PRELIMINARY STORMWATER REPORT

Elmonica Mixed Use
17160 SW Baseline Rd,
Beaverton, OR, 97006

March 12, 2023

Prepared by:

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JAN. 12, 2021
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EXPIRES: 6/30/2024

TABLE OF CONTENTS

Attachments

I. PROJECT DESCRIPTION

The purpose of this document is to describe the stormwater management plan that will be used to capture, treat, and convey stormwater runoff from the Elmonica mixed use project, pursuant to City of Beaverton (COB) and Clean Water Services (CWS) standards. This report discusses on-site stormwater management only; fee-in-lieu is proposed for the stormwater management of the public right-of-way improvements and a justification memo provided by Janet Turner Engineering (JTE) can be found as Attachment G.

The proposed project includes two tax lots with a total combined area of approximately 5.2 acres – these lots are configured in a "U" shape around an adjacent corner lot and are proposed to be consolidated into one lot for the Elmonica project.

When finished, the proposed site will include 3 new, 5-story apartment buildings, a separate ~2,650 leasing/clubhouse building and pool, as well as a separate retail building. Development will include vehicular and pedestrian pavement improvements and all necessary utility infrastructure to serve the proposed improvements. See Attachment A for a vicinity map and overall site plan.

II. EXISTING CONDITIONS

The existing land is partially developed with multiple business enterprises and buildings, with several paved parking areas, all of which are to be removed in order to accommodate this project.

On-site soil conditions do not allow for stormwater infiltration – the site has a combination of poorly draining soils and seasonally high/perched groundwater. The groundwater table was found to be between 7 and 15 feet below ground surface, and groundwater seepage was observed at depths between 2 and 10 feet in 5 of 6 test pits. The Geotechnical Report documenting this information is included as Attachment F of this report.

Generally, the site slopes from the northern end of the site down to the southeast corner where stormwater is able to enter a storm culvert and flow under the TriMet tracks to be discharged to what appears to be a drainage channel to the south. There appears to be no existing stormwater structures or pipes currently on-site.

III. PROPOSED CONDITIONS

This report and stormwater design are preliminary and based on the land use plan set dated February 27, 2023. The proposed on-site improvements will modify/disturb an area of approximately 226,245 sf. There will be additional improvements made within the public right-of-way (R.O.W.) to provide access to the site including two new driveways and public sidewalk (not discussed in this report).

The proposed project is required to treat and control all stormwater runoff before it leaves the site and enters the public storm system. The water quality portion of the proposed stormwater management plan will be handled primarily by LIDA flow-through planters and proprietary stormwater treatment devices.

Due to the low infiltration capacity and seasonally high groundwater table on-site (per attached Geotechnical Report), the detention/hydromodification requirements will be met using an underground storage system and flow control device.

IV. REFERENCES

Clean Water Services Design and Construction Standards (CWS DCS)

City of Beaverton Development Code (COB DC)

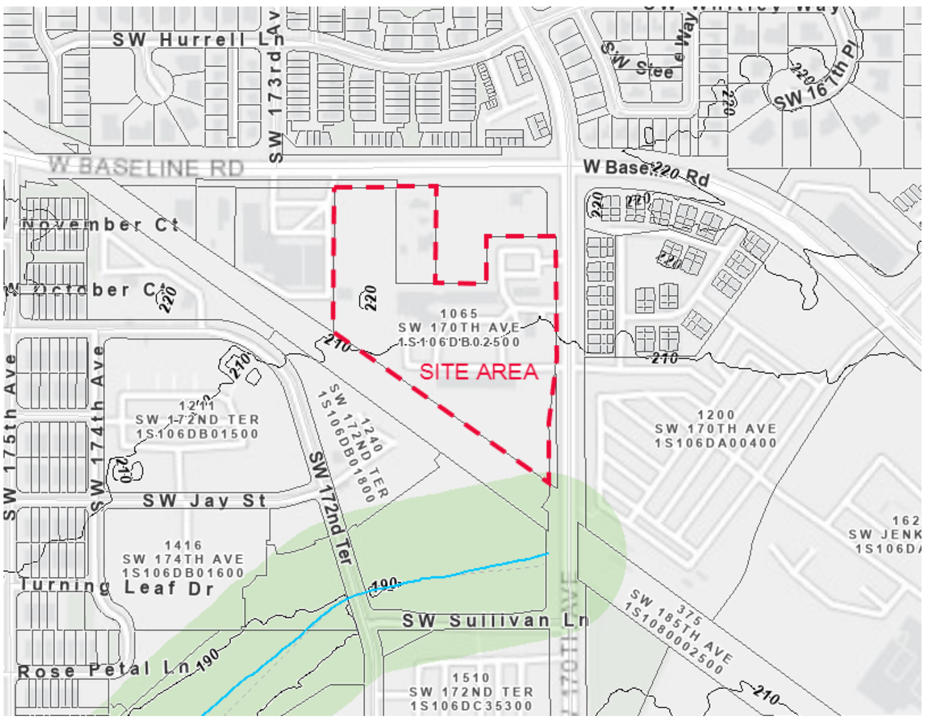
City of Beaverton Engineering Design Manual (COB EDM)

USB NRCS Web Soil Survey Map

Geotechnical Report: Report of Geotechnical Engineering Services, prepared by NV5 for Rembold Properties, LLC, dated January 12, 2022

V. DESIGN AND CONSTRUCTION STANDARDS

The proposed development will provide stormwater management in accordance with Chapter 4 of the CWS DCS, as well as Section 500 of the COB EDM. This project will impact an area >80,000 sf and will therefore require hydromodification to meet CWS standards. Given the size of the project and its location, this site falls under "Category 2" of CWS Table 4-2 below.



Per the CWS Hydromodification Planning Tool, this site falls into the "Developed Area" Development Class and "Low" Risk Level

TABLE 4-2
HYDROMODIFICATION APPROACH PROJECT CATEGORY TABLE

Development Class/ Risk Level	Small Project 1,000 – 12,000 SF	Medium Project >12,000 – 80,000 SF	Large Project > 80,000 SF
Expansion/High	Category 1	Category 3	Category 3
Expansion/ Moderate			
Expansion/ Low		Category 2	
Developed/ High		Category 3	
Developed/ Moderate		Category 2	Category 2
Developed/ Low			

VI. HYDROLOGY

The City of Beaverton EDM and Clean Water Services DCS were used to design the proposed underground detention system. Given the "Category 2" classification and low infiltration capacity on-site, the proposed underground detention system was sized using the "Peak Flow Matching" method per CWS DCS.

Impervious Areas:

The pre and post-development conditions on-site are summarized in the table below:

PRE AND POST DEVELOPMENT IMPERVIOUS AREA			
Site Condition	**Pervious Area (sf)**	**Impervious Area (sf)**	**Total Site Area (sf)**
Pre-Developed	110,518	115,727	226,245
Post-Developed	36,645	189,600	226,245

Detention Sizing Requirements:

The "Peak Flow Matching" detention method per CWS section 4.08.6 was used to size the required underground detention facility.

Hydromodification:

The peak flow matching detention for hydromodification method requires that no more then 1/2 or 50% of the pre-developed 2-year peak flow rate be released from the developed site.

Flow control for conveyance:

The developed peak flow rates for the 10 and 25-year storm events must be equal to or less than the pre-developed peak flow rates on-site.

Detention Sizing Method:

The pre and post-developed peak flows on-site were found using a HydraFlow model (see Attachment C). The model used an SCS or Soil Conservation Service design storm based on precipitation values from CWS Table 4-4. Because of the significant amount of modified impervious surfaces on-site, a curve number (CN) of 75 was used as the pre-developed CN for all of the impervious area on-site. The post-developed conditions on site were analyzed using a CN of 92, which is a composite CN based on the new and modified impervious areas on-site. A table including all of the requisite pre and post-development release rates can be found on the next page.

Water quantity will be controlled via underground detention and flow attenuation. A CWS flow control manhole with COB modifications will be used to control the flow leaving the underground detention system. One 2.25" orifice and one 9"x12" rectangular weir will control the flow so that it meets COB EDM and CWS DCS water quantity requirements. The orifice sizing calculations were done in HydraFlow. The water quantity/detention and orifice sizing calculations are provided as Attachment C of this report.

The pre and post developed release rates are in the table below:

PRE AND POST DEVELOPMENT RELEASE RATES			
Design Storm	**Pre-Developed Q (cfs)**	**Allowable Post-Development Q (cfs)**	**Modeled Post-Development Q (cfs)**
2-year 24-hour	0.538	0.269 (50% of 2-year)	0.265
10-year 24-hour	1.387	1.387	0.865
25-year 24-hour	1.851	1.851	1.498

TABLE 4-4

Recurrence Interval	Total 24-Hour Precipitation Depth (water equivalent inches)
2-year	2.5
5-year	3.10
10-year	3.45
25-year	3.90

VII. STORMWATER RUNOFF TREATMENT

The City of Beaverton EDM and Clean Water Services DCS were used to design the proposed stormwater treatment facilities. For water quality treatment, LIDA facilities are designed per the CWS Simplified Sizing requirements in section 4.08.4 of the CWS Design and Construction Standards. Proprietary treatment is sized to COB and manufacturer standards.

Water Quality Requirements:

CWS DCS Section 4.08.1 requires that "all new impervious surfaces and three times the modified impervious surface, up to the total existing impervious surface on the site" is subject to water quality treatment.

The proposed development includes 73,873 sf of new impervious area and 115,727 sf of existing/modified impervious area. Since three times the modified impervious surface is greater than the total existing impervious surface, the site area required to be treated is the sum of the new and modified impervious surfaces (189,600 sf). The CWS water quality requirement is met.

Stormwater Management (SWM) Facility Types:

For this development, 3rd order "Private Vegetated SWM" and 4th order "Private Proprietary Treatment" facilities will be used to meet COB and CWS water quality treatment requirements. 1st order "Enhancement and/or Expansion of Existing Public SWM" facilities are not feasible to use for this project due to the lack of existing SWM facilities between the project site and the stormwater outfall location. 2nd order "New Public Vegetated SWM" facilities are also not feasible to use for this project due to insufficient space within the public right-of-way or adjacent to the public right-of-way within an access easement.

3rd Order Private Vegetated SWM Sizing (per COB):

A simplified sizing approach can be used when contributing impervious areas are <15,000 square feet. The simplified sizing approach assumes a 6% sizing factor to size SWM planters.

The proposed site is broken down into "Drainage Management Areas" (DMAs) that are less than 15,000 square feet. The DMAs and their respective Surface Water Management (SWM) planters can be found in the "Drainage Management Area Map" (see Attachment B). As such, most of the new and modified impervious surface will be treated by a combination of structural flow-through planters and street-side planters that have been sized using the "Simplified LIDA Sizing" approach found in section 4.09.11 of the CWS DCS.

4th Order Private Proprietary Treatment Facility Sizing (per COB):

Contech Stormfilters are proposed in areas on-site where LIDA facilities are not feasible. Proposed Stormfilter locations are shown in the DMA map (see Attachment B).

Proprietary treatment facility sizing is based on Section 510 of the COB EDM. A water quality flow rate is determined from the equation below, which is then used to determine the size of the required proprietary stormwater treatment facility:

$$\text{Water Quality Flow (cfs)} = \frac{0.36\text{ (in.)} \times \text{Area (sq. ft.)}}{12\text{(in/ft.) } (\mathbf{3}\text{ hr.) (60 min/hr.) (60 sec/min.)}}$$

Per the COB EDM and the manufacturer, an 18" Contech Stormfilter can treat 12,027 sf of impervious area and a 27" Contech Stormfilter can treat 18,040 sf of impervious area.

