Portland, Oregon

Location			Conduit Properties			Conduit Results						Node Information (Manhole, Pond, Tee, Outfall, Ditch Inlet, Catch Basin)							
Link	Station																		
	From	To	Diameter	Length	Slope	Design Capacity	Qmax/ Qdesign	Max Flow	Max Velocity	Max Flow Depth	y/d0	US Ground Elev.	DS Ground Elev.	US IE	DS IE	US Freeboard	DS Freeboard	US HGL	DS HGL
			ft	ft	%	cfs		cfs	ft/s	ft		ft	ft	ft	ft	ft	ft	ft	ft
P1	Drywell#1	Drywell#2	1.00	25.00	0.00	0.10	2.75	0.29	1.47	0.32	0.32	172.00	172.00	144.00	144.00	4.68	4.73	167.32	167.27
P2	Drywell#2	Drywell#3	1.00	25.00	0.00	0.10	2.03	0.21	1.31	0.27	0.27	172.00	172.00	144.00	144.00	4.73	19.03	167.27	152.97
P3	Drywell#4	Drywell#5	1.00	25.00	0.00	0.10	1.91	0.20	1.28	0.26	0.26	172.00	172.00	144.00	144.00	4.74	9.65	167.26	162.35

xpswmm CONVEYANCE DATA (100-YEAR STORM EVENT)

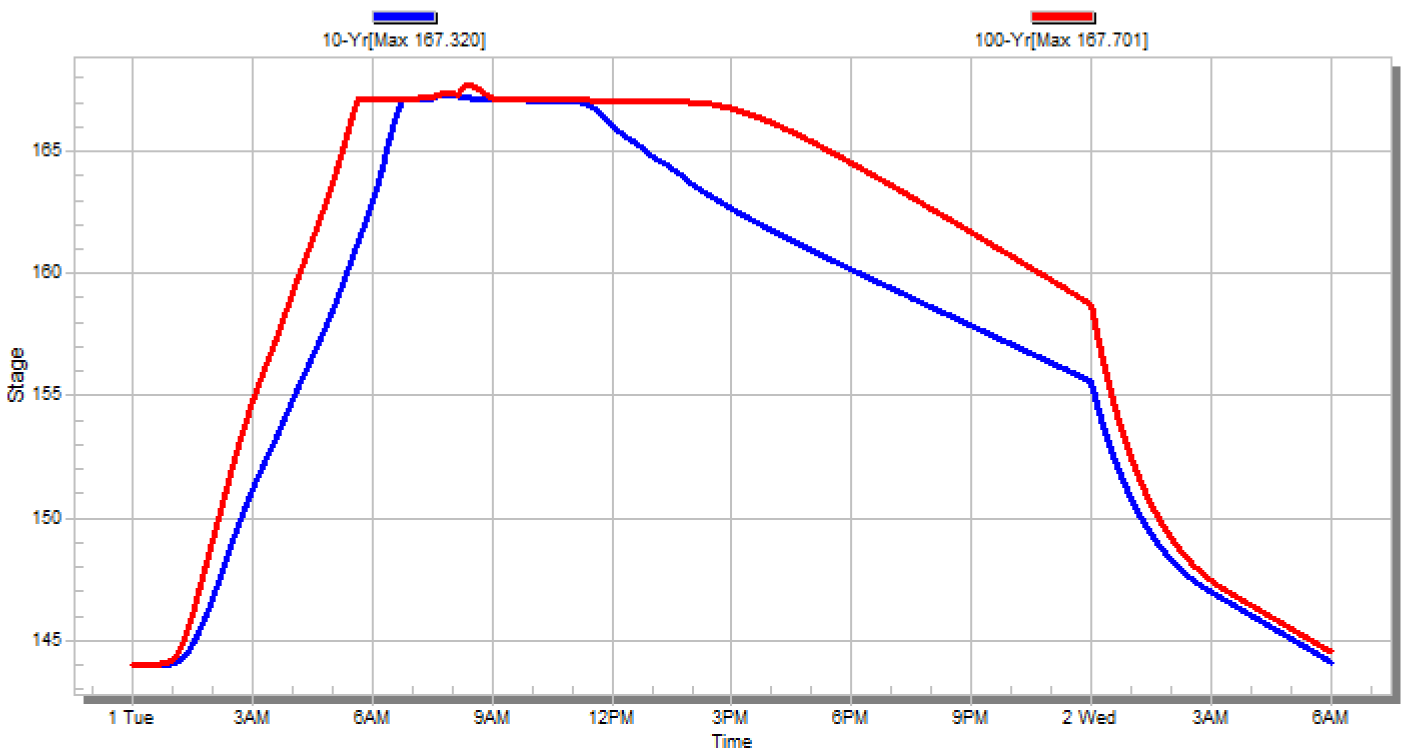
Freemont & Mississippi - City of Portland, Oregon

Location			Conduit Properties			Conduit Results						Node Information (Manhole, Pond, Tee, Outfall, Ditch Inlet, Catch Basin)							
Link	Station																		
	From	To	Diameter	Length	Slope	Design Capacity	Qmax/ Qdesign	Max Flow	Max Velocity	Max Flow Depth	y/d0	US Ground Elev.	DS Ground Elev.	US IE	DS IE	US Freeboard	DS Freeboard	US HGL	DS HGL
			ft	ft	%	cfs		cfs	ft/s	ft		ft	ft	ft	ft	ft	ft	ft	ft
P1	Drywell#1	Drywell#2	1.00	25.00	0.00	0.10	3.79	0.40	1.59	0.70	0.70	172.00	172.00	144.00	144.00	4.30	4.30	167.70	167.70
P2	Drywell#2	Drywell#3	1.00	25.00	0.00	0.10	3.16	0.33	1.55	0.70	0.70	172.00	172.00	144.00	144.00	4.30	4.30	167.70	167.70
P3	Drywell#4	Drywell#5	1.00	25.00	0.00	0.10	2.69	0.28	1.46	3.57	3.57	172.00	172.00	144.00	144.00	1.43	1.43	170.57	170.57