VIII. REVIEW OF DOWNSTREAM CONVEYANCE

Offsite Downstream 24" Culvert Capacity Analysis:

A capacity study was performed for the 25-year storm on the existing 24" concrete culvert which ultimately discharges to a wetland area. Seeing that the proposed site flows are designed not to exceed the existing condition 25-year storm, the existing drainage area is only considered in this study. Runoff flows to the culvert are computed based on the rational method as referenced in CWS DCS Section 5.04.2 and the Oregon Department of Transportation (ODOT) Hydraulics Manual.

- Area (A): A single basin is used for this study. In general, per the ODOT Hydraulics Manual, the drainage area cannot exceed 200 acres. To the best of our knowledge the drainage area limits are shown in Attachment D. This area is based on Beaverton Utilities Viewer and Hydromodification Planning tool from arcGIS, site photos, and google earth.
  - 13.6 acres
- Time of Concentration (Tc): The ODOT Hydraulics Manual is used for time of concentration computation. See Attachment D for calculations
  - 35 minutes
- Intensity (I): See the IDF table (Standard Drawing 1275) in Attachment D.
  - 1.28
- Runoff Coefficient (C): Approximate landscape areas were scaled out on google maps and account for 32% or 4.4 acres of the existing landscape area, see Attachment D for landscape drainage area. Pavement and roads account for the remaining 68% or 9.2 acres of the drainage area. Drainage coefficients are based on the ODOT Hydraulic Manual runoff coefficient table. For the 25-year storm an adjustment factor of 1.1 is used, see Attachment D.
  - Drainage Coefficient: Pavement and Roofs 0.90

  - Drainage Coefficient: Lawns 0.22
  - Drainage Coefficient Ratio for site 0.68
- Site Flow: Q=(1.1)(13.6)(1.28)(0.68)
  - Q = 13.02 cfs

The existing concrete culvert has a running slope of approximately 0.5%. The minimum pipe required for these design parameters was computed in civil tools pro and is shown below.

Flow (cfs)	Diameter (in)	Manning's N	Slope (%)
13.02	22.22	0.013	0.50

Based on the table above the minimum pipe diameter required to convey the 13.02cfs is 22.22". In conclusion, the existing 24" culvert has sufficient capacity to convey the existing upstream demand.

Additional Downstream Facility Analysis:

Per the CWS section 2.04.2.m.3, the flow analysis shall continue downstream to a point until the additional flow constitutes less than 5 percent of the total tributary drainage flow.

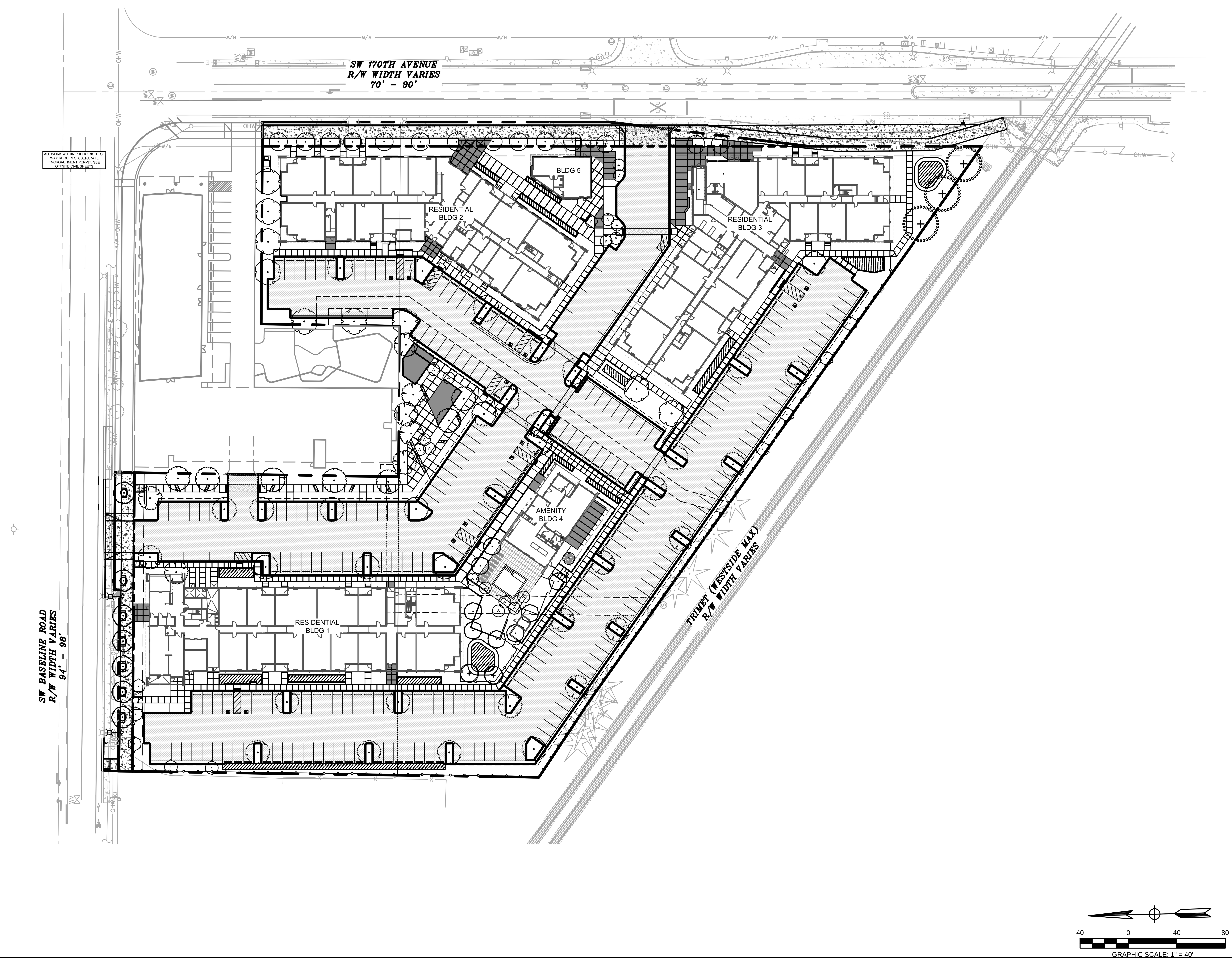
After the stormwater passes through the 24" culvert, it proceeds to drain to an existing wetland area. Based off arcGIS maps, there are a total of five existing outfall culverts in this location that drain offsite upstream tributary areas. The total approximate tributary area at this point of compliance is 150.6 acres, see Attachment E. A similar rational method study of the 25-year storm was performed on this drainage area to verify that the post development flow rate is less than 5% of the total tributary drainage flow. The total tributary area drainage flow calculation is shown below.

- Area (A): A single basin is used for this study. In general, per the ODOT Hydraulics Manual, the drainage area cannot exceed 200 acres. To the best of our knowledge the drainage area limits are shown in Attachment E. This area is based on Beaverton Utilities Viewer and Hydromodification Planning tool from arcGIS, site photos, and google earth.
  - 150.6 acres
- Time of Concentration (Tc): The ODOT Hydraulics Manual is used for time of concentration computation. See Attachment E for calculations
  - 91 minutes
- Intensity (I): See the IDF table (Standard Drawing 1275) in Attachment E.
  - 0.72
- Runoff Coefficient (C): Approximate large landscape areas were scaled out on google maps and account for 29% or 44.1 acres of the existing landscape area. To account for smaller landscape strips throughout the drainage area an additional 15% or 16 acres of the remaining area is presumed to be landscape. This amounts to 40% or 60.1 acres of the total existing site consisting of landscape area, see Attachment E for landscape drainage area. Pavement and roads account for the remaining 60% or 90.5 acres of the drainage area. Drainage coefficients are based on the ODOT Hydraulic Manual runoff coefficient table. For the 25-year storm an adjustment factor of 1.1 is used, see Attachment D.
  - Drainage Coefficient: Pavement and Roofs 0.90

    - Drainage Coefficient: Lawns 0.22
    - Drainage Coefficient Ratio for site 0.63
- Site Flow: Q=(1.1)(150.6)(0.72)(0.63)
    - Q = 75.14 cfs

As shown in the "Pre and Post Release Rates" table of this report the 25-year flow from the proposed site is 1.498 cfs. This proposed flow constitutes for only 2% of the 75.14cfs tributary upstream flow. Based on this finding, the downstream hydraulic analysis in concluded.