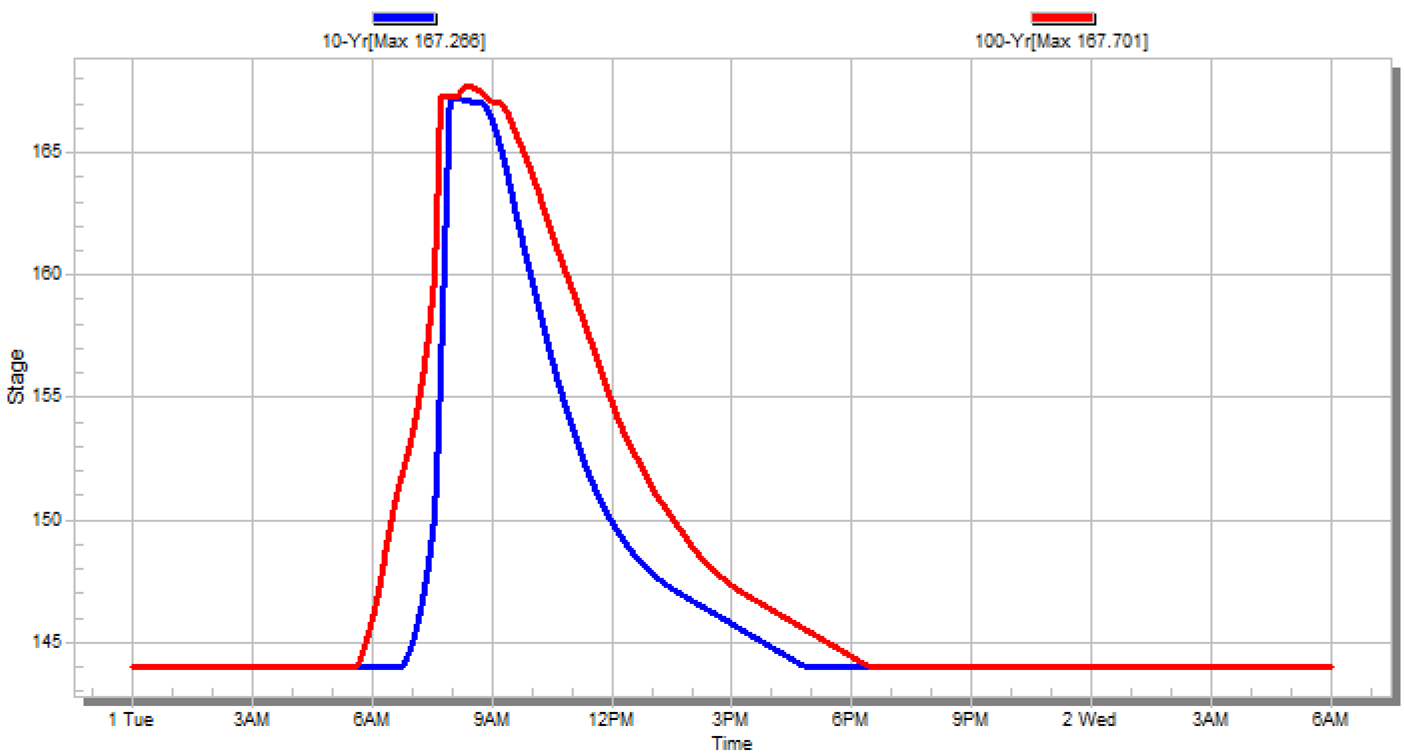
Drywell Stage

Overlook Development

Drywell#1

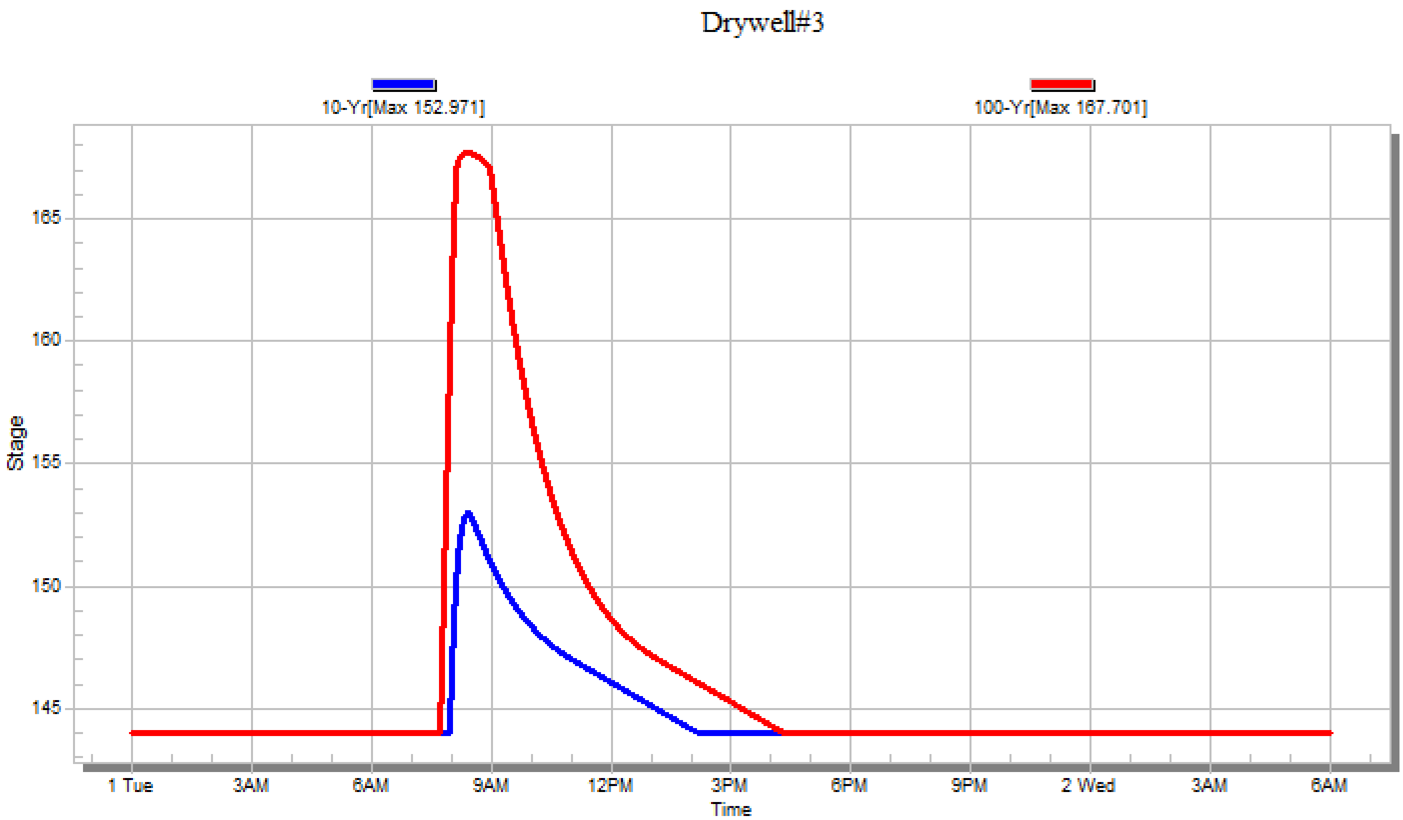


Drywell#2



Drywell Stage

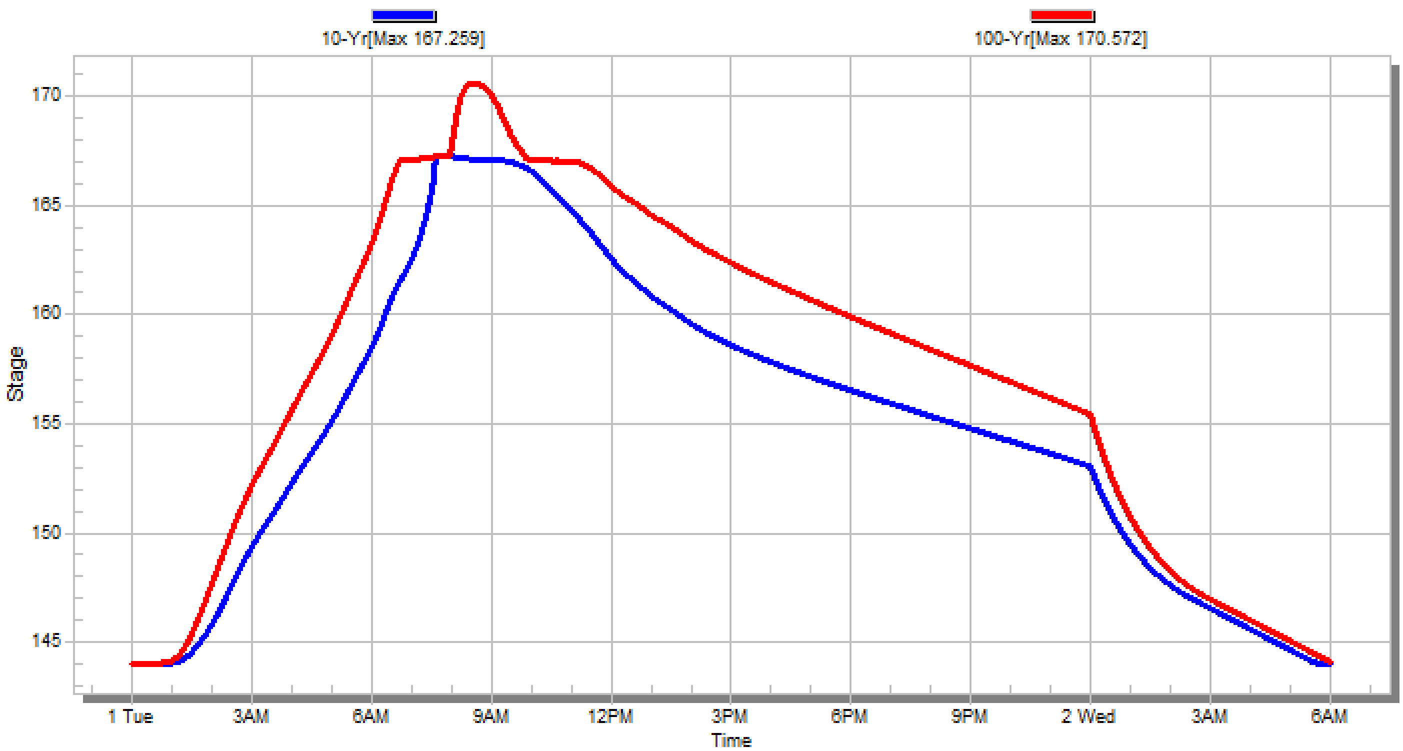
Overlook Development



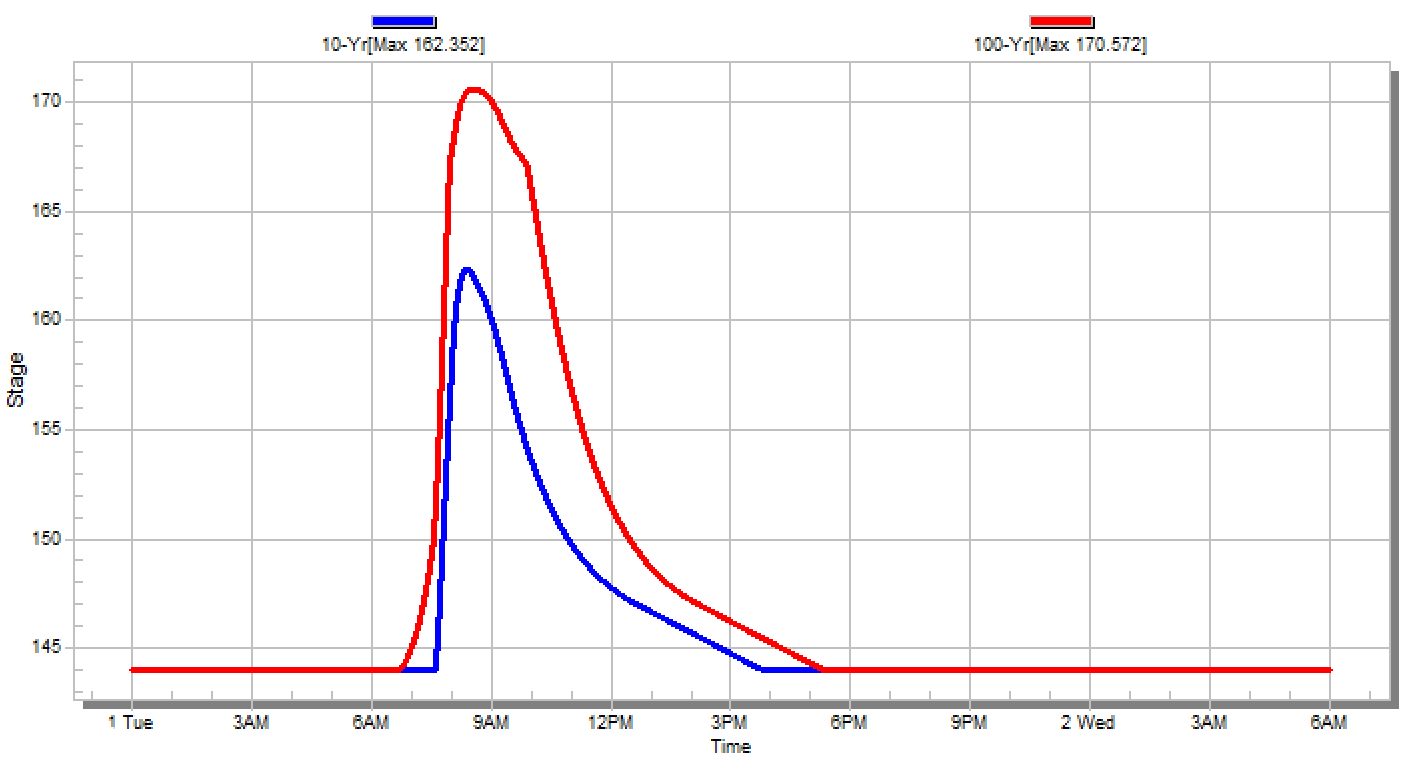
Drywell Stage

Overlook Development

Drywell#4

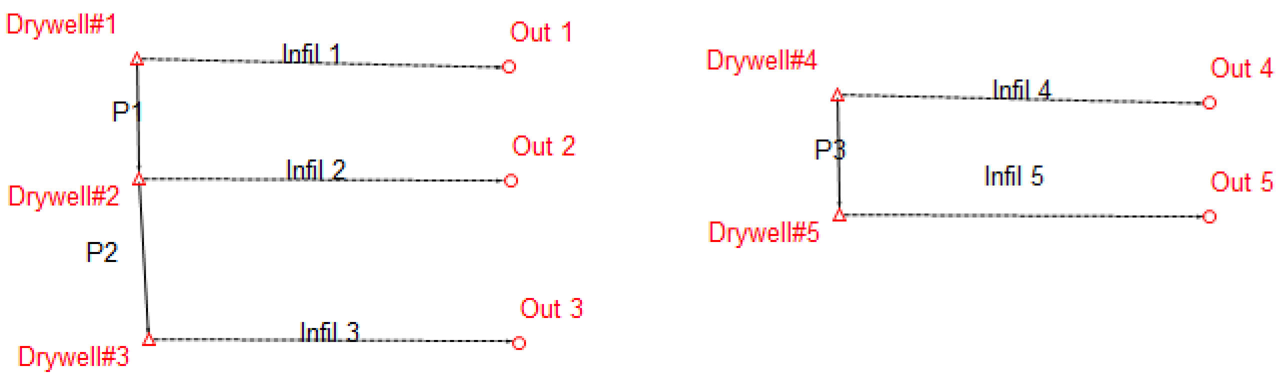


Drywell#5



Schematic Layout

Overlook Development



Reports and Studies

Report of Geotechnical Engineering Services, GeoDesign Inc., September 15, 2016.

REPORT OF GEOTECHNICAL ENGINEERING SERVICES

Overlook & Skidmore
N Skidmore Street
Portland, Oregon

For
Fore Green Development
September 15, 2016

GeoDesign Project: ForeProp-6-01

September 15, 2016

Fore Green Development
1741 Village Center Circle
Las Vegas, NV 89134

Attention: Lee Novak

Report of Geotechnical Engineering Services
Overlook & Skidmore
N Skidmore Street
Portland, Oregon
GeoDesign Project: ForeProp-6-01

GeoDesign, Inc. is pleased to submit our report of geotechnical engineering services for the proposed development located southeast of the intersection of N Skidmore Street and N Maryland Avenue in Portland, Oregon. Our services for this project were conducted in accordance with our proposal dated August 9, 2016.

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

GJS:BAS:kt
Attachments
One copy submitted (via email only)
Document ID: ForeProp-6-01-091516-geor.docx

EXECUTIVE SUMMARY

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

- The proposed structure can be supported on spread footings bearing on firm native soil. All undocumented fill should be removed from under foundation elements.

- For the proposed embedded portion of the building, excavation walls will need to be shored if they are adjacent to existing pavement and utility right-of-ways. If excavations are not adjacent to existing pavement or utility right-of-ways, temporary excavation walls can be sloped.

- The fine-grained soil at the site can be sensitive to small changes in moisture content and difficult 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. If the moisture content of the soil is currently above optimum, drying will be required if used as structural fill.

- The site will require demolition of existing buildings, concrete slabs, and other site features. In particular, wet, sensitive subgrade should be anticipated beneath the pavement areas.

- The granular soil is prone to raveling, and special precautions will be required to prevent undermining adjacent infrastructure if excavations are required nearby.

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1.0 INTRODUCTION

GeoDesign, Inc. is pleased to submit this geotechnical engineering report for the proposed development located southeast of the intersection of N Skidmore Street and N Maryland Avenue in Portland, Oregon. Figure 1 shows the site relative to existing physical features. Figure 2 shows the current site layout and our approximate exploration locations. Acronyms and abbreviations used herein are defined at the end of this document.