ATTACHMENT A



NOT FOR CONSTRUCTION

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ELMONICA MIXED USE

SW 170TH AND W BASELINE

REMBOLD PROPERTIES

REVISION	DATE	REASON FOR ISSUE
A	2/27/23	LAND USE REVISION

OVERALL SITE PLAN

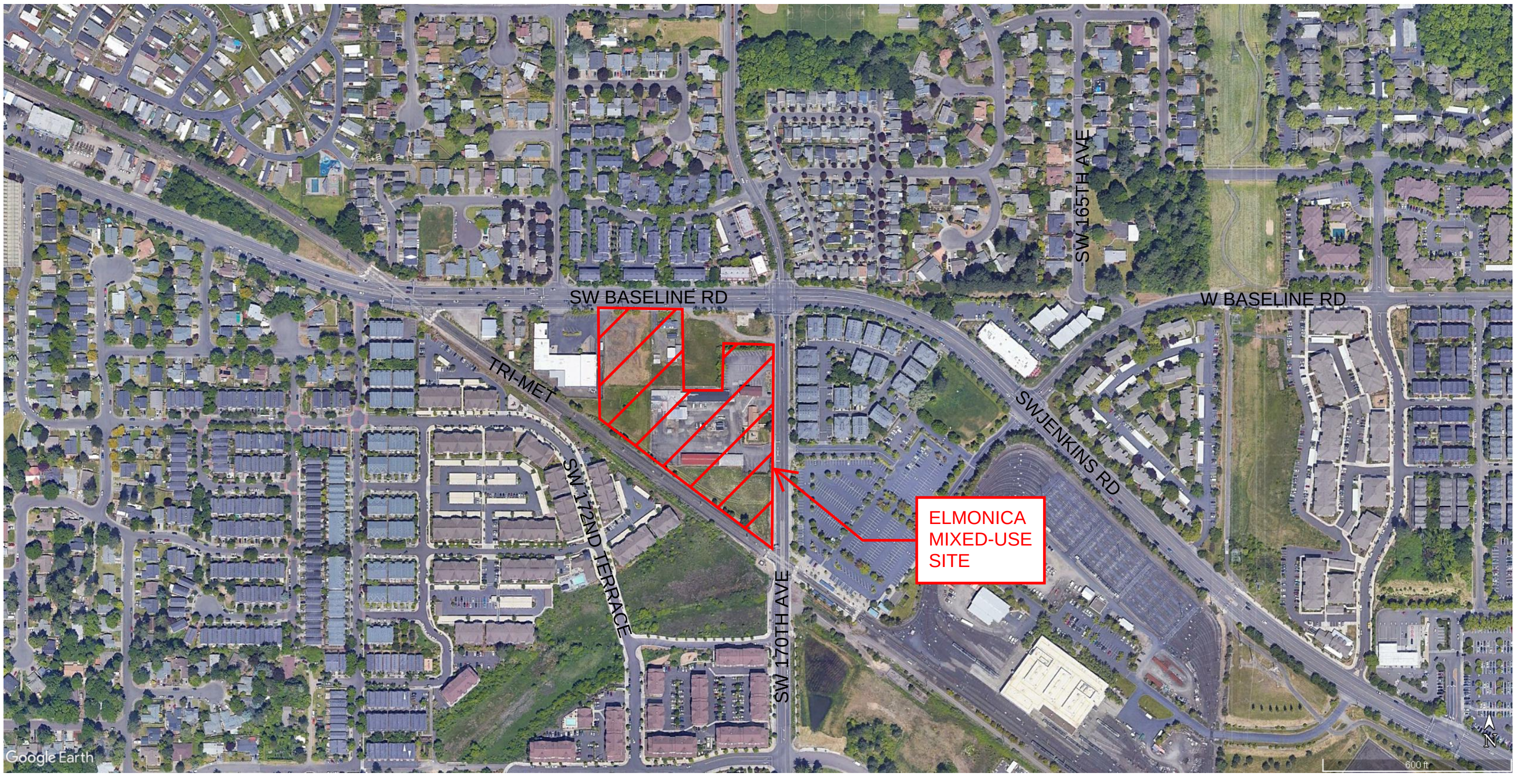
LAND USE SET

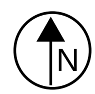
DATE 2.27.2023

PROJECT NUMBER 212225

SHEET NUMBER C1.1

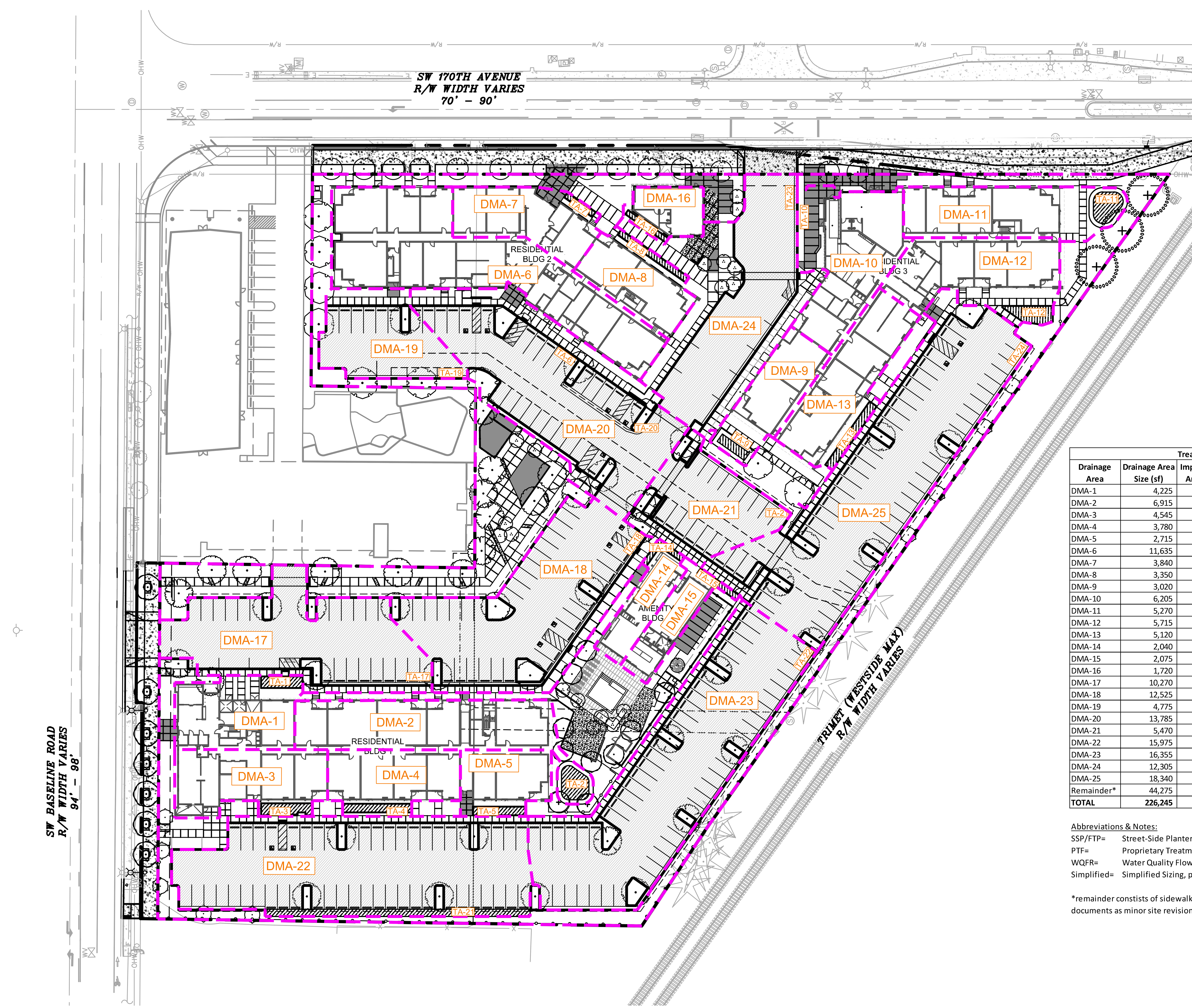
ELMONICA MIXED-USE VICINITY MAP





NOT TO SCALE

ATTACHMENT B



Treatment Measure Summary Table						
Drainage Area	Drainage Area Size (sf)	Impervious Area (sf)	Pervious Area (sf)	Treatment Type	Sizing Method	Treatment Area Required (sf)
DMA-1	4,225	3,450	775	SSP/FTP	simplified	207
DMA-2	6,915	6,125	790	SSP/FTP	simplified	368
DMA-3	4,545	4,135	410	SSP/FTP	simplified	248
DMA-4	3,780	3,255	525	SSP/FTP	simplified	195
DMA-5	2,715	2,255	460	SSP/FTP	simplified	135
DMA-6	11,635	11,635	-	PTF	WQFR	-
DMA-7	3,840	3,620	220	SSP/FTP	simplified	217
DMA-8	3,350	2,850	500	SSP/FTP	simplified	171
DMA-9	3,020	2,465	555	SSP/FTP	simplified	148
DMA-10	6,205	5,720	485	PTF	WQFR	-
DMA-11	5,270	4,100	1,170	SSP/FTP	simplified	246
DMA-12	5,715	5,005	710	SSP/FTP	simplified	300
DMA-13	5,120	4,465	655	SSP/FTP	simplified	268
DMA-14	2,040	1,915	125	SSP/FTP	simplified	115
DMA-15	2,075	1,955	120	SSP/FTP	simplified	117
DMA-16	1,720	1,545	175	SSP/FTP	simplified	93
DMA-17	10,270	10,010	260	PTF	WQFR	-
DMA-18	12,525	11,760	765	PTF	WQFR	-
DMA-19	4,775	4,350	425	PTF	WQFR	-
DMA-20	13,785	12,195	1,590	PTF	WQFR	-
DMA-21	5,470	5,470	-	PTF	WQFR	-
DMA-22	15,975	14,645	1,330	LS filter strip	simplified	879
DMA-23	16,355	15,565	790	PTF	WQFR	-
DMA-24	12,305	11,600	705	PTF	WQFR	-
DMA-25	18,340	17,070	1,270	PTF	WQFR	-
Remainder*	44,275	22,440	21,835	-	-	-
TOTAL	**226,245**	**189,600**	**36,645**	-	-	-

Abbreviations & Notes:

SSP/FTP= Street-Side Planter/ Flow-Through Planter
PTF= Proprietary Treatment Facility
WQFR= Water Quality Flow Rate, per City of Beaverton Engineering Design Manual
Simplified= Simplified Sizing, per City of Beaverton Engineering Design Manual

*remainder constists of sidewalk and landscaping to be detailed during construction documents as minor site revisions are expected

GRAPHIC SCALE: 1" = 40'

NOT FOR CONSTRUCTION

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ELMONICA MIXED USE

SW 170TH AND W BASELINE

REMBOLD PROPERTIES

REVISION	DATE	REASON FOR ISSUE
A	2/27/23	LAND USE REVISION

DRAINAGE MANAGEMENT AREAS MAP

LAND USE SET

DATE 2.27.2023

PROJECT NUMBER 212225

SHEET NUMBER

ATTACHMENT B

ATTACHMENT C

Hydrograph Summary Report

Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2021

Hyd. No.	Hydrograph type (origin)	Peak flow (cfs)	Time interval (min)	Time to Peak (min)	Hyd. volume (cuft)	Inflow hyd(s)	Maximum elevation (ft)	Total strge used (cuft)	Hydrograph Description
1	SCS Runoff	0.538	2	484	12,907	------	------	------	Pre Developed
2	SCS Runoff	2.174	2	474	30,538	------	------	------	Post Developed
3	Reservoir	0.265	2	1276	30,519	2	196.21	15,052	UG Outfall

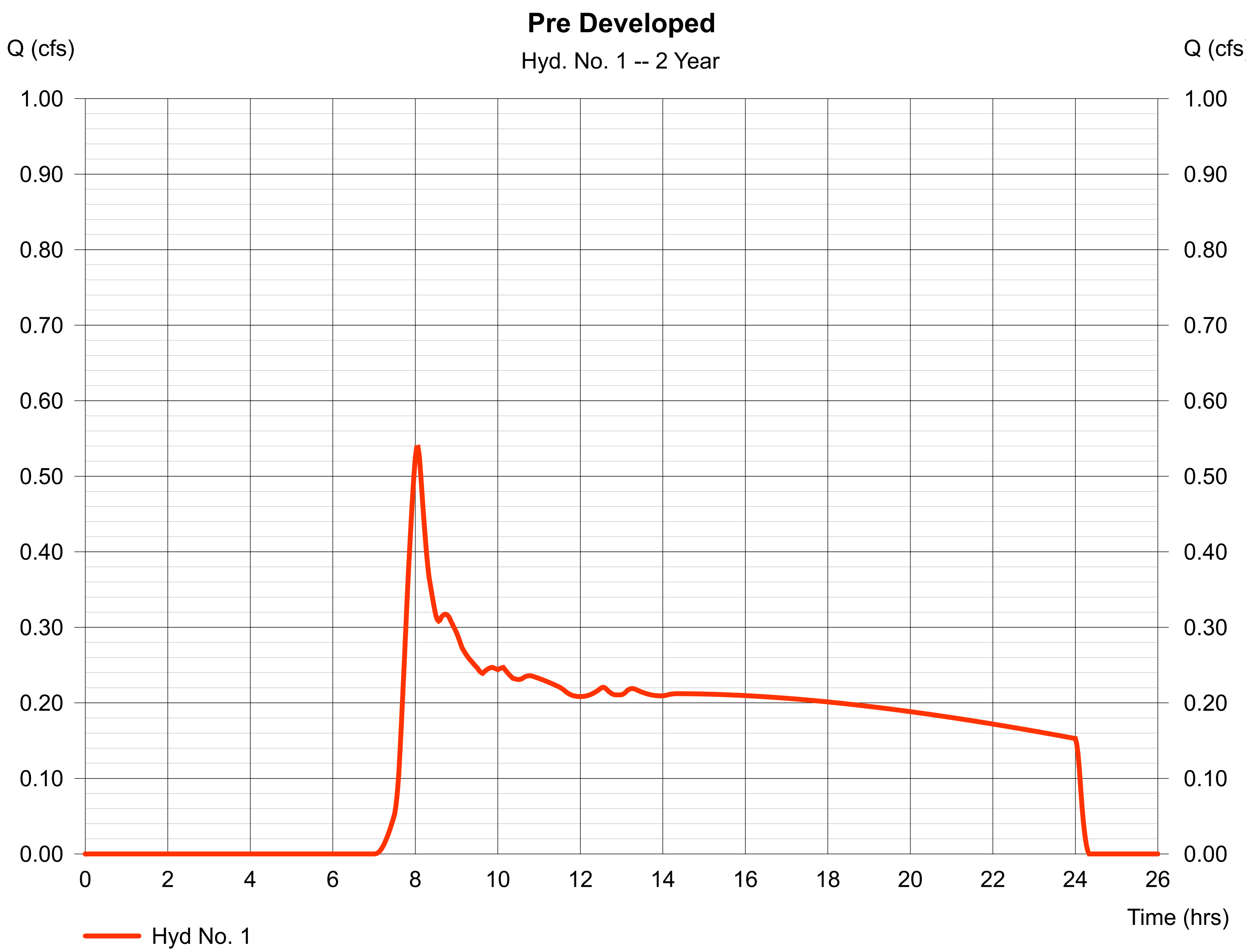
Hydrograph Report

Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2021 Thursday, 07 / 14 / 2022

Hyd. No. 1

Pre Developed

Hydrograph type	= SCS Runoff	Peak discharge	= 0.538 cfs
Storm frequency	= 2 yrs	Time to peak	= 8.07 hrs
Time interval	= 2 min	Hyd. volume	= 12,907 cuft
Drainage area	= 5.300 ac	Curve number	= 75
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= TR55	Time of conc. (Tc)	= 12.30 min
Total precip.	= 2.50 in	Distribution	= Type IA
Storm duration	= 24 hrs	Shape factor	= 484



TR55 Tc Worksheet

Hyd. No. 1

Pre Developed

Description		A		B		C		Totals
Sheet Flow								
Manning's n-value	=	0.150		0.011		0.011		
Flow length (ft)	=	100.0		0.0		0.0		
Two-year 24-hr precip. (in)	=	2.50		0.00		0.00		
Land slope (%)	=	2.20		0.00		0.00		
Travel Time (min)	**=**	**10.67**	**+**	**0.00**	**+**	**0.00**	**=**	**10.67**
Shallow Concentrated Flow								
Flow length (ft)	=	300.00		0.00		0.00		
Watercourse slope (%)	=	2.40		0.00		0.00		
Surface description	=	Paved		Paved		Paved		
Average velocity (ft/s)	=	3.15		0.00		0.00		
Travel Time (min)	**=**	**1.59**	**+**	**0.00**	**+**	**0.00**	**=**	**1.59**
Channel Flow								
X sectional flow area (sqft)	=	0.00		0.00		0.00		
Wetted perimeter (ft)	=	0.00		0.00		0.00		
Channel slope (%)	=	0.00		0.00		0.00		
Manning's n-value	=	0.015		0.015		0.015		
Velocity (ft/s)	=	0.00		0.00		0.00		
Flow length (ft)	({0})	0.0		0.0		0.0		
Travel Time (min)	**=**	**0.00**	**+**	**0.00**	**+**	**0.00**	**=**	**0.00**

Total Travel Time, Tc .... **12.30 min**

Hydrograph Report

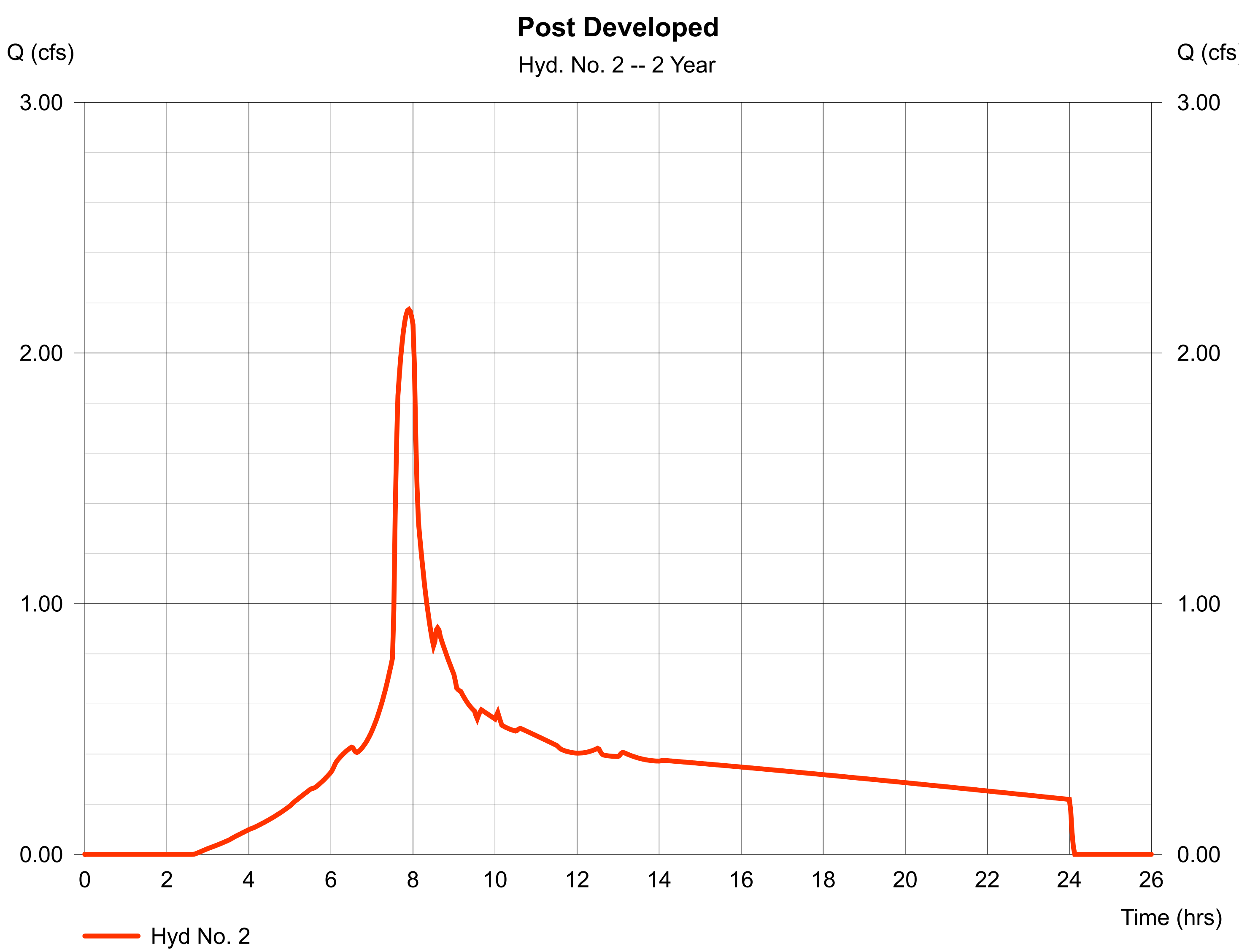
Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2021 Thursday, 07 / 14 / 2022

Hyd. No. 2

Post Developed

Hydrograph type	= SCS Runoff	Peak discharge	= 2.174 cfs
Storm frequency	= 2 yrs	Time to peak	= 7.90 hrs
Time interval	= 2 min	Hyd. volume	= 30,538 cuft
Drainage area	= 5.300 ac	Curve number	= 92*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 5.00 min
Total precip.	= 2.50 in	Distribution	= Type IA
Storm duration	= 24 hrs	Shape factor	= 484

* Composite (Area/CN) = [(3.900 x 98) + (1.400 x 75)] / 5.300



Hydrograph Report

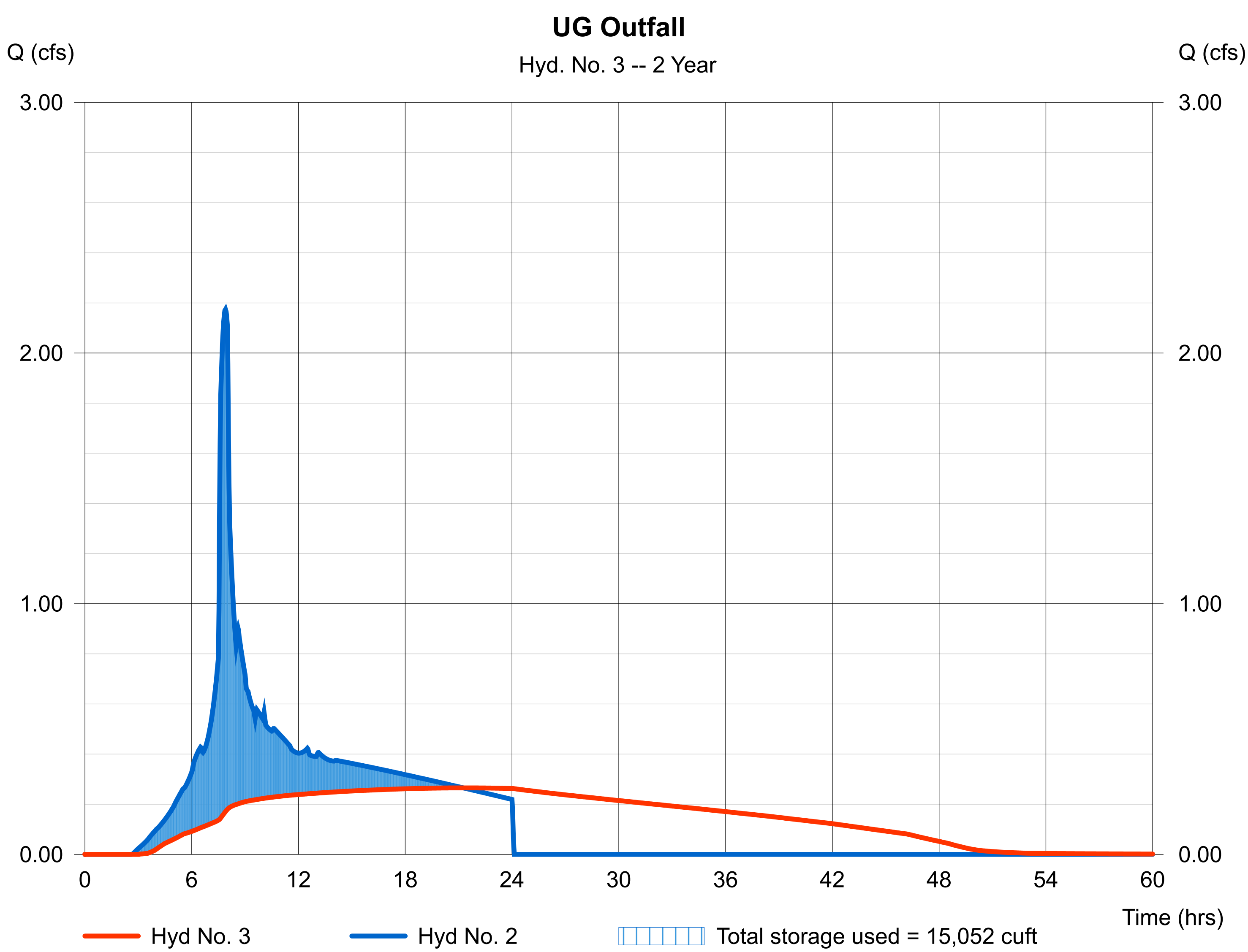
Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2021 Thursday, 07 / 14 / 2022

Hyd. No. 3

UG Outfall

Hydrograph type	= Reservoir	Peak discharge	= 0.265 cfs
Storm frequency	= 2 yrs	Time to peak	= 21.27 hrs
Time interval	= 2 min	Hyd. volume	= 30,519 cuft
Inflow hyd. No.	= 2 - Post Developed	Max. Elevation	= 196.21 ft
Reservoir name	= UG	Max. Storage	= 15,052 cuft

Storage Indication method used.