The property is currently occupied by one single-story structure and a paved parking area. We understand the existing structures will be demolished to accommodate the new development.

Plans are preliminary at the time of this report. Based on our review of a conceptual site plan, the development will consist of a new six-story apartment building with one level of below-ground parking.

Structural loads were not available at the time of this report. We have assumed that column loads will be between 300 and 400 kips and wall loads will be less than 4 kips per foot. Floor slab loads are assumed to be less than 150 psf.

2.0 SCOPE OF SERVICES

The purpose of our services was to provide geotechnical engineering recommendations for use in design and construction of the proposed development. Our scope of work included the following:

- Reviewed readily available published geologic data and our in-house files for existing information on subsurface conditions in the site vicinity.
- Completed a subsurface exploration program consisting of three borings to depths ranging between 31.5 and 51.5 feet BGS. Infiltration testing was conducted at a depth of 10.2 feet BGS in boring B-1.
- Maintained continuous logs of the explorations and collected samples at representative intervals.
- Performed the following laboratory tests:
  - Eight moisture content determinations in general accordance with ASTM D 2216
  - Three fines content determinations in general accordance with ASTM D 1140
- Provided recommendations for site preparation and grading, including 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 structure. Our recommendations include 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, if required.
- Provided recommendations for construction of AC pavements for on-site access roads and parking areas, including subbase, base course, and AC paving thickness.

- Provided seismic design recommendations in accordance with the procedures outlined in the 2012 IBC and 2014 SOSSC.
- Prepared this geotechnical engineering report that presents our findings, conclusions, and recommendations.

3.0 SITE CONDITIONS

3.1 SURFACE CONDITIONS

The site is located on the southeast corner of the intersection at N Skidmore Street and N Maryland Avenue in Portland, Oregon, and is surrounded by a mix of residential and commercial properties. The site is currently occupied by a single-story building with paved parking lots to the east and west. The parking areas are relatively flat with ground surface elevations ranging from approximately 185 to 190 feet.

3.2 SUBSURFACE CONDITIONS

3.2.1 General

We explored subsurface conditions at the site by drilling three borings (B-1 through B-3) to depths ranging between 31.5 and 51.5 feet BGS. The approximate exploration locations are shown on Figure 2. A description of the subsurface exploration program and the exploration logs and laboratory test results are presented in the Appendix.

Our explorations generally encountered silt underlain by sand. A pavement section consisting of 3 inches AC over 4 to 9 inches of aggregate base was encountered at the ground surface at the boring locations. The following sections summarize the subsurface units encountered.

3.2.2 Silt

Silt was encountered to depths of approximately 11.5 and 7 feet BGS in borings B-2 and B-3, respectively. Silt was not encountered in boring B-1. SPTs conducted in this unit indicate that the silt is generally medium stiff to stiff in consistency. Laboratory testing on selected samples of the silt indicates the moisture contents varied from 33 to 41 percent at the time of our explorations.

3.2.3 Sand

Silty sand was encountered below the pavement section in boring B-1 and underlies the silt in borings B-2 and B-3. SPTs conducted in this unit indicate that the sand is generally loose to dense in consistency. Laboratory testing on selected samples of the sand indicates the moisture contents varied from 17 to 34 percent at the time of our explorations.

3.2.4 Groundwater

We did not observe groundwater in our explorations. Based on our review of water well logs on file with the Oregon Water Resources Department and projects completed in the site vicinity, groundwater is generally at a depth greater than 50 feet BGS. 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.

3.3 INFILTRATION TESTING

Infiltration testing was completed in boring B-1 to assist in the evaluation of stormwater infiltration facilities for the project. The infiltration testing was conducted in general accordance with the recommendations for the "Encased Falling Head" method included in the 2014 City of Portland Stormwater Management Manual. We performed the falling-head infiltration test in the boring within an 8-inch-diameter casing. The infiltration rate was measured under low-head conditions of approximately 12 inches of water or less after saturated conditions had been achieved.

Table 1 summarizes the infiltration test results and fines content determinations. The exploration logs and the laboratory test results are presented in the Appendix.

Table 1. Infiltration Rates

Location	Depth (feet BGS)	Material	Infiltration Rate (inches/hour)
B-1	10.2	Silty SAND	14

1. Fines content: material passing the U.S. Standard No. 200 sieve

The infiltration rate provided in Table 1 is a measured rate and is unfactored. Correction factors should be applied to the measured infiltration rate by the civil engineer during design to account for the degree of long-term maintenance and influent/pre-treatment control, as well as the potential for long-term clogging due to siltation and bio-buildup, depending on the proposed length, location, and type of infiltration facility. In addition, correction factors to be applied to the test results are provided in Exhibit F.2-1 of the 2014 City of Portland Stormwater Management Manual.

4.0 DESIGN RECOMMENDATIONS

The following sections provide our design recommendations for the project.

4.1 FOUNDATION SUPPORT

The proposed structure can be supported on conventional spread footings bearing on granular pads that are at least 12 inches thick overlying undisturbed, firm native soil. The granular pads should extend at least 6 inches beyond the footing perimeter. Footings should not be directly supported on soft or loose soil. Granular pads should consist of Imported granular material.

4.1.1 Bearing Capacity

We recommend that spread footings bearing on the sand be sized based on an allowable bearing pressure of 3,000 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 pressures apply to the total of dead and long-term live loads and may be increased by 50 percent for short-term loads, such as those resulting from wind or seismic forces.

We recommend that isolated column footings have a minimum width of 24 inches and continuous wall footings have minimum width of 18 inches. The bottom of exterior footings should be founded at least 18 inches below the lowest adjacent grade. Interior footings should be founded at least 12 inches below the base of the floor slab.

4.1.2 Lateral Resistance

Lateral loads on footings can be resisted by passive earth pressure on the sides of the structure 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 300 pcf, modeled as an equivalent fluid pressure. Adjacent floor slabs, pavements, or the upper 12-inch depth of adjacent unpaved areas should not be considered when calculating passive resistance.

Footings in contact with crushed rock should be designed using a coefficient of friction of 0.40.

4.1.3 Settlement

We anticipate that total post-construction settlement will be less than 1 inch for spread foundations designed in accordance with the recommendations provided above. Differential settlement between similarly loaded footings is expected to be less than ½ inch.

4.1.4 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, and softened subgrades (if present) have been removed. Localized deepening of footing excavations may be required to penetrate any deleterious material.

4.2 FLOOR SLABS

To help reduce moisture transmission and to provide uniform support, we recommend a minimum 6-inch-thick layer of floor slab base rock be placed and compacted over prepared subgrade. The floor slab base rock should meet the requirements in the "Materials" section of this report and compacted to at least 95 percent of the maximum dry density as determined by ASTM D 1557.

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

Slabs should be reinforced according to their proposed use and per the structural engineer's recommendations. Slabs-on-grade may be designed assuming a modulus of subgrade reaction, k, of 120 psi per inch.

4.3 RETAINING STRUCTURES

Our recommendations for permanent retaining walls are based on the following assumptions: (1) the walls are not in contact with temporary shoring, (2) the walls consist of conventional, cantilevered retaining walls or embedded building walls, (3) the walls are less than 15 feet in height, (4) the retained soil is level, and (5) drainage is provided behind the walls to prevent hydrostatic pressures for developing. Re-evaluation of our recommendations will be required if the retaining wall design criteria for the project vary from these assumptions.

Walls not restrained from rotation should be designed using an equivalent fluid pressure of 35 pcf. An equivalent fluid pressure of 55 pcf should be used for design of walls restrained from rotation. These values do not consider hydrostatic pressures. Permanent basement walls with more than one level of bracing should be designed to resist lateral earth pressures presented on Figure 3.

Seismic earth pressures on embedded walls should be designed using a dynamic force of 7H2 pounds per linear foot of wall, where H is the wall height. This seismic force should be applied as a distributed load throughout the excavated depth of the retaining wall, with the centroid located at a distance of 0.6H from the base of the wall. Surcharge-induced lateral earth pressures should be computed using the methods presented on Figure 4.

4.4 PAVEMENT

New pavement should be installed on competent subgrade or new engineered fill prepared in conformance with the "Site Preparation" and "Materials" sections of this report. Given the building proposed, our pavement recommendations are based on the assumption that the standard-duty traffic section will be subject to passenger cars and occasional maintenance and delivery-type trucks. We do not have specific information on the frequency and types of vehicles that will use the area; however, we have assumed that standard traffic conditions will consist of a maximum of 2 trucks per day and a maximum of 200 cars per day. We recommend the heavy-duty pavement section be constructed in areas that will be subject to higher traffic volumes (such as entrances and areas subject to repeated delivery vehicles). The heavy-duty section assumes traffic will consist of up to ten trucks per day.

We calculated pavement sections using the above-referenced traffic conditions using a design life of 10 and 20 years and AASHTO design methods. The design of the recommended pavement section is based on an assumed resilient modulus of 4,000 psi and the assumption that construction will be completed during an extended period of dry weather. Wet weather construction may require an increased thickness of aggregate base to support the rock trucks and compaction equipment. Table 2 summarizes the recommended pavement sections.

Table 2. Pavement Section Thickness

Design Life (years)	Standard-Duty Section		Heavy-Duty Section	
	AC Thickness (inches)	Aggregate Base Thickness (inches)	AC Thickness (inches)	Aggregate Base Thickness (inches)
10	2.5	7.0	3.0	10.0
20	2.5	8.0	3.5	10.0

The AC and aggregate base should meet the specifications for ACP and aggregate base rock provided in the "Materials" section of this report.

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.

4.5 SEISMIC DESIGN CONSIDERATIONS

4.5.1 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 3 should be used to compute seismic base shear forces.

Table 3. 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 = 0.97 g	S_1 = 0.42 g
Site Class	D	
Site Coefficient, F	F_a = 1.11	F_v = 1.58
Adjusted Spectral Acceleration, S_M	S_{MS} = 1.08 g	S_{M1} = 0.66 g
Design Spectral Response Acceleration Parameters, S_D	S_{DS} = 0.72 g	S_{D1} = 0.44 g
Design Spectral PGA	0.29 g	

4.5.2 Liquefaction

Liquefaction is a phenomenon caused by a rapid increase in pore water pressure that reduces the effective stress between soil particles to near zero. The excessive buildup of pore water pressure results in the sudden loss of shear strength in a soil. Granular soil, which relies on inter-particle friction for strength, is susceptible to liquefaction until the excess pore pressures can dissipate. Sand boils and flows observed at the ground surface after an earthquake are the result of excess pore pressures dissipating upwards, carrying soil particles with the draining water. In general, loose, saturated sand soil with low silt and clay content is the most susceptible to liquefaction. Low plasticity, silty sand may be moderately susceptible to liquefaction under relatively higher levels of ground shaking. Liquefaction is not considered a site hazard.

5.0 CONSTRUCTION

5.1 SITE PREPARATION

5.1.1 Demolition

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

Voids resulting from the removal of improvements should be backfilled with compacted structural fill, as discussed in the "Structural Fill" section of this report. Utility lines abandoned under new structural components should be completely removed and backfilled with structural fill. Firm subgrade should be exposed at the bottom of the excavations before backfilling, and the sides of the temporary excavations should be sloped at a minimum of 1.5H:1V.

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

5.1.2 Clearing

There are some grass areas and trees at the site that will need to be removed. In addition, stumps and root balls should be grubbed out to the depth of the roots, which could exceed 3 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, undisturbed subgrade. The resulting excavations should be backfilled with structural fill.

Where present, the existing topsoil zone should be stripped and removed from all fill areas. The average depth of stripping for vegetated areas will be approximately 1 to 2 inches, although greater stripping depths may be required to remove localized zones of loose or organic soil. 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. Stripping should extend at least 5 feet beyond the limits of proposed structural areas.

5.1.3 Fill Improvement

Fill material was not observed during our subsurface investigation. However, within all proposed structural fill, pavement, at-grade floor slabs, and improvement areas; for a 5-foot margin beyond such areas; and where less than 3 feet of cut is required, if fill is observed at the subgrade elevation, we recommend the surface foot of the stripped subgrade be removed and replaced with structural fill or the subgrade scarified and compacted as structural fill to a depth of 1 foot.