Pond Report

Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2021 Thursday, 07 / 14 / 2022

Pond No. 1 - UG

Pond Data

UG Chambers -Invert elev. = 192.00 ft, Rise x Span = 5.00 x 5.00 ft, Barrel Len = 200.00 ft, No. Barrels = 4, Slope = 0.00%, Headers = Yes

Stage / Storage Table

Stage (ft)	Elevation (ft)	Contour area (sqft)	Incr. Storage (cuft)	Total storage (cuft)
0.00	192.00	n/a	0	0
0.50	192.50	n/a	874	874
1.00	193.00	n/a	1,518	2,392
1.50	193.50	n/a	1,846	4,239
2.00	194.00	n/a	2,034	6,273
2.50	194.50	n/a	2,125	8,397
3.00	195.00	n/a	2,125	10,522
3.50	195.50	n/a	2,033	12,556
4.00	196.00	n/a	1,846	14,401
4.50	196.50	n/a	1,517	15,918
5.00	197.00	n/a	873	16,791

Culvert / Orifice Structures

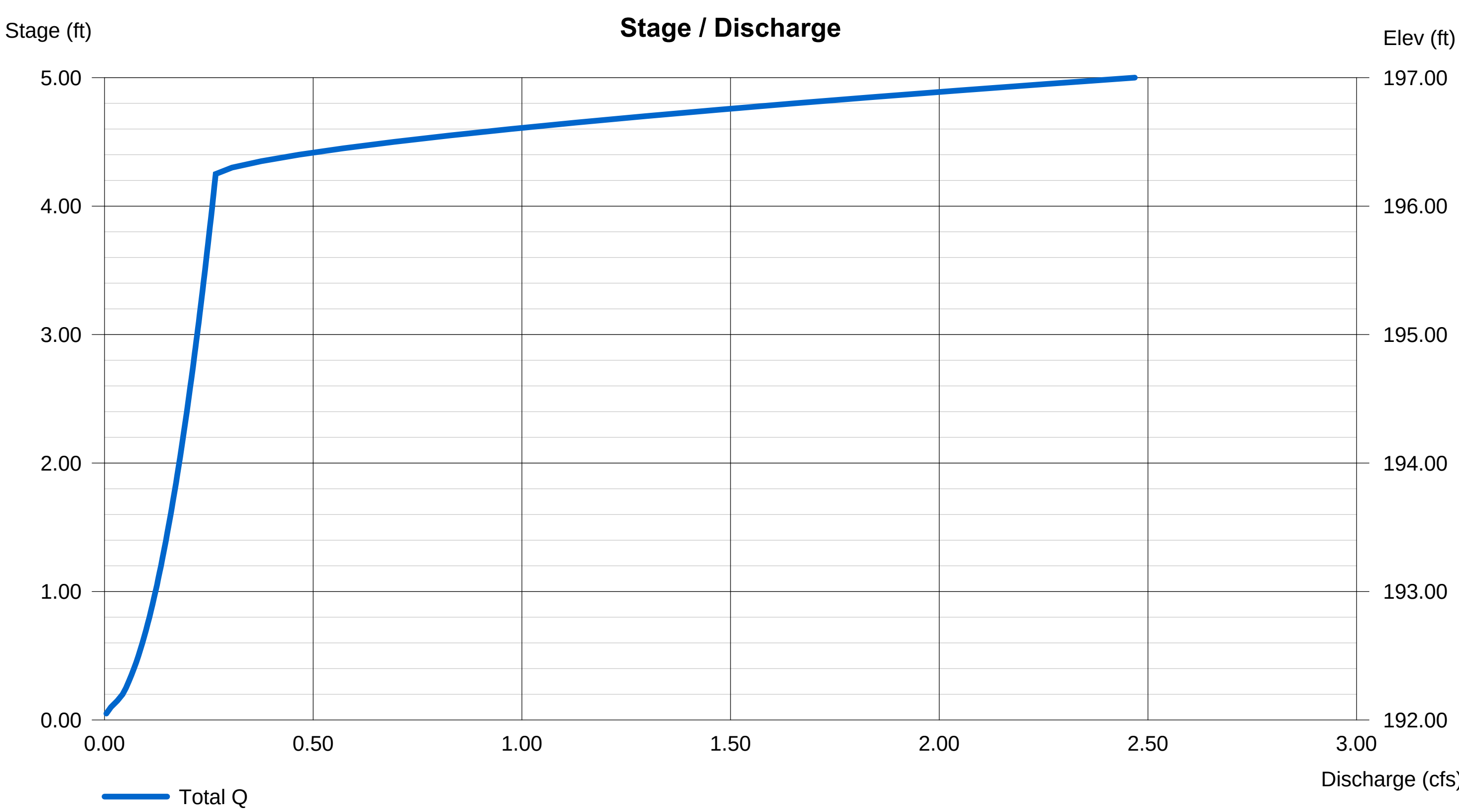
	[A]	[B]	[C]	[PrfRsr]
Rise (in)	= 15.00	2.25	9.00	0.00
Span (in)	= 15.00	2.25	12.00	0.00
No. Barrels	= 1	1	1	0
Invert El. (ft)	= 192.00	192.00	196.25	0.00
Length (ft)	= 10.00	0.00	0.00	0.00
Slope (%)	= 1.00	0.00	0.00	n/a
N-Value	= .013	.013	.013	n/a
Orifice Coeff.	= 0.60	0.60	0.60	0.60
Multi-Stage	= n/a	Yes	Yes	No

Weir Structures

	[A]	[B]	[C]	[D]
Crest Len (ft)	= 10.00	0.00	0.00	0.00
Crest El. (ft)	= 197.00	0.00	0.00	0.00
Weir Coeff.	= 3.33	3.33	3.33	3.33
Weir Type	= 1	---	---	---
Multi-Stage	= Yes	No	No	No

Exfil.(in/hr) = 0.000 (by Contour)
TW Elev. (ft) = 0.00

Note: Culvert/Orifice outflows are analyzed under inlet (ic) and outlet (oc) control. Weir risers checked for orifice conditions (ic) and submergence (s).



Hydrograph Summary Report

Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2021

Hyd. No.	Hydrograph type (origin)	Peak flow (cfs)	Time interval (min)	Time to Peak (min)	Hyd. volume (cuft)	Inflow hyd(s)	Maximum elevation (ft)	Total strge used (cuft)	Hydrograph Description
1	SCS Runoff	1.387	2	482	25,128	------	------	------	Pre Developed
2	SCS Runoff	3.374	2	472	46,695	------	------	------	Post Developed
3	Reservoir	0.865	2	566	46,675	2	196.56	16,028	UG Outfall

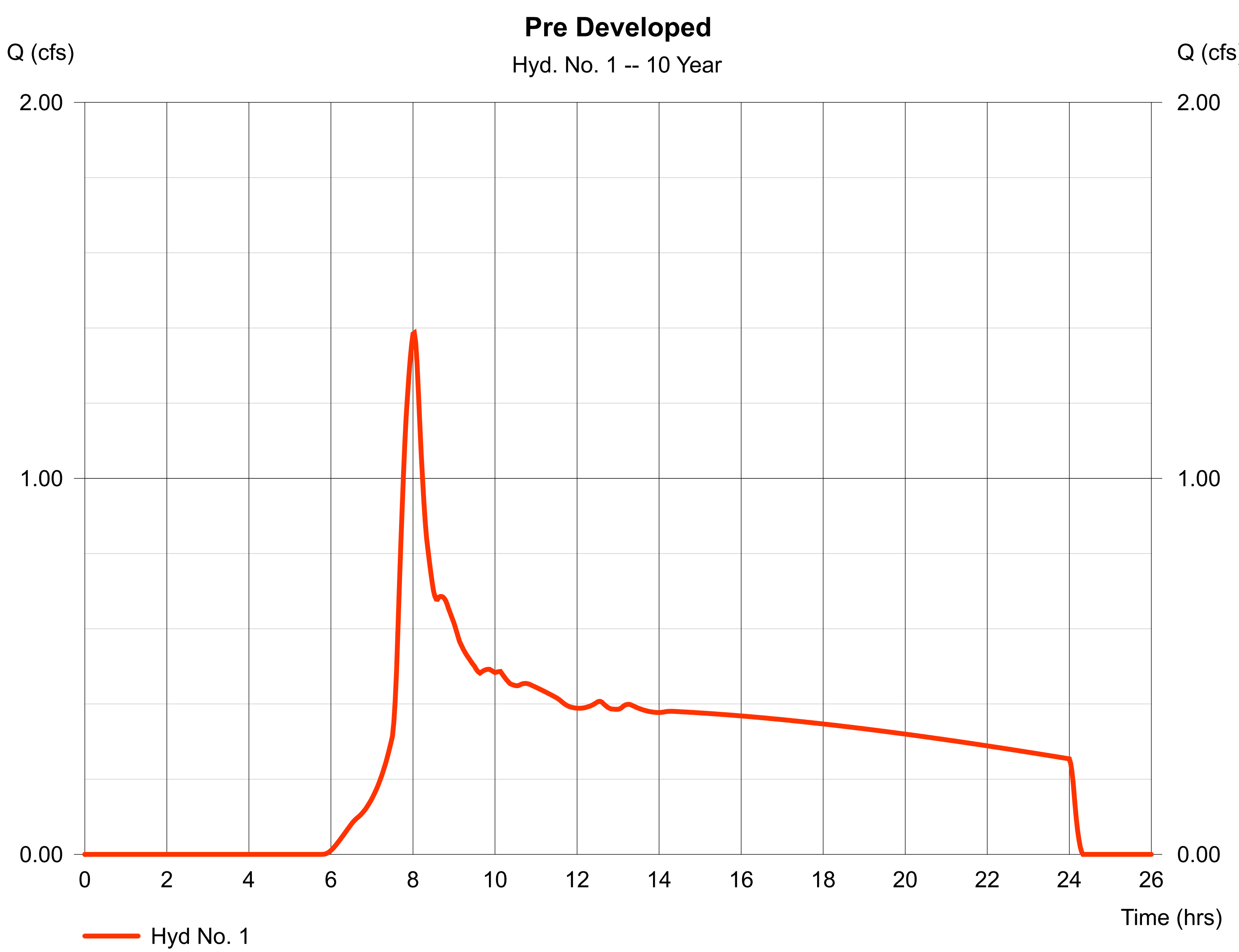
Hydrograph Report

Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2021 Thursday, 07 / 14 / 2022

Hyd. No. 1

Pre Developed

Hydrograph type	= SCS Runoff	Peak discharge	= 1.387 cfs
Storm frequency	= 10 yrs	Time to peak	= 8.03 hrs
Time interval	= 2 min	Hyd. volume	= 25,128 cuft
Drainage area	= 5.300 ac	Curve number	= 75
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= TR55	Time of conc. (Tc)	= 12.30 min
Total precip.	= 3.45 in	Distribution	= Type IA
Storm duration	= 24 hrs	Shape factor	= 484



Hydrograph Report

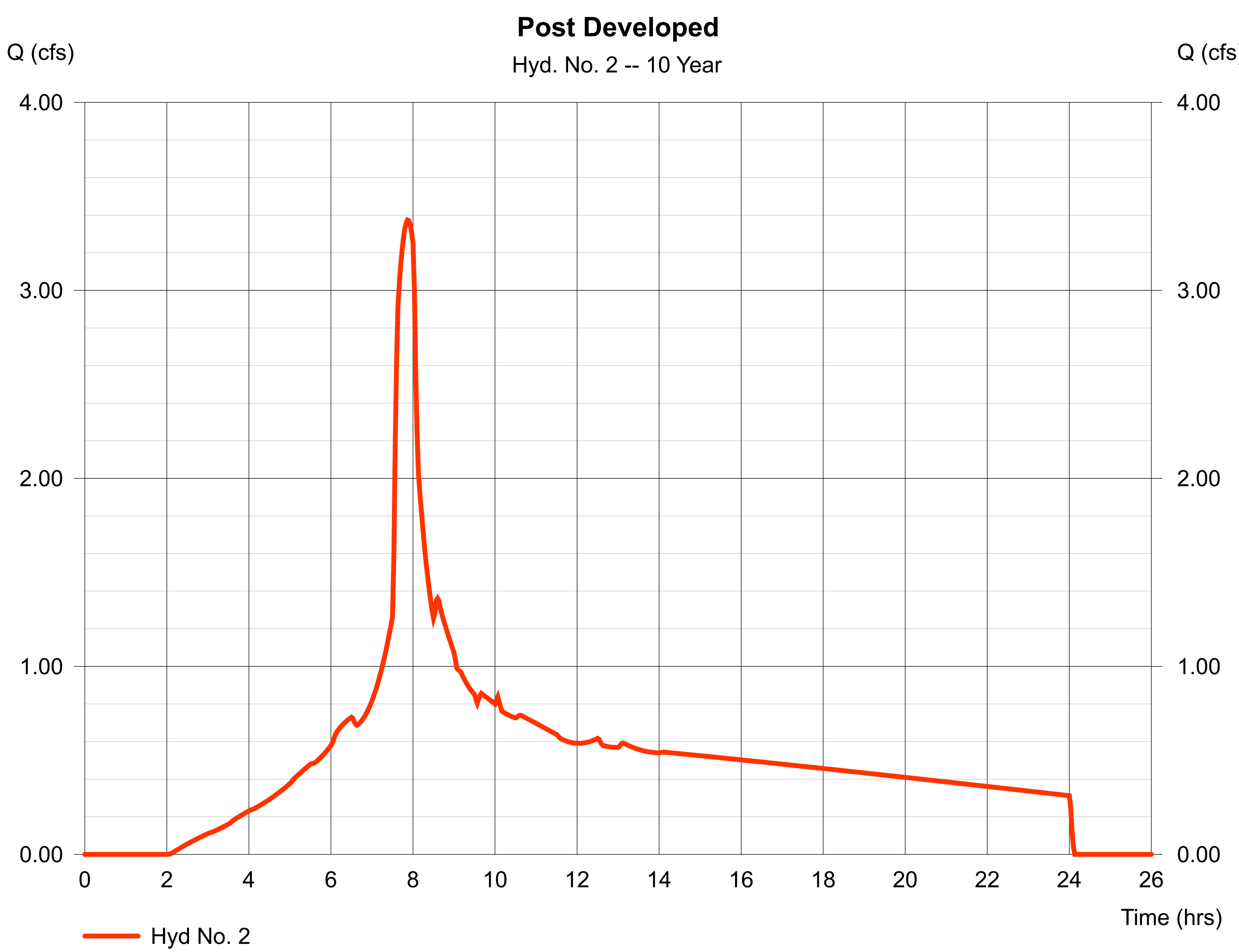
Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2021 Thursday, 07 / 14 / 2022

Hyd. No. 2

Post Developed

Hydrograph type	= SCS Runoff	Peak discharge	= 3.374 cfs
Storm frequency	= 10 yrs	Time to peak	= 7.87 hrs
Time interval	= 2 min	Hyd. volume	= 46,695 cuft
Drainage area	= 5.300 ac	Curve number	= 92*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 5.00 min
Total precip.	= 3.45 in	Distribution	= Type IA
Storm duration	= 24 hrs	Shape factor	= 484

* Composite (Area/CN) = [(3.900 x 98) + (1.400 x 75)] / 5.300



Hydrograph Report

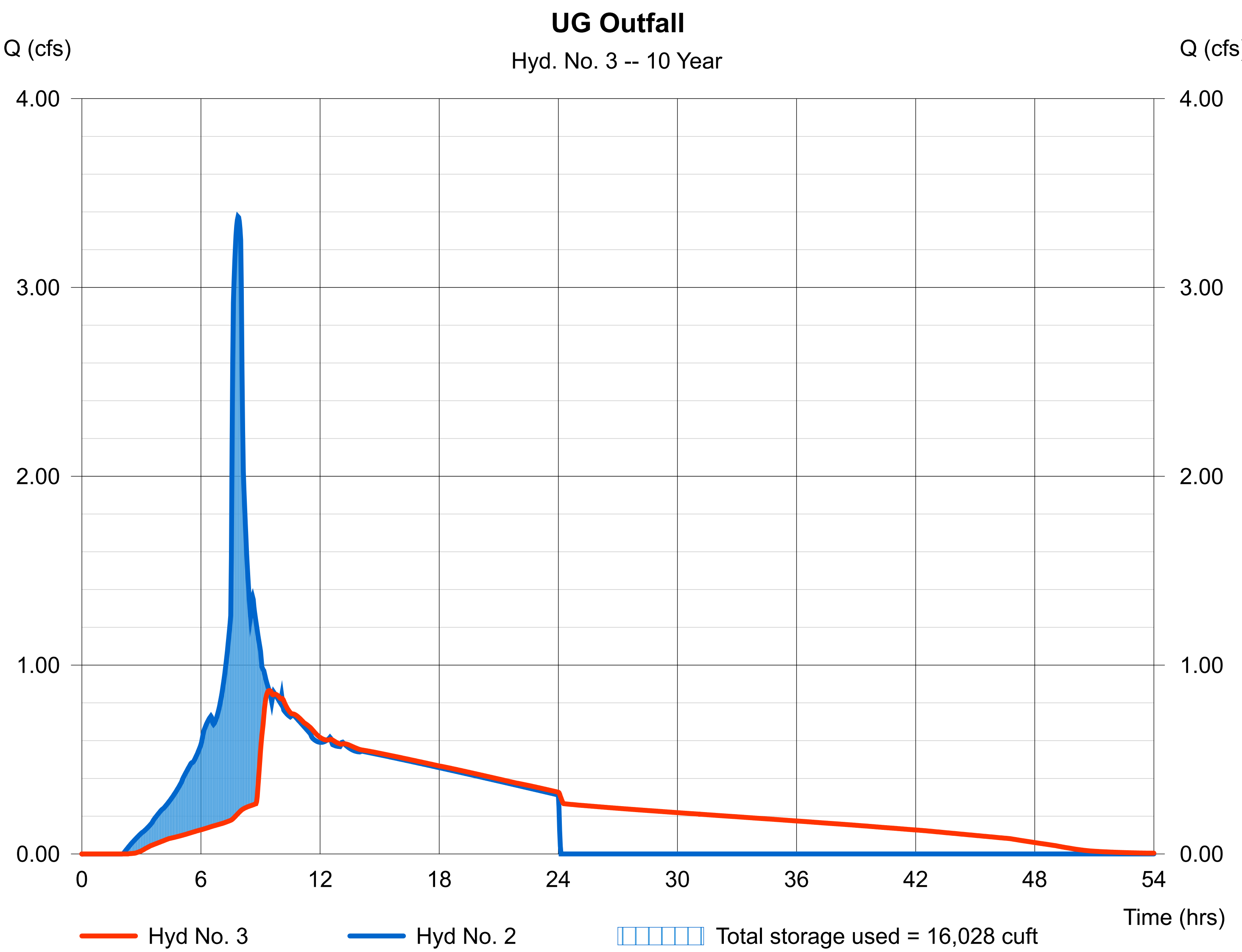
Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2021 Thursday, 07 / 14 / 2022

Hyd. No. 3

UG Outfall

Hydrograph type	= Reservoir	Peak discharge	= 0.865 cfs
Storm frequency	= 10 yrs	Time to peak	= 9.43 hrs
Time interval	= 2 min	Hyd. volume	= 46,675 cuft
Inflow hyd. No.	= 2 - Post Developed	Max. Elevation	= 196.56 ft
Reservoir name	= UG	Max. Storage	= 16,028 cuft

Storage Indication method used.



Hydrograph Summary Report

Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2021

Hyd. No.	Hydrograph type (origin)	Peak flow (cfs)	Time interval (min)	Time to Peak (min)	Hyd. volume (cuft)	Inflow hyd(s)	Maximum elevation (ft)	Total strge used (cuft)	Hydrograph Description
1	SCS Runoff	1.851	2	480	31,587	------	------	------	Pre Developed
2	SCS Runoff	3.947	2	472	54,489	------	------	------	Post Developed
3	Reservoir	1.498	2	522	54,470	2	196.76	16,368	UG Outfall

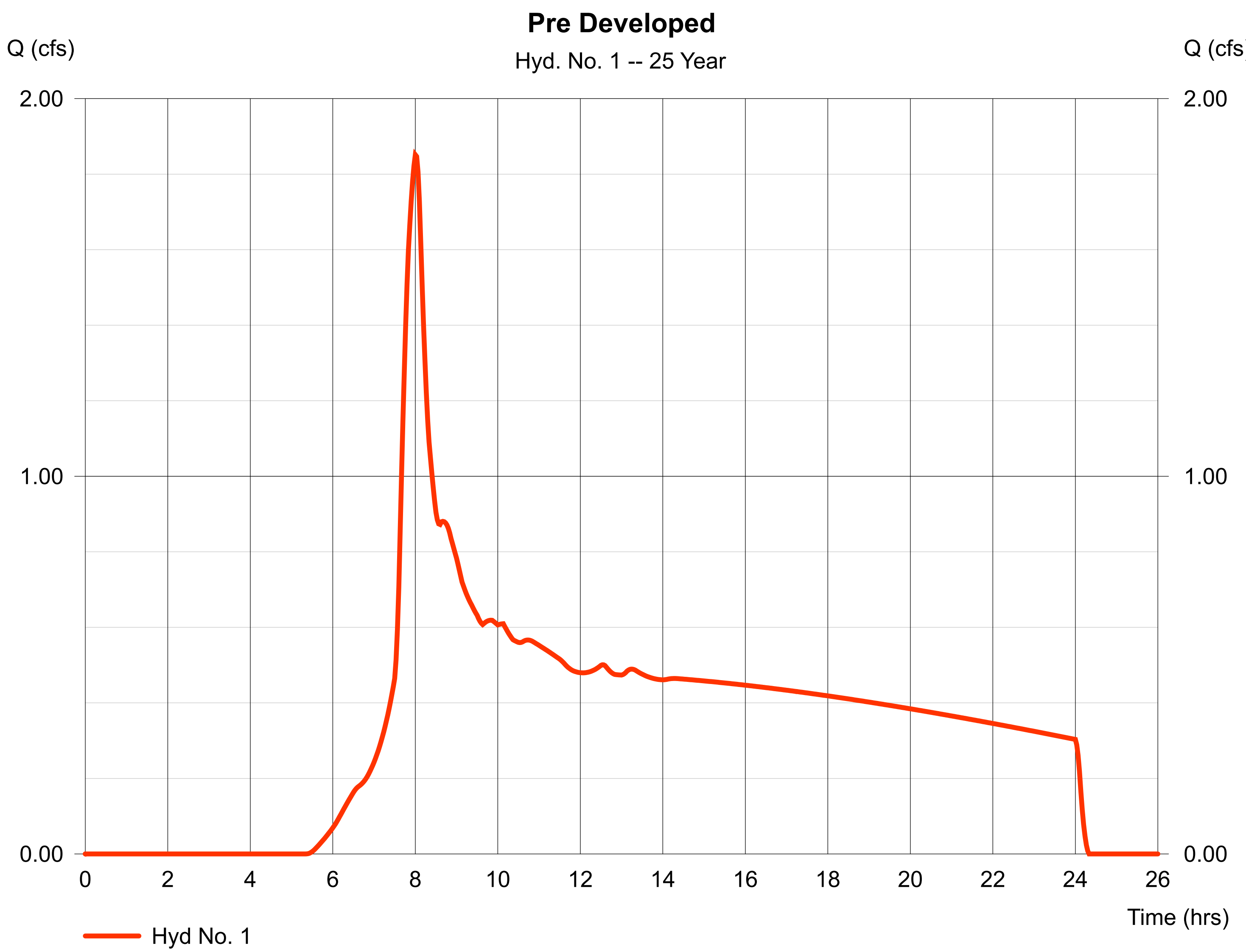
Hydrograph Report

Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2021 Thursday, 07 / 14 / 2022

Hyd. No. 1

Pre Developed

Hydrograph type	= SCS Runoff	Peak discharge	= 1.851 cfs
Storm frequency	= 25 yrs	Time to peak	= 8.00 hrs
Time interval	= 2 min	Hyd. volume	= 31,587 cuft
Drainage area	= 5.300 ac	Curve number	= 75
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= TR55	Time of conc. (Tc)	= 12.30 min
Total precip.	= 3.90 in	Distribution	= Type IA
Storm duration	= 24 hrs	Shape factor	= 484