The exposed subgrade should be closely evaluated by a geotechnical engineer during the process. Considerable soil processing, including moisture conditioning and the removal of roots or other deleterious material from the soil, may be required to use the excavated material as structural fill. Because of the moisture-sensitive nature of the on-site soil, scarification and compaction of the subgrade should be completed during the summer dry period. Compaction should be performed as described in the "Materials" section of this report.

5.1.4 Subgrade Evaluation

Upon completion of demolition, clearing, and subgrade stabilization and prior to the placement of fill, structures, or pavement improvements, the exposed subgrade should be evaluated by proof rolling. Based on the results of our explorations, our experience with the local soil conditions, and experience with subgrade under prior structures (especially building slabs), we anticipate that relatively easily disturbed soil will be encountered under the existing building.

The silt to silty sand material can be easily damaged during demolition and construction activities. Methods to protect the subgrade from disturbance are provided in the "Construction Considerations" section of this report.

A member of our geotechnical staff should observe the exposed subgrade after the demolition, site cutting, and fill removal have been completed to determine if there are additional areas of unsuitable or unstable soil. Our representative should observe a proof roll with a fully loaded dump truck or similar heavy, rubber-tired construction equipment to identify soft, loose, or unsuitable areas. Areas that appear to be too wet and soft to support proof rolling equipment should be evaluated by probing and prepared in accordance with the recommendations for wet weather construction presented in the "Construction Considerations" section of this report.

5.2 CONSTRUCTION CONSIDERATIONS

The fine-grained soil present on this site is 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 may not support construction traffic or pavement construction when the subgrade soil is wet. Accordingly, if construction is planned for periods when the subgrade soil is wet, staging and haul roads with increased thicknesses of base rock will be required. The 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. Stabilization material may be used as a substitute provided the top 4 inches of material consists of imported granular material. The actual thickness will depend on the contractor's means and methods and, accordingly, should be the contractor's responsibility. In addition, a geotextile fabric should be placed as a barrier between the subgrade and imported granular material in areas of repeated construction traffic. The imported granular material, stabilization material, and geotextile fabric should meet the specifications in the "Materials" section of this report.

5.3 EXCAVATION

5.3.1 General

Conventional heavy earthmoving equipment in proper working condition should be capable of making necessary excavations of the on-site soil for site cuts and utilities. Soil with more sand content may be prone to raveling, and shoring will be required to maintain vertical excavation walls and protect adjacent facilities. In our opinion, a soldier pile shoring system with tieback anchors is preferred for the support of the below-grade parking excavation. Geotechnical parameters for use in shoring design are provided in subsequent sections of this report.

5.3.2 Temporary Slopes

Where construction slopes are possible, temporary slopes of 1.5H:1V for excavation of the basement may be used to vertical depths of 15 feet or less, provided groundwater seepage is not encountered. At this inclination, the slopes will likely ravel and require some ongoing repair. If seepage is encountered, the slopes should be flattened to protect the surface from raveling. All cut slopes should be protected from erosion by covering them with plastic sheeting during the rainy season. If sloughing or instability is observed, the slope might need to be flattened or the cut supported by shoring.

Excavations should not undermine adjacent utilities, foundations, walkways, streets, or other hardscapes unless special shoring or underpinned support is provided. We recommend a minimum horizontal distance of 5 feet from the edge of the existing improvements to the top of the temporary slope. Unsupported excavations should not be conducted within a downward and outward projection of a 1H:1V line from 2 feet outside the edge of an adjacent structural feature.

5.3.3 Utility Trench Excavation

Trench cuts should stand vertical to a depth of approximately 4 feet in competent soil provided groundwater seepage does not occur in the trench walls. As discussed in the "Temporary Slopes" section of this report, open excavation techniques may be used to excavate trenches with depths up to 10 feet, provided the walls of the excavation are cut at a slope of 1H:1V, groundwater seepage is not present, and surcharge loads are not present within 10 feet of the top of the slope. The walls of the trench should be flattened or braced for stability and a dewatering system installed if seepage is encountered or excessive sloughing and caving occurs. Use of a trench box or other approved temporary shoring is recommended for cuts below the water table. 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.

5.3.4 Excavation Dewatering

Excavation dewatering might be required to maintain dry working conditions in excavations depending on the time of year and the severity of rainfall during construction. Based on the results of previous studies at the site, groundwater is anticipated to be relatively deep, at a depth greater than 50 feet BGS. However, perched or static groundwater could be present at shallower depths after prolonged wet periods. Excavation dewatering will be necessary if groundwater is encountered.

The selection, design, and construction of the temporary dewatering system should be the responsibility of the contractor who is in the best position to modify or adapt the system to changing groundwater conditions and construction sequencing and requirements. The construction dewatering system should be adaptable to varying flow and conditions and be capable of lowering the level of the groundwater to a minimum of 2 feet below the base of the excavation.

If groundwater is present at the base of utility excavations, we recommend placing up to 12 inches of stabilization material at the base of the excavation. Specifications for stabilization material are provided in the "Materials" section of this report.

5.3.5 Safety

All excavations should be made in accordance with applicable OSHA and state regulations. While we have described certain approaches to utility trench excavations in the foregoing discussion, the contractor should be responsible for selecting the excavation and dewatering methods, monitoring the trench excavations for safety, and providing shoring as required to protect personnel and adjacent improvements.

5.4 SHORING

5.4.1 General

If excavations for site development are within the influence zone of the footings of the adjacent structures, shoring will be required to protect the adjacent structures. The influence zone of the existing footings generally extends downwards at a 1H:1V slope from the bottom corner of the footings. We recommend the locations and depths of the existing footings be checked in the field to verify these assumptions. We have provided recommendations below for shoring design.

5.4.2 Lateral Earth Pressures

Shoring should be designed using the values on Figure 5. The recommended design parameters for cantilevered shoring and anchored shoring are shown on Figure 5. The above equivalent fluid pressures do not include effects from surcharge loads. The values on Figure 4 can be used to compute surcharge-induced lateral earth pressures.

5.4.3 Soldier Piles

Structural design of the soldier piles should consider the lateral earth pressures discussed above. In addition to lateral earth pressures, the soldier piles will be subject to compressive forces as a result of the downward component of the tieback anchor loads. We recommend a minimum soldier pile embedment of 10 feet below the base of the excavation. We recommend an allowable end bearing capacity of 4 ksf for piles embedded in the sand. An allowable skin friction of 0.5 ksf between the grout and surrounding soil is recommended. In addition, we recommend the grout at the tip of the pile have sufficient strength to withstand the imposed loads. These values should be verified by the structural engineer designing the shoring. Grout should be placed using tremie pipe methods.

We anticipate that lagging will consist of pressure-treated lumber. To maintain the integrity of the excavation, prompt and careful installation of lagging, particularly in areas of seepage and loose soil, is recommended. All voids behind the lagging should be backfilled promptly. To minimize the risk of hydrostatic pressures from developing behind the wall, lean concrete or other low-permeability material should not be used as backfill.

5.4.4 Tieback Anchors

We have provided recommendations for anchored or braced shoring if necessary. The bonded zone for the tieback anchors should be maintained outside of the "unbonded zone" shown on Figure 5. We anticipate the tieback anchors will be capable of achieving allowable bond strengths of between 3 and 4 kips per foot in the silt and sand, depending on the method of construction. A variety of methods are available for construction of tieback anchors. Therefore, we recommend the contractor be responsible for selecting the appropriate bonded length and installation methods to achieve the required anchor capacity. Tieback anchors should be locked off at 100 percent of the design load.

Prior to installing production anchors, we recommend performance tests be conducted on a minimum of two anchors. The purpose of these tests is to verify the installation procedure selected by the contractor before a large number of anchors are installed. Performance tests should be performed to 150 percent of the design load and in accordance with the guidelines provided in *Recommendations for Prestressed Rock and Soil Anchors* (Post Tensioning Institute, 2014).

We recommend proof tests be conducted on all production anchors in accordance with the guidelines presented in *Recommendations for Prestressed Rock and Soil Anchors*. The anchors should be proof tested to at least 133 percent of the design load.

5.5 DRAINAGE

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

5.6 PERMANENT SLOPES

Permanent cut and fill slopes should not exceed 2H: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.

5.7 MATERIALS

5.7.1 Structural Fill

Fill should be placed on subgrade that has been prepared in conformance with the "Site Preparation" section of this report. 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.

5.7.1.1 On-Site Soil

The native on-site soil is suitable for use as general structural fill, provided it is properly moisture conditioned; free of debris, organic material, and particles over 3 inches in diameter; and meets the specifications provided in OSSC 00330.12 (Borrow Material). We anticipate some moisture conditioning may be required to dry the soil to a moisture content near optimum. This will require an extended period of dry weather, typically experienced between early July and mid-October. It will be difficult, if not impossible, to adequately compact on-site soil during the rainy season or during prolonged periods of rainfall.

When used as structural fill, the on-site soil should be placed in lifts with a maximum uncompacted thickness of 6 to 8 inches and compacted to not less than 92 percent of the maximum dry density for fine-grained soil and 95 percent of the maximum dry density for granular soil, as determined by ASTM D 1557.

5.7.1.2 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.

5.7.1.3 Stabilization Material

Stabilization material used in staging or haul road areas, or as trench stabilization material, should consist of 4- or 6-inch-minus pit- or quarry-run rock, crushed rock, or crushed gravel and sand and should meet the specifications provided in OSSC 00330.15 (Selected Stone Backfill). The material should have a maximum particle size of 6 inches, have less than 5 percent by dry weight passing the U.S. Standard No. 4 sieve, and have 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 24 inches thick and compacted to a firm condition.

5.7.1.4 Trench Backfill

Trench backfill placed beneath, adjacent to, and for at least 12 inches above utility lines (i.e., the pipe zone) should consist of well-graded granular material with a maximum particle size of 1½ inches and less than 10 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 pavement areas and building pad, 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 10 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 90 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.

5.7.1.5 Drain Rock

Drain rock should consist of angular, granular material with a maximum particle size of 2 inches and should meet the specifications provided in OSSC 00430.11 (Granular Drain Backfill Material). The material should be free of roots, organic matter, and other unsuitable material; have less than 2 percent by dry weight passing the U.S. Standard No. 200 sieve (washed analysis); and have at least two mechanically fractured faces. Drain rock should be compacted to a well-keyed, firm condition.

5.7.1.6 Aggregate Base Rock

Imported granular material used as base rock for building floor slabs and pavements 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.

5.7.1.7 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 specifications provided in OSSC 00510.12 (Granular Wall Backfill) or OSSC 00510.13 (Granular Structure Backfill).

The backfill should be placed and compacted as recommended for structural fill, with the exception of backfill placed immediately adjacent to walls. Backfill adjacent to walls should be compacted to a lesser standard to reduce the potential for generation of excessive pressure on the walls. Backfill located within a horizontal distance of 3 feet from the retaining walls should 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 (slabs, sidewalk, or pavement) will be placed adjacent to the wall, we recommend that the upper 2 feet of fill be compacted to 95 percent of the maximum dry density, as determined by ASTM D 1557.

5.7.1.8 Recycled On-Site Material
On-site AC, conventional concrete, and aggregate base or gravel may be used as fill if they are processed to meet the requirements for their intended use and the use of these materials do not result in an environmental concern. Processing includes crushing and screening, grinding in place, or other methods to meet the requirements for structural fill as described above. The processed material should be fairly well graded and contain no metal, organic, or other deleterious material. The processed material may be mixed with on-site soil or imported fill to assist in achieving the gradation requirements. We recommend that processed recycled fill have the maximum particle sizes listed in Table 4.

Table 4. Processed Fill Maximum Particle Size

Depth of Placement[1]	Maximum Particle Size
0 to 2 feet	½ inch
2 to 6 feet	2 inches
6 to 10 feet	4 inches
deeper than 10 feet	8 inches

1. below subgrade of structural element

Recycled on-site fill material should not be used within a depth of 2 feet from foundations, floor slabs, pavements, or other subsurface elements. We also caution that excavation through recycled material that is placed as structural fill may be difficult if a significant fraction of oversized particles is present. In addition, these excavations may also be prone to raveling and caving.

5.7.1.9 AC
The AC should be Level 2, ½-inch, dense ACP according to OSSC 00744 (Asphalt Concrete Pavement). Minimum lift thickness for ½-inch ACP is 2.0 inches. Asphalt binder should be performance graded and conform to PG 64-22. The AC should be compacted using a minimum lift of 2.0 inches and a maximum lift of 3.0 inches.

5.7.1.10 Geotextile Fabric
Subgrade Geotextile Fabric
A subgrade geotextile fabric should be placed as a barrier between the subgrade and granular material in staging areas, haul road areas, or in areas of repeated construction traffic. The geotextile should meet the specifications provided in OSSC 02320 (Geosynthetics) for separation geotextiles (Table 02320-4) and be installed in accordance with OSSC 00350 (Geosynthetic Installation). The geotextile should have a Level "B" certification.

Drainage Geotextile Fabric
Drain rock and other granular material used for subsurface drains should be wrapped in a geotextile fabric that meets the specifications provided in OSSC 02320 (Geosynthetics) for drainage geotextiles (Table 02320-1) and be installed in accordance with OSSC 00350 (Geosynthetic Installation).

5.8 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.

6.0 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.

7.0 LIMITATIONS

We have prepared this report for use by Fore Green Development 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,

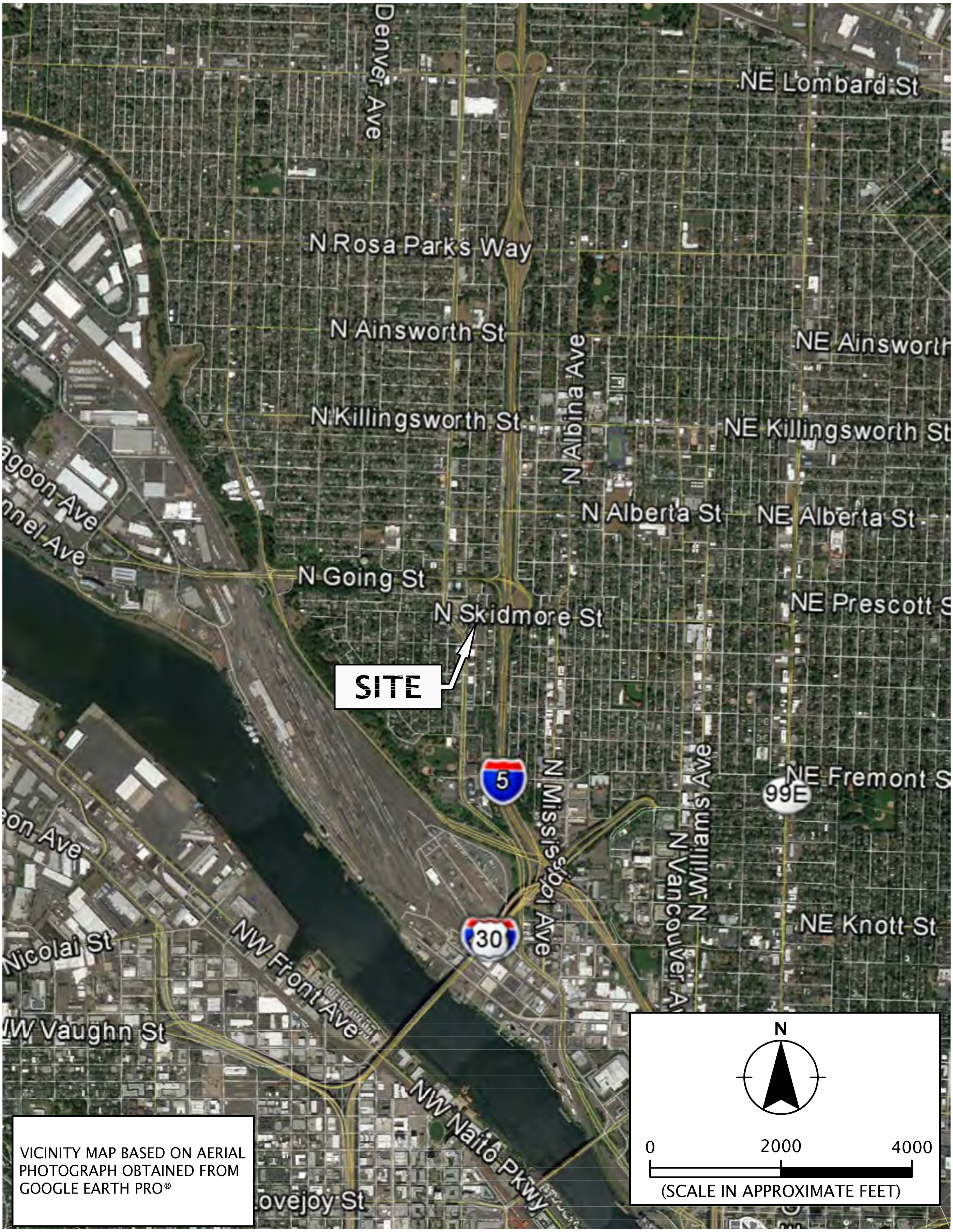
GeoDesign, Inc.

Gregory J. Schaertl, P.E. (California)
Staff 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

FIGURES



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

GEODESIGN INC 15575 SW Sequoia Parkway - Suite 100 Portland OR 97224 Off 503.968.8787 Fax 503.968.3068	FOREPROP-6-01	**VICINITY MAP**	
	SEPTEMBER 2016	OVERLOOK & SKIDMORE PORTLAND, OR	**FIGURE 1**

Printed By: mmiller | Print Date: 9/15/2016 11:21:36 AM
File Name: J:\E-L\ForeProp\ForeProp-6\ForeProp-6-01\Figures\CAD\ForeProp-6-01-VM01.dwg | Layout: FIGURE 1

LEGEND:

B-1 BORING

SITE BOUNDARY

N

0 40 80

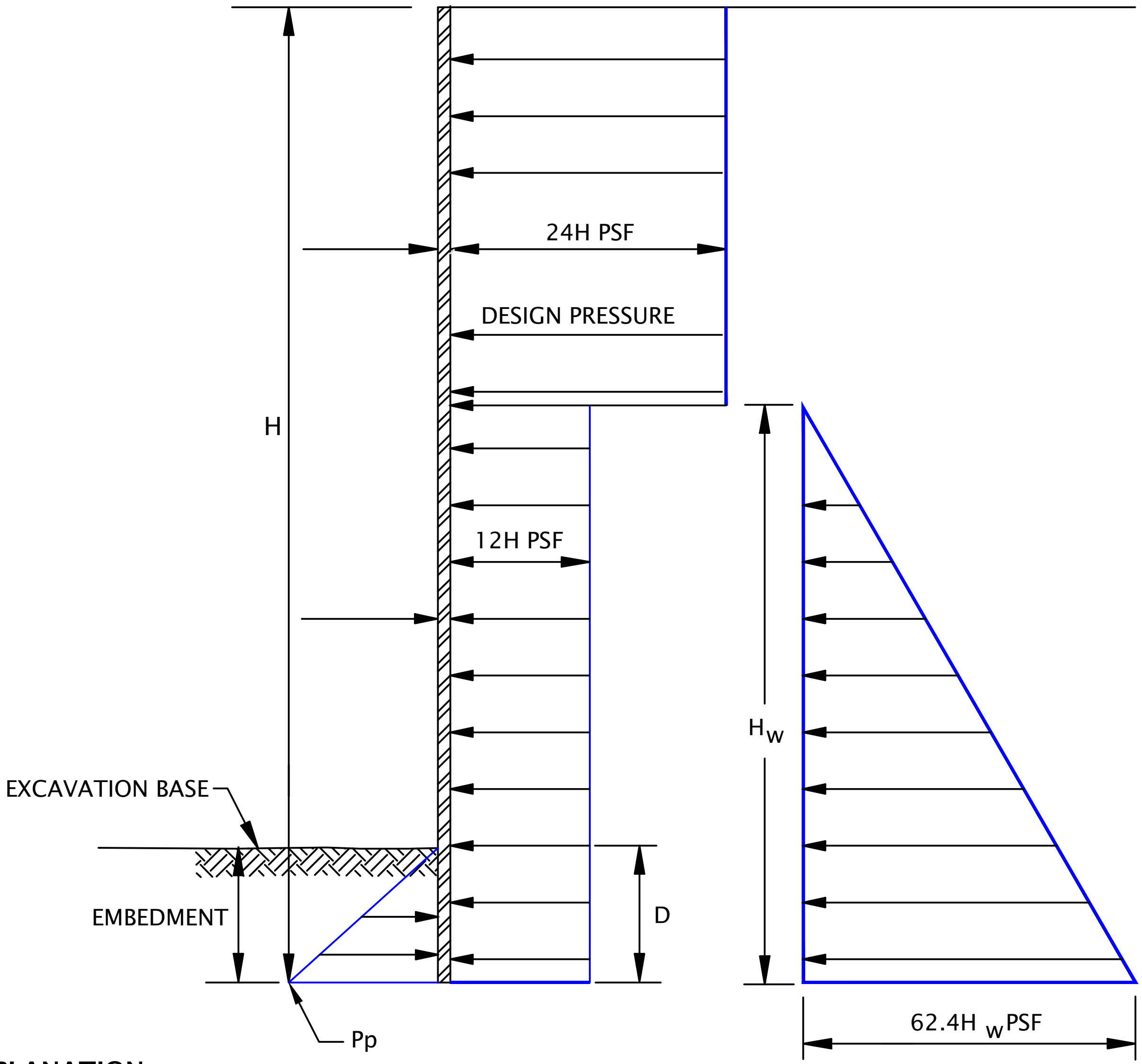
(SCALE IN FEET)

SITE PLAN BASED ON AERIAL PHOTOGRAPH OBTAINED FROM GOOGLE EARTH PRO®, SEPTEMBER 7, 2016

GEODESIGN INC 15575 SW Sequoia Parkway - Suite 100 Portland OR 97224 Off 503.968.8787 Fax 503.968.3068	FOREPROP-6-01	SITE PLAN
	SEPTEMBER 2016	OVERLOOK & SKIDMORE PORTLAND, OR
		FIGURE 2

Printed By: mmiller | Print Date: 9/15/2016 11:21:44 AM
File Name: J:\E-L\ForeProp\ForeProp-6\ForeProp-6-01\Figures\CAD\ForeProp-6-01-SP01.dwg | Layout: FIGURE 2

RECOMMENDED DESIGN PARAMETERS FOR BRACED
BASEMENT WALLS WITH HYDROSTATIC PRESSURES



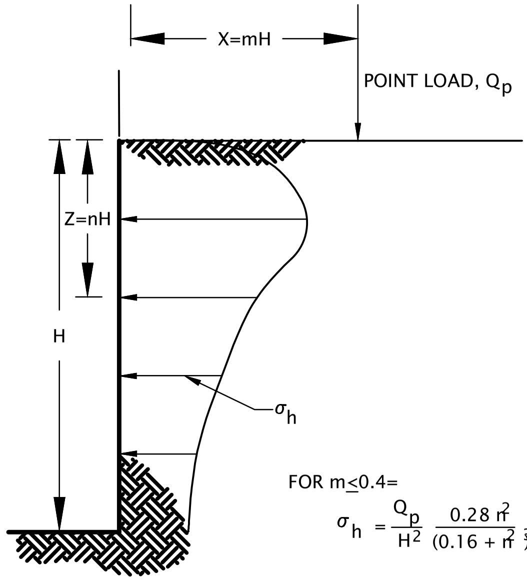
EXPLANATION:

Pp = 300D PCF, 180D PCF (BELOW GROUNDWATER)
H_W = DEPTH OF WATER ABOVE BOTTOM OF WALL IN FEET
H = DEPTH OF WALL IN FEET
D = EFFECTIVE PILE EMBEDMENT DEPTH

NOTES:

1. FIGURE SHOULD BE USED IN CONJUNCTION WITH REPORT TEXT.
2. SURCHARGE LOADS ASSOCIATED WITH CONSTRUCTION ACTIVITIES AND TRAFFIC OR ADJACENT STRUCTURES SHOULD BE ADDED TO THE EARTH PRESSURE SHOWN ABOVE, WHERE APPLICABLE.
3. ASSUMES WALL IS INTERALLY BRACED AT MORE THAN ONE LEVEL.
4. HYDROSTATIC PRESSURE CAUSED BY THE STATIC GROUNDWATER TABLE ON THE ACTIVE PRESSURE SIDE SHOULD BE INCLUDED IN FINAL DESIGN. CONSTRUCTION DEWATERING EXTERNAL TO THE EXCAVATION WILL BE REQUIRED TO REMOVE PERCHED GROUNDWATER BEFORE THE EXCAVATION IS ADVANCED.
5. THE LATERAL EARTH PRESSURES ARE UNFACTORED.
6. VALUES DO NOT INCLUDE SEISMIC LOADS.