Hydrograph Report

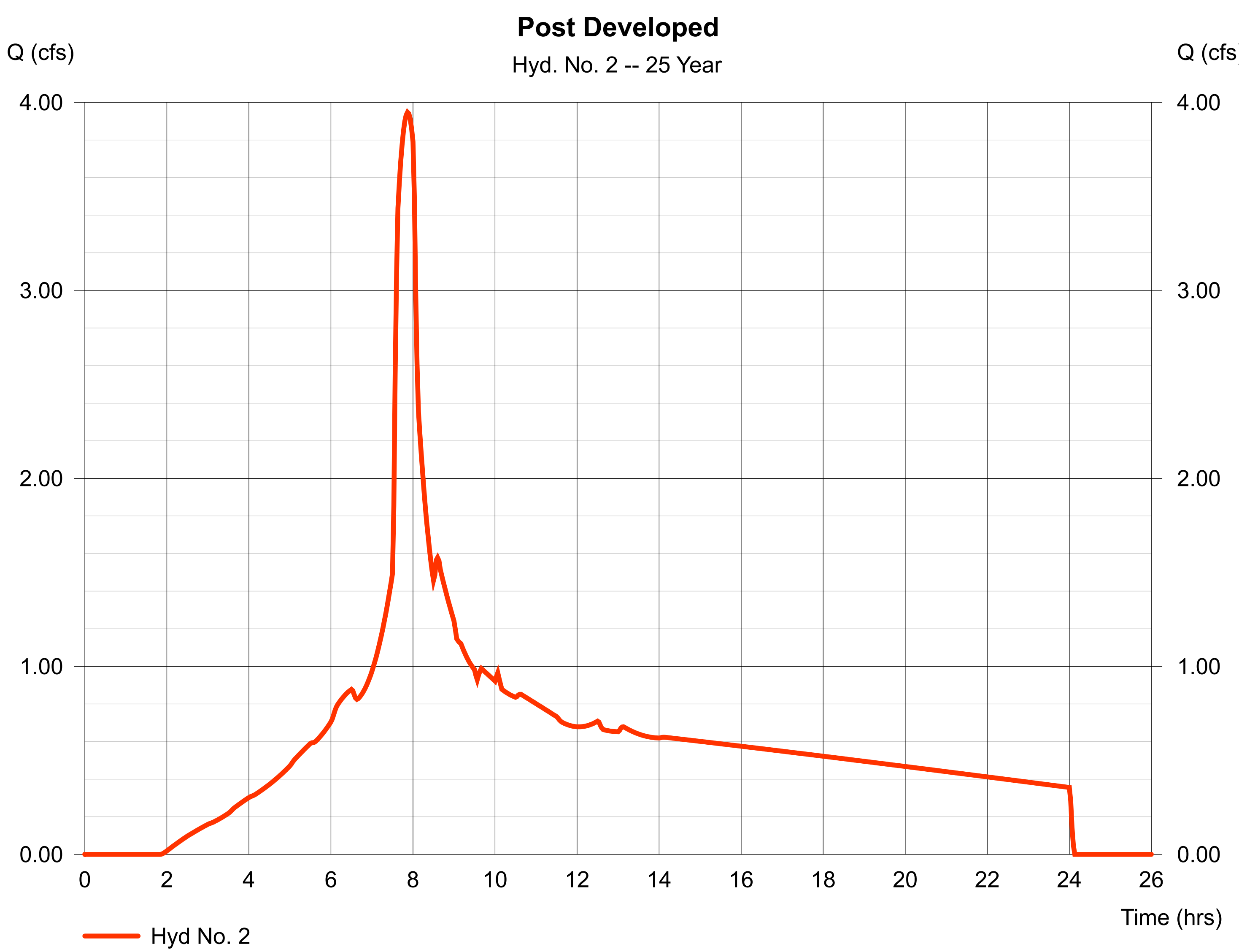
Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2021 Thursday, 07 / 14 / 2022

Hyd. No. 2

Post Developed

Hydrograph type	= SCS Runoff	Peak discharge	= 3.947 cfs
Storm frequency	= 25 yrs	Time to peak	= 7.87 hrs
Time interval	= 2 min	Hyd. volume	= 54,489 cuft
Drainage area	= 5.300 ac	Curve number	= 92*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 5.00 min
Total precip.	= 3.90 in	Distribution	= Type IA
Storm duration	= 24 hrs	Shape factor	= 484

* Composite (Area/CN) = [(3.900 x 98) + (1.400 x 75)] / 5.300



Hydrograph Report

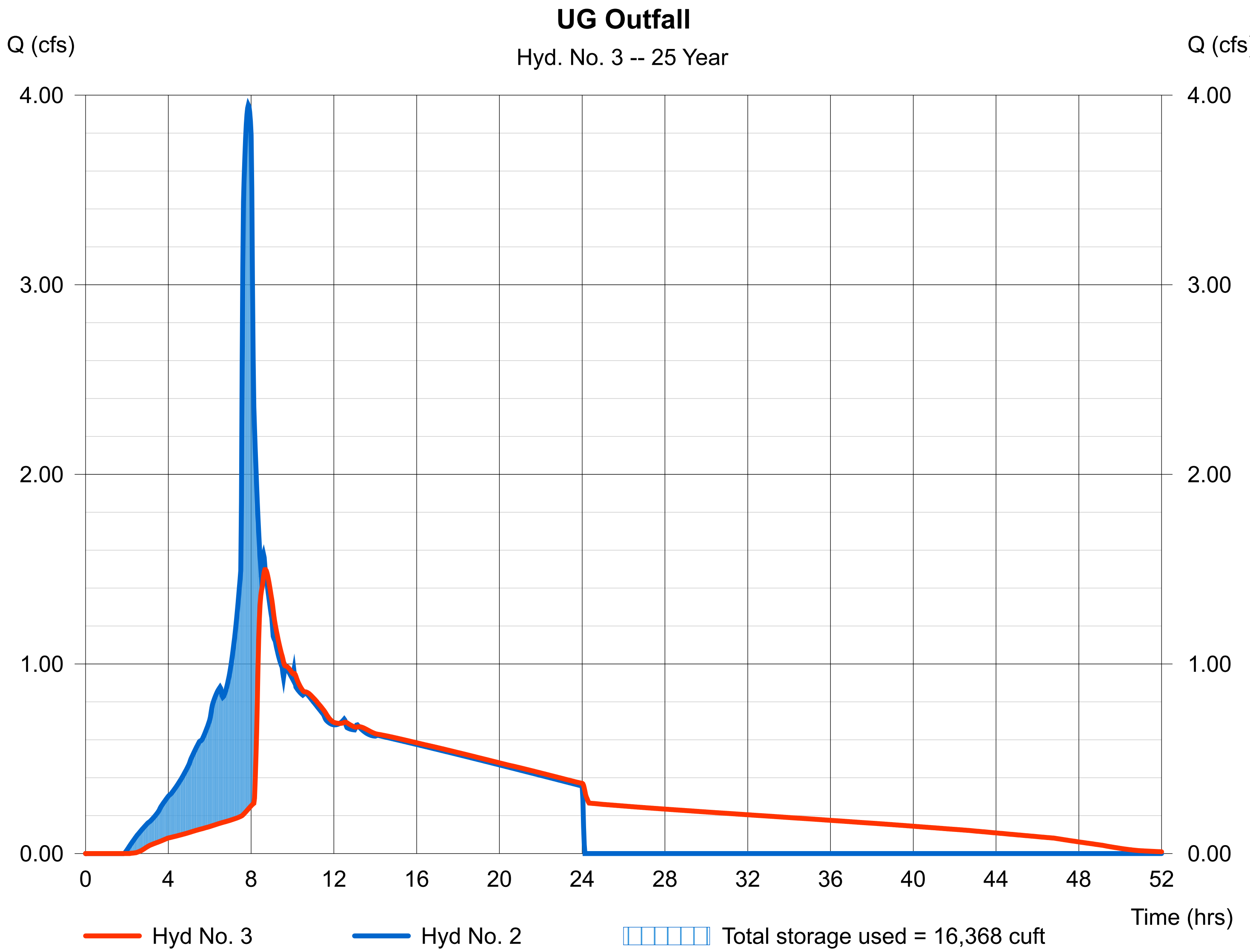
Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2021 Thursday, 07 / 14 / 2022

Hyd. No. 3

UG Outfall

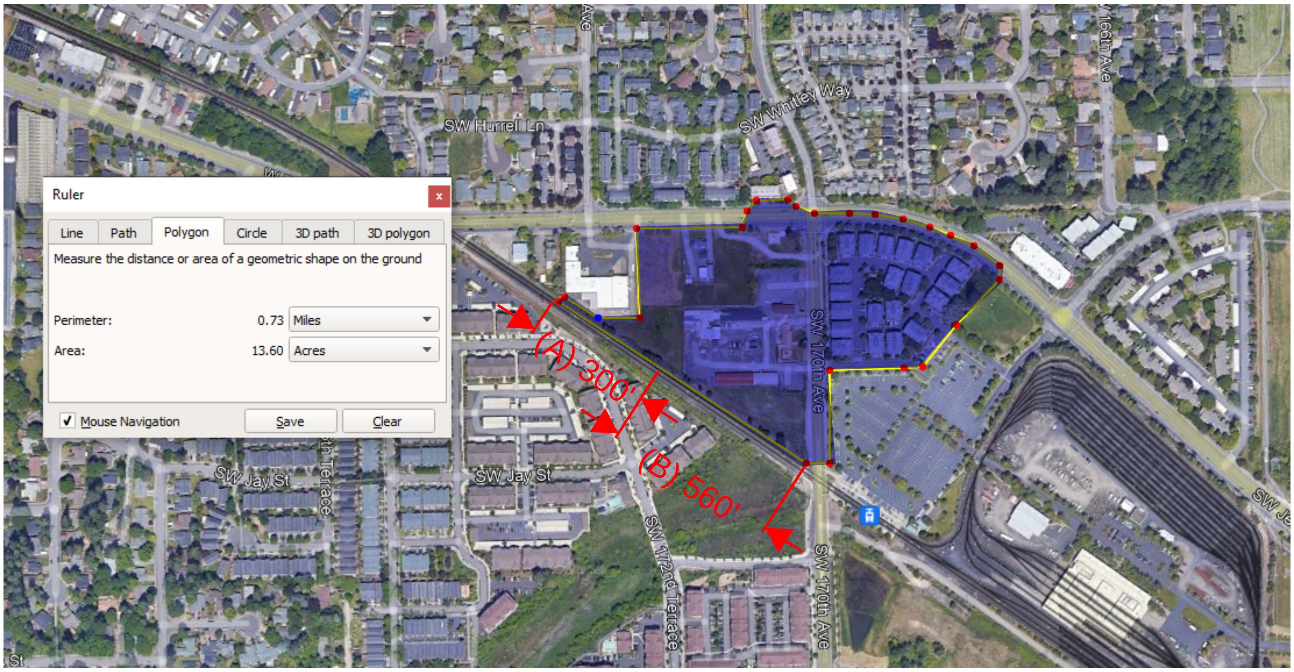
Hydrograph type	= Reservoir	Peak discharge	= 1.498 cfs
Storm frequency	= 25 yrs	Time to peak	= 8.70 hrs
Time interval	= 2 min	Hyd. volume	= 54,470 cuft
Inflow hyd. No.	= 2 - Post Developed	Max. Elevation	= 196.76 ft
Reservoir name	= UG	Max. Storage	= 16,368 cuft

Storage Indication method used.



ATTACHMENT D

Attachment D
Existing 24" Culvert Drainage Area & Time of Concentration



*Time of concentration calculated using methods from Appendix F of the ODOT Hydraulics Manual.