GEODESIGN INC 15575 SW Sequoia Parkway - Suite 100 Portland OR 97224 Off 503.968.8787 Fax 503.968.3068	FOREPROP-6-01	**LATERAL EARTH PRESSURES FOR PERMANENT BASEMENT WALLS**
	SEPTEMBER 2016	OVERLOOK & SKIDMORE PORTLAND, OR — **FIGURE 3**

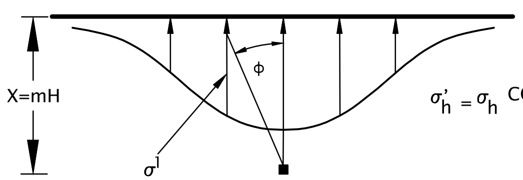


FOR $m \leq 0.4$=

$$\sigma_h = \frac{Q_p}{H^2} \frac{0.28\, n^2}{(0.16 + n^2)^3}$$

FOR $m > 0.4$=

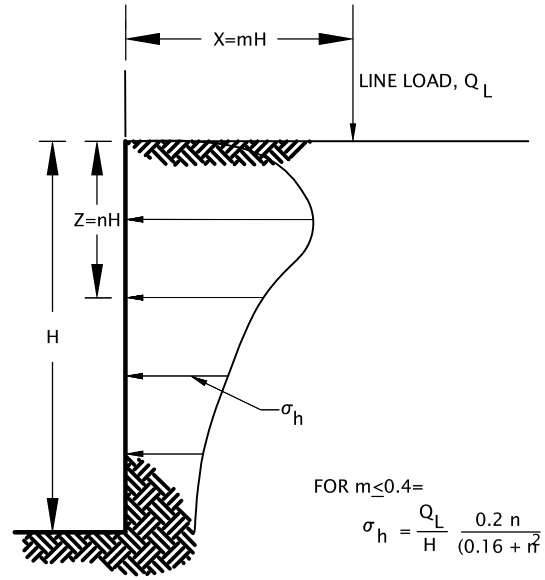
$$\sigma_h = \frac{Q_p}{H^2} \frac{1.77 m^2 n^2}{(m^2 + n^2)^3}$$



$$\sigma'_h = \sigma_h \cos^2(1.1\phi)$$

DISTRIBUTION OF HORIZONTAL PRESSURES

VERTICAL POINT LOAD



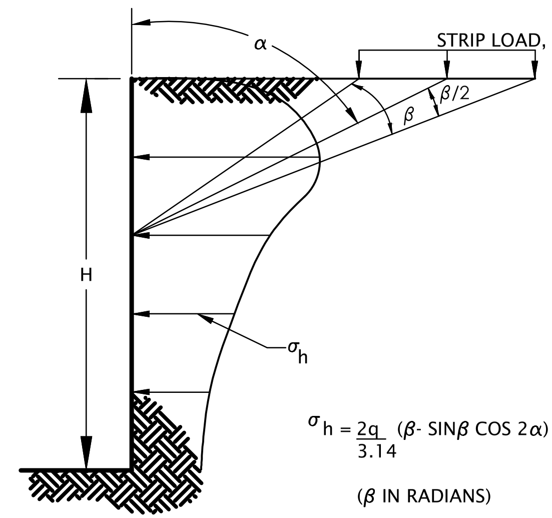
FOR $m \leq 0.4$=

$$\sigma_h = \frac{Q_L}{H} \frac{0.2\, n}{(0.16 + n^2)^2}$$

FOR $m > 0.4$=

$$\sigma_h = \frac{Q_L}{H} \frac{1.28 m^2 n}{(m^2 + n^2)^2}$$

LINE LOAD PARALLEL TO WALL



$$\sigma_h = \frac{2q}{3.14} (\beta - \sin\beta \cos 2\alpha)$$

(β IN RADIANS)

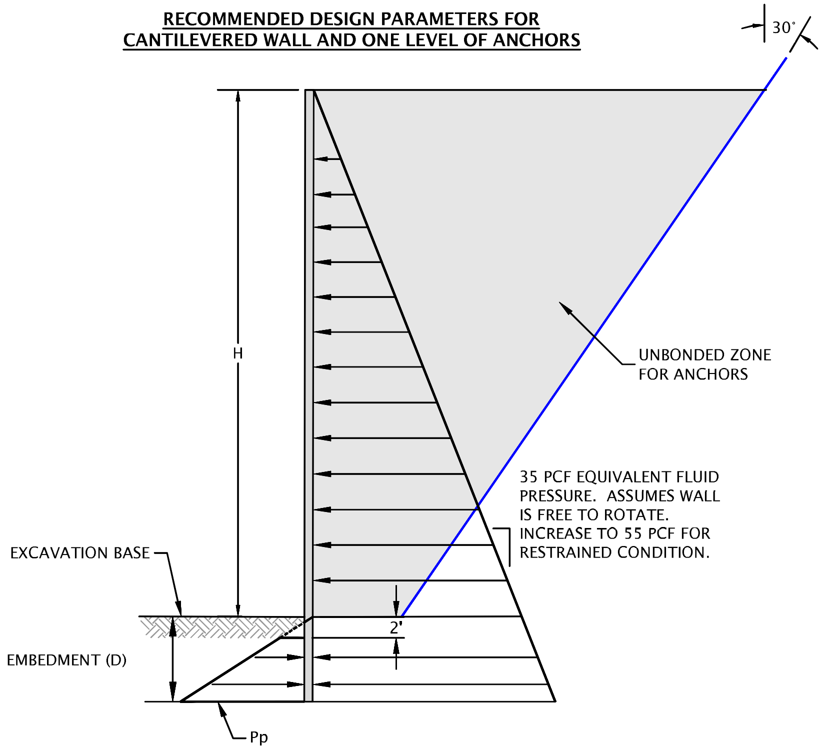
STRIP LOAD PARALLEL TO WALL

NOTES:

1. THESE GUIDELINES APPLY TO RIGID WALLS WITH POISSON'S RATIO ASSUMED TO BE 0.5 FOR BACKFILL MATERIALS.
2. LATERAL PRESSURES FROM ANY COMBINATION OF ABOVE LOADS MAY BE DETERMINED BY THE PRINCIPLE OF SUPERPOSITION.
3. VALUES ON THIS FIGURE ARE UNFACTORED.

GEODESIGN INC 15575 SW Sequoia Parkway - Suite 100 Portland OR 97224 Off 503.968.8787 Fax 503.968.3068	FOREPROP-6-01	**SURCHARGE-INDUCED LATERAL EARTH PRESSURES**	
	SEPTEMBER 2016	OVERLOOK & SKIDMORE PORTLAND, OR	**FIGURE 4**

RECOMMENDED DESIGN PARAMETERS FOR CANTILEVERED WALL AND ONE LEVEL OF ANCHORS



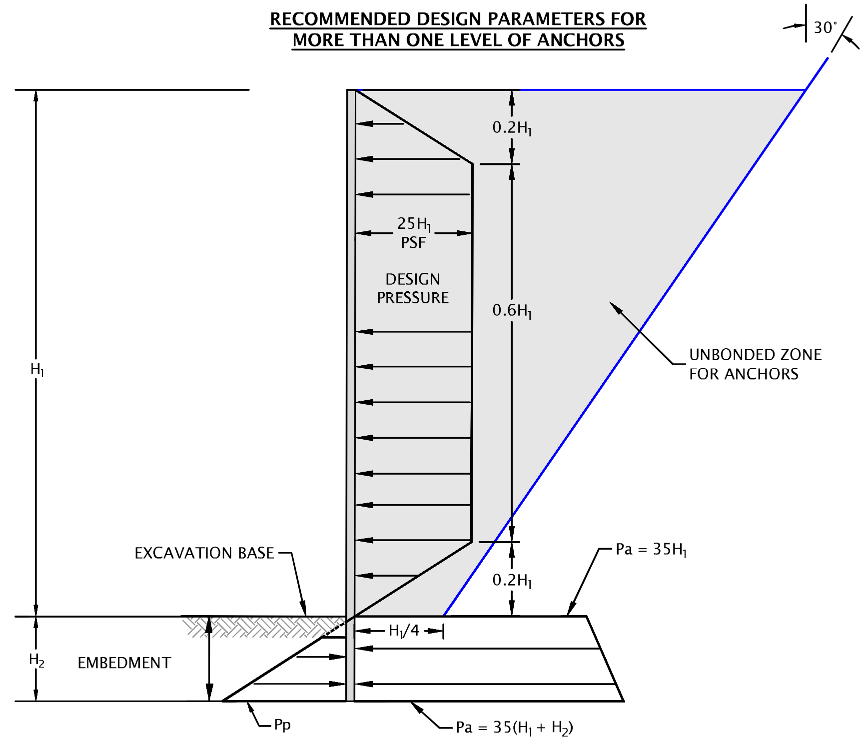
EXPLANATION:

Pp = 350D PCF (ABOVE GROUNDWATER)
H = DEPTH OF SOLDIER PILE IN FEET
D = SOLDIER PILE EMBEDMENT DEPTH
PASSIVE PRESSURE ACTS OVER 3X THE PILE WIDTH
ACTIVE PRESSURE ACTS OVER 1X THE PILE WIDTH BELOW EXCAVATION BASE

NOTES:

1. FIGURE DOES NOT INCLUDE LATERAL EARTH PRESSURES INDUCED BY SLOPED BACKFILL OR SURROUNDING LOADS.
2. LATERAL EARTH PRESSURES BASED ON WATER TABLE BELOW DEPTH OF SOLDIER PILE EMBEDMENT.
3. THE LATERAL EARTH PRESSURES ARE UNFACTORED.

RECOMMENDED DESIGN PARAMETERS FOR MORE THAN ONE LEVEL OF ANCHORS



EXPLANATION:

Pp = $300H_2$ PCF
H_1 = DEPTH OF SOLDIER PILE EXPOSED HEIGHT IN FEET
H_2 = SOLDIER PILE EMBEDMENT DEPTH IN FEET
PASSIVE PRESSURE ACTS OVER 3X THE PILE WIDTH
ACTIVE PRESSURE ACTS OVER 1X THE PILE WIDTH BELOW THE EXCAVATION BASE

NOTES:

1. DOES NOT INCLUDE SURCHARGE OR SEISMIC LOADS.
2. TIEBACKS SHOULD BE LOCKED OFF AT 100 PERCENT OF DESIGN LOAD.
3. THE LATERAL EARTH PRESSURES ARE UNFACTORED.
4. PASSIVE PRESSURE RESISTANCE SHOULD BE NEGLECTED 2 FEET BELOW THE BOTTOM OF THE EXCAVATION.
5. LATERAL EARTH PRESSURES BASED ON WATER TABLE BELOW DEPTH OF SOLDIER PILE EMBEDMENT.

CANTILEVERED AND BRACED WALLS DESIGN CRITERIA

OVERLOOK & SKIDMORE
PORTLAND, OR

FIGURE 5

FOREPROP-6-01

SEPTEMBER 2016

GEODESIGN INC
15575 SW Sequoia Parkway - Suite 100
Portland OR 97224
Off 503.968.8787 Fax 503.968.3068

APPENDIX

APPENDIX

FIELD EXPLORATIONS

GENERAL

Our field explorations consisted of three borings (B-1 through B-3) drilled to depths ranging between 31.5 and 51.5 feet BGS. The borings were drilled on September 6, 2016 by Western States Soil Conservation, Inc. of Hubbard, Oregon, using mud rotary and hollow-stem auger drilling methods. The exploration logs are presented in this appendix.

The approximate locations of our explorations are shown on Figure 2. The exploration locations were chosen based on a preliminary site plan provided to our office by Fore Green Development. The locations of the explorations were determined in the field by pacing from existing site features. This information should be considered accurate only to the degree implied by the methods used.

SOIL SAMPLING

The explorations were observed by a member of our geology staff. We obtained representative samples of the various soil encountered in the explorations for geotechnical laboratory testing. Soil samples were obtained from the borings using SPT sampling methods. SPTs were performed 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 samples were obtained from the split barrel for subsequent classification and index testing. Sampling intervals are shown on the exploration logs

The average efficiency of the automatic SPT hammer used by Western States Soil Conservation, Inc. was 94.2 percent. The calibration testing results are presented at the end of this appendix.

One Shelby tube sample was obtained at a depth of 10 feet BGS in boring B-2 in accordance with ASTM D 1587. A Shelby tube retrieves a relatively undisturbed sample by pushing a thin-wall tube sampler 24 inches ahead of the boring front. Shelby tube samples are preferred for consolidation and strength testing due to the lower level of disturbance.

Sampler types and sampling intervals are shown on the exploration logs

SOIL CLASSIFICATION

The soil samples were classified in accordance with the “Exploration Key” (Table A-1) and “Soil Classification System” (Table A-2), which are presented in this appendix. The exploration logs indicate the depths at which the soil or its characteristics change, although the change could be gradual. A horizontal line between soil types indicates an observed (visual or drill action) change. If the change occurred between sample locations and was not observed or obvious, the depth was interpreted and the change is indicated using a dashed line. 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 log if those classifications differed from the field classifications.

MOISTURE CONTENT

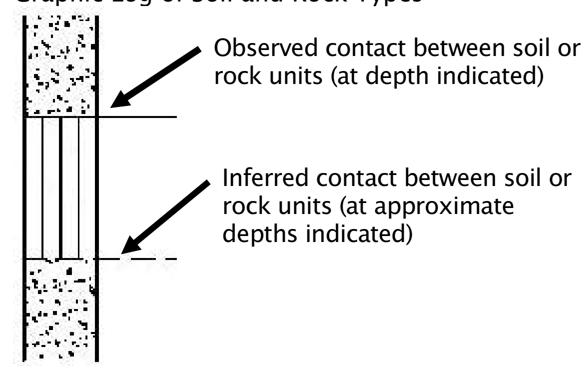
We tested the natural moisture content of selected samples in general accordance with ASTM D 2216. The natural moisture content is a ratio of the weight of the water to soil in a test sample and is expressed as a percentage. The test results are presented in this appendix.

GRAIN-SIZE TESTING

Grain-size testing was performed on selected samples to determine the distribution of soil particle sizes. The testing consisted of particle-size analysis completed in accordance with percent fines determination (percent passing the U.S. Standard No. 200 sieve) completed in general accordance with ASTM D 1140 (P200). 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

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: Fine-Grained Soils	Silt and Clay In: Coarse-Grained Soils	Percent	Sand and Gravel In: Fine-Grained Soils	Sand and Gravel In: 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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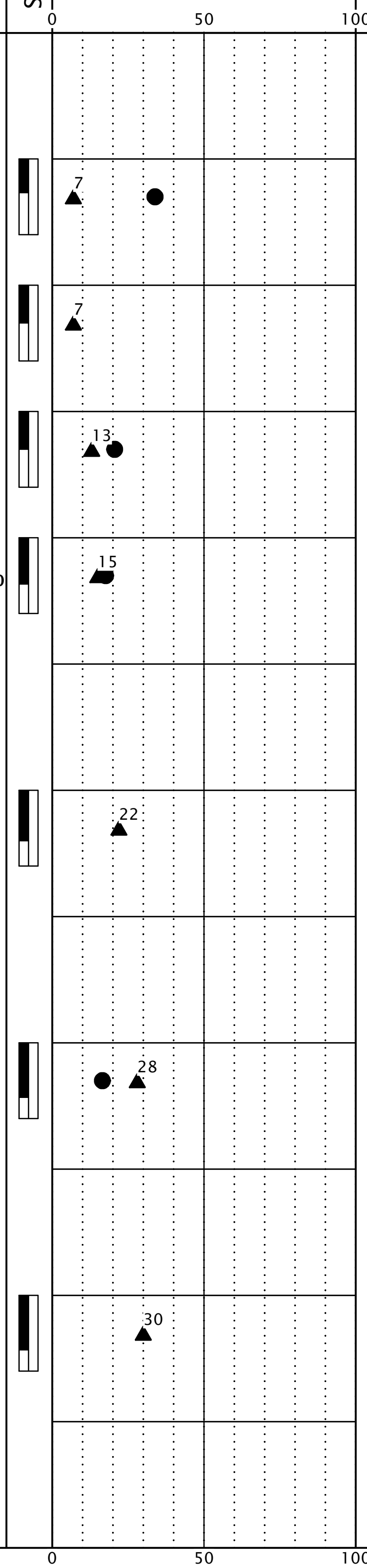
SOIL CLASSIFICATION SYSTEM

TABLE A-2

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		
	Loose, brown, silty SAND (SM); moist, fine.			
7.5	medium dense at 7.5 feet			
10.0			P200	P200 = 24% Infiltration test: 14 inches per hour at 10.2 feet.
25.0	dense at 25.0 feet			
30.0				

▲ BLOW COUNT
● MOISTURE CONTENT %
RQD% CORE REC%



DRILLED BY: Western States Soil Conservation, Inc.
LOGGED BY: CR
COMPLETED: 09/06/16

BORING METHOD: mud rotary and hollow-stem auger (see document text)
BORING BIT DIAMETER: 3 7/8 inches/ 8 inches

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BORING B-1

OVERLOOK & SKIDMORE
PORTLAND, OR

FIGURE A-1

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	▲ BLOW COUNT ● MOISTURE CONTENT % RQD% CORE REC% (0 50 100)	INSTALLATION AND COMMENTS
30.0		medium dense at 30.0 feet				29	
		Exploration completed at a depth of 31.5 feet.	31.5				Surface elevation was not measured at the time of exploration.
32.5		Hammer efficiency factor is 94.2 percent.					
35.0							
37.5							
40.0							
42.5							
45.0							
47.5							
50.0							
52.5							
55.0							
57.5							
60.0							

0 50 100

DRILLED BY: Western States Soil Conservation, Inc. LOGGED BY: CR COMPLETED: 09/06/16

BORING METHOD: mud rotary and hollow-stem auger (see document text) BORING BIT DIAMETER: 3 7/8 inches/ 8 inches

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BORING B-1
(continued)

OVERLOOK & SKIDMORE
PORTLAND, OR

FIGURE A-1

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	BLOW COUNT	INSTALLATION AND COMMENTS
0.0	ASPHALT CONCRETE (3.0 inches).	0.3			
	AGGREGATE BASE (5.0 inches).	0.7			
	Medium stiff, brown SILT (ML), trace sand; moist, sand is fine.				
2.5				6	
5.0				7	
7.5	brown with orange mottles at 7.5 feet			6	
10.0			PP		PP = 2.0 tsf
	Medium dense, brown, silty SAND (SM); moist, fine.	11.5			
12.5				10	
15.0	loose at 15.0 feet			8	
17.5					
20.0	medium dense at 20.0 feet			15	
22.5					
25.0	loose at 25.0 feet			9	
27.5					
30.0					

▲ BLOW COUNT
● MOISTURE CONTENT %
RQD% CORE REC%
0 50 100

DRILLED BY: Western States Soil Conservation, Inc. LOGGED BY: CR COMPLETED: 09/06/16

BORING METHOD: mud rotary (see document text) BORING BIT DIAMETER: 3 7/8 inches

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BORING B-2

OVERLOOK & SKIDMORE
PORTLAND, OR

FIGURE A-2

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE	▲ BLOW COUNT / ● MOISTURE CONTENT % / RQD% / CORE REC% (0 – 50 – 100)	INSTALLATION AND COMMENTS
30.0	medium dense at 30.0 feet		P200		▲ 17 ●	P200 = 15%
32.5						
35.0					▲ 17	
37.5						
	Medium dense, brown, silty SAND (SM); moist, fine.	38.0				
40.0					▲ 12	
42.5						
45.0	fine to medium at 45.0 feet				▲ 19	
47.5						
	Medium dense, brown SAND with silt (SP-SM); moist, fine to medium.	48.0				
50.0					▲ 24	
	Exploration completed at a depth of 51.5 feet. Hammer efficiency factor is 94.2 percent.	51.5				Surface elevation was not measured at the time of exploration.
52.5						
55.0						
57.5						
60.0						

DRILLED BY: Western States Soil Conservation, Inc. LOGGED BY: CR COMPLETED: 09/06/16

BORING METHOD: mud rotary (see document text) BORING BIT DIAMETER: 3 7/8 inches

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BORING B-2
(continued)

OVERLOOK & SKIDMORE
PORTLAND, OR

FIGURE A-2

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	▲ BLOW COUNT ● MOISTURE CONTENT % RQD% CORE REC% (0 50 100)	INSTALLATION AND COMMENTS
0.0		ASPHALT CONCRETE (3.0 inches).	0.3				
		AGGREGATE BASE (4.0 inches).	0.6				
		Stiff, brown SILT (ML), trace sand; moist, sand is fine.					
2.5				PP		10	PP = 1.5 tsf
5.0		minor sand at 5.0 feet		PP		15	PP = 1.5 tsf
7.5		Loose, brown, silty SAND (SM); moist, fine.	7.0			7	
10.0						8	
12.5							
15.0		medium dense at 15.0 feet				15	
17.5							
20.0						19	
22.5							
25.0						23	
27.5							
30.0							

DRILLED BY: Western States Soil Conservation, Inc. LOGGED BY: CR COMPLETED: 09/06/16

BORING METHOD: mud rotary (see document text) BORING BIT DIAMETER: 3 7/8 inches

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FOREPROP-6-01

BORING B-3

SEPTEMBER 2016

OVERLOOK & SKIDMORE
PORTLAND, OR

FIGURE A-3

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	▲ BLOW COUNT ● MOISTURE CONTENT % RQD% CORE REC% (0–50–100)	INSTALLATION AND COMMENTS
30.0		dense at 30.0 feet				41	
32.5							
35.0		medium dense at 35.0 feet		P200		23	P200 = 33%
37.5							
40.0						25	
		Exploration completed at a depth of 41.5 feet. Hammer efficiency factor is 94.2 percent.	41.5				Surface elevation was not measured at the time of exploration.
42.5							
45.0							
47.5							
50.0							
52.5							
55.0							
57.5							
60.0							

DRILLED BY: Western States Soil Conservation, Inc. LOGGED BY: CR COMPLETED: 09/06/16

BORING METHOD: mud rotary (see document text) BORING BIT DIAMETER: 3 7/8 inches

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SEPTEMBER 2016

BORING B-3
(continued)

OVERLOOK & SKIDMORE
PORTLAND, OR

FIGURE A-3

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		34							
B-1	7.5		21							
B-1	10.0		18				24			
B-1	20.0		17							
B-2	7.5		33							
B-2	15.0		33							
B-2	30.0		26				15			
B-3	5.0		41							
B-3	10.0		22							
B-3	20.0		20							
B-3	35.0		21				33			

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SEPTEMBER 2016

SUMMARY OF LABORATORY DATA

OVERLOOK & SKIDMORE
PORTLAND, OR

FIGURE A-4

WSSC-7-01 - TEST BORING B-6 25FT
OP: WMN

TRUCK NO. 5
Date: 30-May-2015

AR: 1.41 in²
LE: 29.42 ft
WS: 16,807.9 f/s

SP: 0.492 k/ft³
EM: 30,000 ksi
JC: 0.00 []

ETR: Energy Transfer Ratio
EMX: Max Transferred Energy
CSB: Compression Stress at Bottom
BPM: Blows per Minute
FFS: Force Full Scale
DMX: Maximum Displacement
SFR: Skin friction w/ damping correction
MEX: Maximum Strain
VMX: Maximum Velocity

BL#	depth	BLC	ETR	EMX	CSB	BPM	FFS	DMX	SFR	MEX	VMX
	ft	bl/ft	(%)	k-ft	ksi	bpm	kips	in	kips	µE	f/s
10	25.00	6	87.8	0.3	0.0	42.9	60	1.16	0	1,087	17.7
11	25.18	6	92.2	0.3	0.0	43.1	60	1.86	0	1,119	18.6
12	25.36	6	95.3	0.3	0.0	43.1	60	0.87	0	1,116	18.4
13	25.54	6	94.2	0.3	0.0	43.1	60	1.08	0	1,183	18.6
14	25.71	6	88.3	0.3	0.0	43.3	60	0.66	0	1,113	17.4
15	25.89	6	90.2	0.3	0.0	43.1	60	1.41	0	1,064	17.6
16	26.07	6	95.2	0.3	0.0	43.2	60	1.38	0	1,105	18.3
17	26.25	6	86.0	0.3	0.0	43.2	60	0.90	0	1,060	17.0
18	26.43	6	88.7	0.3	0.0	43.2	60	1.02	0	1,139	17.3
19	26.61	6	89.6	0.3	0.0	43.2	60	1.53	0	1,125	18.0
20	26.79	6	93.7	0.3	0.0	43.1	60	1.02	0	1,150	18.0
21	26.96	6	91.3	0.3	0.0	43.2	60	1.44	0	1,098	17.4
22	27.14	6	93.2	0.3	0.0	43.1	60	0.91	0	1,123	17.9
23	27.32	6	90.9	0.3	0.0	43.2	60	0.98	0	1,111	17.3
24	27.50	6	94.6	0.3	0.0	43.1	60	0.85	0	1,201	18.0
25	27.68	6	95.9	0.3	0.0	43.1	60	0.89	0	1,197	18.1
26	27.86	6	92.4	0.3	0.0	43.2	60	1.63	0	1,066	17.0
27	28.04	6	85.8	0.3	0.0	43.2	60	0.52	0	1,116	16.0
28	28.21	6	90.5	0.3	0.0	43.2	60	0.62	0	1,120	16.6
29	28.39	6	89.1	0.3	0.0	43.2	60	0.97	0	1,133	16.4
30	28.57	6	89.5	0.3	0.0	43.4	60	0.62	0	1,146	16.2
31	28.75	6	90.7	0.3	0.0	43.0	60	0.80	0	1,092	16.3
38	30.00	6	92.2	0.3	0.0	48.0	60	0.92	0	1,004	18.2
39	30.17	6	90.3	0.3	0.0	47.8	60	1.17	0	1,025	18.2
40	30.33	6	94.2	0.3	0.0	47.9	60	0.90	0	1,008	18.2
41	30.50	6	96.5	0.3	0.0	47.5	60	1.02	0	1,027	18.3
42	30.67	6	92.7	0.3	0.0	47.9	60	1.27	0	1,000	18.1
43	30.83	6	91.8	0.3	0.0	47.9	60	1.00	0	1,018	18.4
44	31.00	6	94.9	0.3	0.0	47.8	60	1.42	0	1,023	18.1
45	31.17	6	95.2	0.3	0.0	47.7	60	1.20	0	1,072	18.4
46	31.33	6	97.9	0.3	0.0	47.8	60	1.57	0	998	18.0
47	31.50	6	93.0	0.3	0.0	47.8	60	0.90	0	1,008	18.0
48	31.67	6	91.1	0.3	0.0	47.7	60	0.92	0	981	17.7
49	31.83	6	94.3	0.3	0.0	48.1	60	1.01	0	1,013	18.2
50	32.00	6	95.1	0.3	0.0	47.8	60	0.92	0	1,073	18.5
51	32.17	6	90.9	0.3	0.0	47.8	60	0.72	0	1,003	17.7
52	32.33	6	93.5	0.3	0.0	47.7	60	0.91	0	1,005	17.8
53	32.50	6	97.8	0.3	0.0	48.0	60	0.96	0	1,065	18.4
54	32.67	6	100.2	0.4	0.0	47.8	60	1.31	0	1,017	18.2
55	32.83	6	91.6	0.3	0.0	47.6	60	0.64	0	1,054	18.1
56	33.00	6	84.5	0.3	0.0	48.0	60	0.80	0	983	17.3
57	33.17	6	88.4	0.3	0.0	47.9	60	0.40	0	1,050	18.1
58	33.33	6	99.6	0.3	0.0	47.6	60	1.72	0	1,012	17.9
68	35.00	6	96.0	0.3	0.0	46.9	60	0.85	0	1,023	17.8
69	35.12	8	89.8	0.3	0.0	47.0	60	0.70	0	972	17.1

WSSC-7-01 - TEST BORING B-6 25FT
OP: WMN

TRUCK NO. 5
Date: 30-May-2015

BL#	depth ft	BLC bl/ft	ETR (%)	EMX k-ft	CSB ksi	BPM bpm	FFS kips	DMX in	SFR kips	MEX µE	VMX f/s
70	35.24	8	96.5	0.3	0.0	46.9	60	0.75	0	1,089	18.4
71	35.37	8	73.6	0.3	0.0	46.5	60	0.96	0	906	15.5
72	35.49	8	99.6	0.3	0.0	47.4	60	0.67	0	1,028	18.3
73	35.61	8	93.9	0.3	0.0	47.0	60	0.68	0	1,018	17.5
74	35.73	8	93.0	0.3	0.0	47.0	60	0.71	0	1,007	17.6
75	35.85	8	93.1	0.3	0.0	46.9	60	0.94	0	1,014	17.3
76	35.98	8	97.3	0.3	0.0	46.9	60	1.05	0	1,013	17.7
77	36.10	8	92.0	0.3	0.0	47.1	60	0.56	0	1,024	17.3
78	36.22	8	95.5	0.3	0.0	46.9	60	0.82	0	1,015	17.6
79	36.34	8	96.7	0.3	0.0	47.0	60	1.26	0	1,037	17.9
80	36.46	8	97.5	0.3	0.0	47.0	60	0.66	0	1,051	18.2
81	36.59	8	99.7	0.3	0.0	47.1	60	0.57	0	1,071	18.4
82	36.71	8	93.1	0.3	0.0	47.0	60	0.75	0	1,041	17.6
83	36.83	8	101.8	0.4	0.0	46.9	60	1.14	0	1,043	18.4
84	36.95	8	93.0	0.3	0.0	47.0	60	0.54	0	1,033	17.6
85	37.07	8	101.3	0.4	0.0	46.9	60	1.11	0	1,076	18.4
86	37.20	8	96.0	0.3	0.0	47.0	60	0.75	0	1,030	18.1
87	37.32	8	94.5	0.3	0.0	47.1	60	0.38	0	1,069	18.0
88	37.44	8	100.3	0.4	0.0	46.9	60	1.11	0	1,079	18.4
89	37.56	8	103.0	0.4	0.0	47.0	60	1.24	0	1,065	18.4
90	37.68	8	92.4	0.3	0.0	46.9	60	0.61	0	1,022	17.7
91	37.80	8	97.4	0.3	0.0	47.0	60	0.46	0	1,034	18.4
92	37.93	8	94.7	0.3	0.0	47.0	60	0.83	0	1,044	18.0
93	38.05	8	97.4	0.3	0.0	47.1	60	0.98	0	1,026	17.8
94	38.17	8	97.9	0.3	0.0	46.9	60	0.75	0	1,030	17.9
95	38.29	8	95.1	0.3	0.0	46.9	60	0.44	0	1,050	18.0
96	38.41	8	93.9	0.3	0.0	47.0	60	0.34	0	1,046	17.9
97	38.54	8	94.2	0.3	0.0	47.1	60	0.33	0	1,069	18.4
109	40.00	8	95.5	0.3	0.0	49.4	60	0.81	0	1,056	18.7
110	40.12	8	96.5	0.3	0.0	49.5	60	1.18	0	1,080	18.9
111	40.24	8	99.1	0.3	0.0	49.6	60	1.42	0	1,119	19.4
112	40.37	8	97.5	0.3	0.0	49.6	60	1.07	0	1,110	19.0
113	40.49	8	93.5	0.3	0.0	49.3	60	1.35	0	1,041	18.8
114	40.61	8	91.0	0.3	0.0	49.4	60	0.66	0	1,091	17.7
115	40.73	8	99.7	0.3	0.0	49.4	60	0.78	0	1,084	19.6
116	40.85	8	97.6	0.3	0.0	49.5	60	1.32	0	1,114	19.7
117	40.98	8	97.9	0.3	0.0	49.4	60	1.24	0	1,070	19.5
118	41.10	8	93.1	0.3	0.0	49.5	60	1.26	0	1,055	18.9
119	41.22	8	97.5	0.3	0.0	49.5	60	1.29	0	1,133	19.6
120	41.34	8	96.8	0.3	0.0	49.3	60	1.29	0	1,134	19.2
121	41.46	8	94.7	0.3	0.0	49.5	60	0.79	0	1,107	18.4
122	41.59	8	94.3	0.3	0.0	49.4	60	0.55	0	1,044	17.9
123	41.71	8	96.3	0.3	0.0	49.4	60	2.00	0	1,073	19.4
124	41.83	8	98.9	0.3	0.0	49.4	60	0.68	0	1,114	19.0
125	41.95	8	95.9	0.3	0.0	49.5	60	0.66	0	1,092	18.4
126	42.07	8	98.3	0.3	0.0	49.4	60	1.12	0	1,069	18.3
127	42.20	8	95.6	0.3	0.0	49.3	60	1.41	0	1,075	18.0
128	42.32	8	96.9	0.3	0.0	49.6	60	0.84	0	1,079	18.1
129	42.44	8	94.7	0.3	0.0	49.6	60	0.47	0	1,146	18.5
		Average	94.2	0.3	0.0	46.8	60	0.96	0	1,064	18.0
		Std. Dev.	4.2	0.0	0.0	2.2	0	0.34	0	52	0.7

Total number of blows analyzed: 94

BL#	Sensors
10-129	F3: [SPT B1] 217.8 (1.00); F4: [SPT B2] 218.9 (1.00); A3: [K0232] 290.0 (1.00); A4: [K0231] 325.0 (1.00)

BL#	Comments
31	N: 8,10,11
38	LE = 35.10 ft; WC = 16,715.9 f/s
58	5, 7, 14
68	LE = 40.10 ft; WC = 16,794.3 f/s
97	N: 8,13,17
109	LE = 45.10 ft; WC = 16,714.3 f/s
129	N: 10,10,11

Time Summary

Drive	29 seconds	4:13 PM - 4:13 PM (5/30/2015) BN 10 - 31
Stop	37 minutes 37 seconds	4:13 PM - 4:51 PM
Drive	25 seconds	4:51 PM - 4:51 PM BN 38 - 58
Stop	23 minutes 16 seconds	4:51 PM - 5:14 PM
Drive	37 seconds	5:14 PM - 5:15 PM BN 68 - 97
Stop	26 minutes 48 seconds	5:15 PM - 5:42 PM
Drive	24 seconds	5:42 PM - 5:42 PM BN 109 - 129

Total time [01:29:38] = (Driving [00:01:55] + Stop [01:27:43])

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
g	gravitational acceleration (32.2 feet/second^2)
H:V	horizontal to vertical
IBC	International Building Code
ksf	kips per square foot
MCE	maximum considered earthquake
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

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