(A) Overland Sheet Flow

$$T_{osf} = \frac{0.93(L^{0.6}n^{0.6})}{(i^{0.4}S^{0.3})}$$

L	300	ft
n	0.15	
i	1.13	(Based on ODOT, Zone 7 , 25yr 30min Tc)
S	0.016	ft/ft
T_{osf}	**30**	min

(B) Shallow Concentrated Flow

$$T_{scf} = \frac{L}{60V}$$

L	560	ft	
V	2	ft/sec	Grassed Waterway (1.6%)
T_{scf}	**5**	min	

Attachment D
Existing 24" Culvert Total Landscape Drainage Area

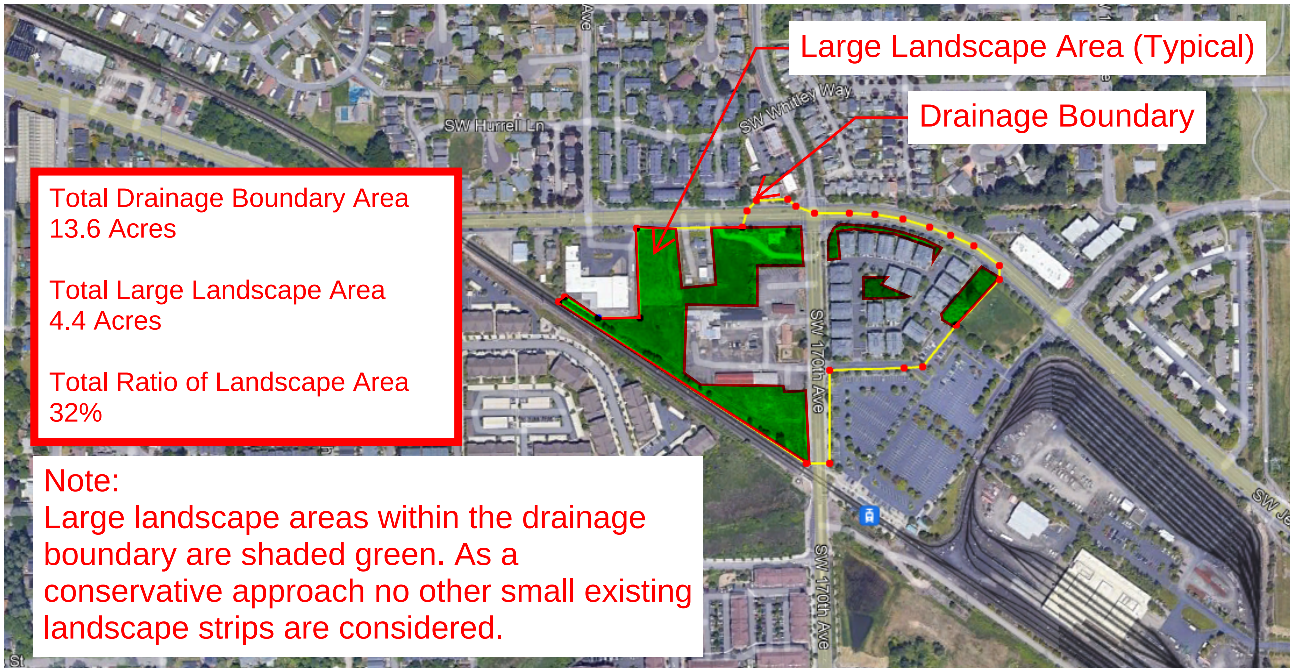


Table 1 Runoff Coefficients for the Rational Method

	FLAT	ROLLING	HILLY
Pavement & Roofs	**0.90**	**0.90**	**0.90**
Earth Shoulders	0.50	0.50	0.50
Drives & Walks	0.75	0.80	**0.85**
Gravel Pavement	**0.85**	**0.85**	**0.85**
City Business Areas	0.80	**0.85**	**0.85**
Apartment Dwelling Areas	0.50	0.60	0.70
Light Residential: 1 to 3 units/acre	0.35	0.40	0.45
Normal Residential: 3 to 6 units/acre	0.50	0.55	0.60
Dense Residential: 6 to 15 units/acre	0.70	0.75	0.80
Lawns	0.17	0.22	0.35
Grass Shoulders	0.25	0.25	0.25
Side Slopes, Earth	0.60	0.60	0.60
Side Slopes, Turf	0.30	0.30	0.30
Median Areas, Turf	0.25	0.30	0.30
Cultivated Land, Clay & Loam	0.50	0.55	0.60
Cultivated Land, Sand & Gravel	0.25	0.30	0.35
Industrial Areas, Light	0.50	0.70	0.80
Industrial Areas, Heavy	0.60	0.80	**0.90**
Parks & Cemeteries	0.10	0.15	0.25
Playgrounds	0.20	0.25	0.30
Woodland & Forests	0.10	0.15	0.20
Meadows & Pasture Land	0.25	0.30	0.35
Unimproved Areas	0.10	0.20	0.30

Note:

- ***Impervious surfaces in bold***
- *Rolling = ground slope between 2 percent to 10 percent*
- *Hilly = ground slope greater than 10 percent*

Table 2 Runoff Coefficient Adjustment Factors

RECURRENCE INTERVAL	RUNOFF COEFFICIENT ADJUSTMENT FACTOR
10 years or less	1.0
25 years	1.1
50 years	1.2
100 years	1.25

- **Time of Concentration "T_c" -** The time of concentration (T_c), is defined as the time it takes for runoff to travel from the hydraulically most distant point in the watershed to the point of reference downstream. Most drainage paths consist of overland flow segments as well as channel flow segments. The overland flow component can be further divided into a sheet flow segment and a shallow concentrated flow segment. Urban drainage basins often will have one or more pipe flow segments. The travel time is computed for each flow segment and the time of concentration is equal to the sum of the individual travel times, as follows:

  $$T_c = T_{osf} + T_{scf} + T_{ocf} + T_{pf} \qquad \text{(Equation 3)}$$

  Where:

  T_c = Time of concentration in minutes (min.)
  T_{osf} = Travel time for the overland sheet flow segment in minutes (min.)
  T_{scf} = Travel time for the shallow concentrated flow segment in minutes (min.)
  T_{ocf} = Travel time for the open-channel flow segment(s) in minutes (min.)
  T_{pf} = Travel time for the pipe flow segment(s) in minutes (min.)

  The drainage path used to determine the time of concentration need not include all of the listed segments. As an example, a roadway pavement bounded by curbs and drained by an inlet connected to a storm drain will have segments of overland sheet flow (pavement), open-channel flow (gutter), and pipe flow (storm drain). There is no shallow concentrated flow segment.

The travel times for the flow segments are determined as follows.

- **Overland Sheet Flow -** Overland sheet flow is shallow flow over a plane surface. It occurs in the furthest upstream segment of the drainage path, which is located immediately downstream from the drainage divide. The length of the overland sheet flow segment is the shorter of: the distance between the drainage divide and the upper end of a defined channel,

RATIONAL METHOD RAINFALL INTENSITIES

RAINFALL INTENSTITY IS FOR EAST WASHINGTON COUNTY AND IS SHOWN AS INCHES PER HOUR

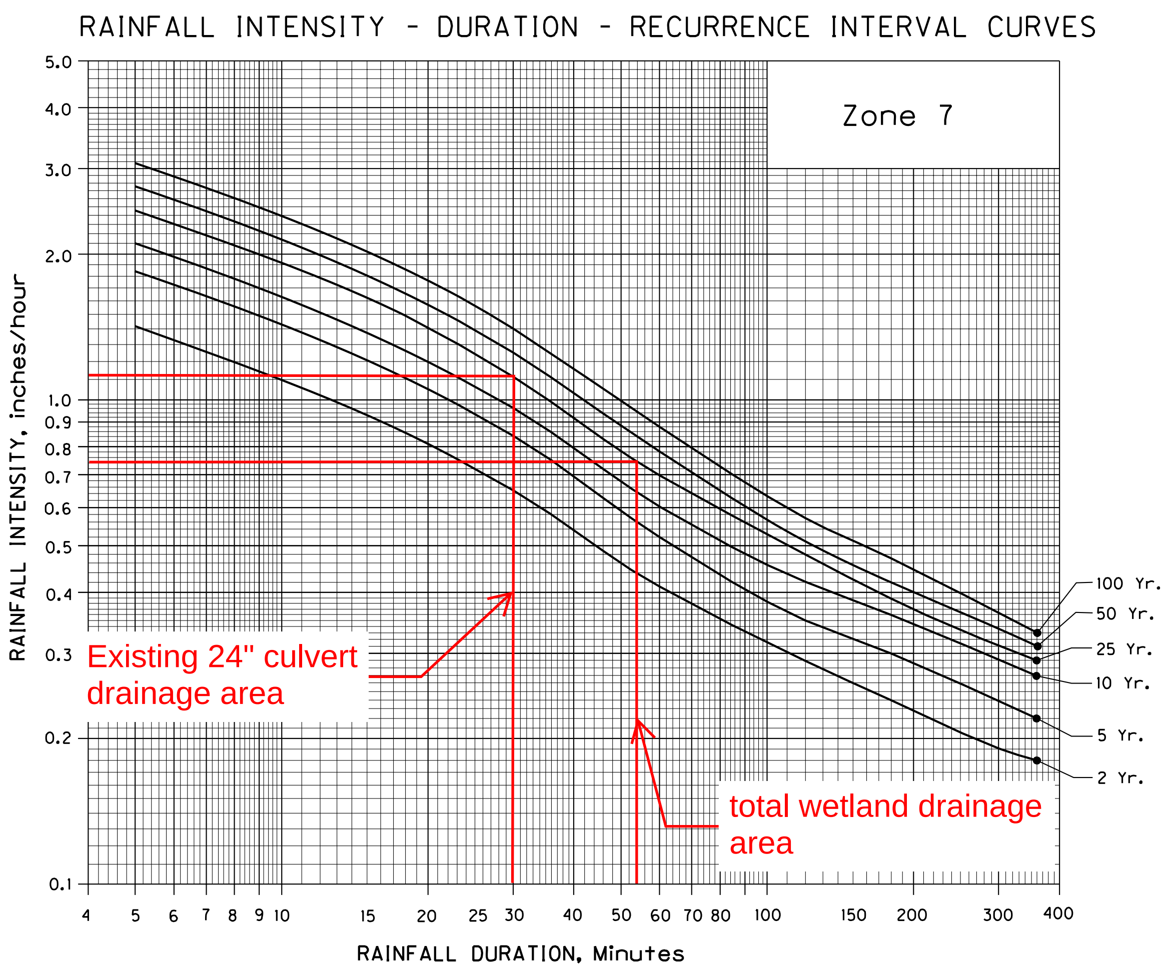
TIME OF CONCENTRATION (MINUTES)	STORM EVENT: YEAR AND PROBABILITY					
	2 50%	5 20%	10 10%	25 4%	50 2%	100 1%
0	1.90	2.50	3.00	3.40	4.00	4.50
5	1.90	2.50	3.00	3.40	4.00	4.50
10	1.30	1.70	2.20	2.50	3.00	3.50
15	1.10	1.40	1.80	2.10	2.50	2.90
20	0.90	1.20	1.50	1.80	2.10	2.40
30	0.75	0.95	1.20	1.40	1.65	1.90
40	0.60	0.75	1.00	1.15	1.30	1.60
50	0.55	0.70	0.85	1.00	1.15	1.35
70	0.45	0.55	0.70	0.82	0.95	1.10
100	0.40	0.45	0.55	0.67	0.75	0.90
180>	0.35	0.40	0.50	0.60	0.70	0.85

RATIONAL METHOD
RAINFALL INTENSITIES

DRAWING NO. 1275

REVISED 05-07

Clean Water Services


RAINFALL INTENSITY - DURATION - RECURRENCE INTERVAL CURVES
Zone 7
RAINFALL INTENSITY, inches/hour
5.0
4.0
3.0
2.0
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
4
5
6
7
8
9
10
15
20
25
30
40
50
60
70
80
100
150
200
300
400
RAINFALL DURATION, Minutes
100 Yr.
50 Yr.
25 Yr.
10 Yr.
5 Yr.
2 Yr.
Existing 24" culvert drainage area
total wetland drainage area

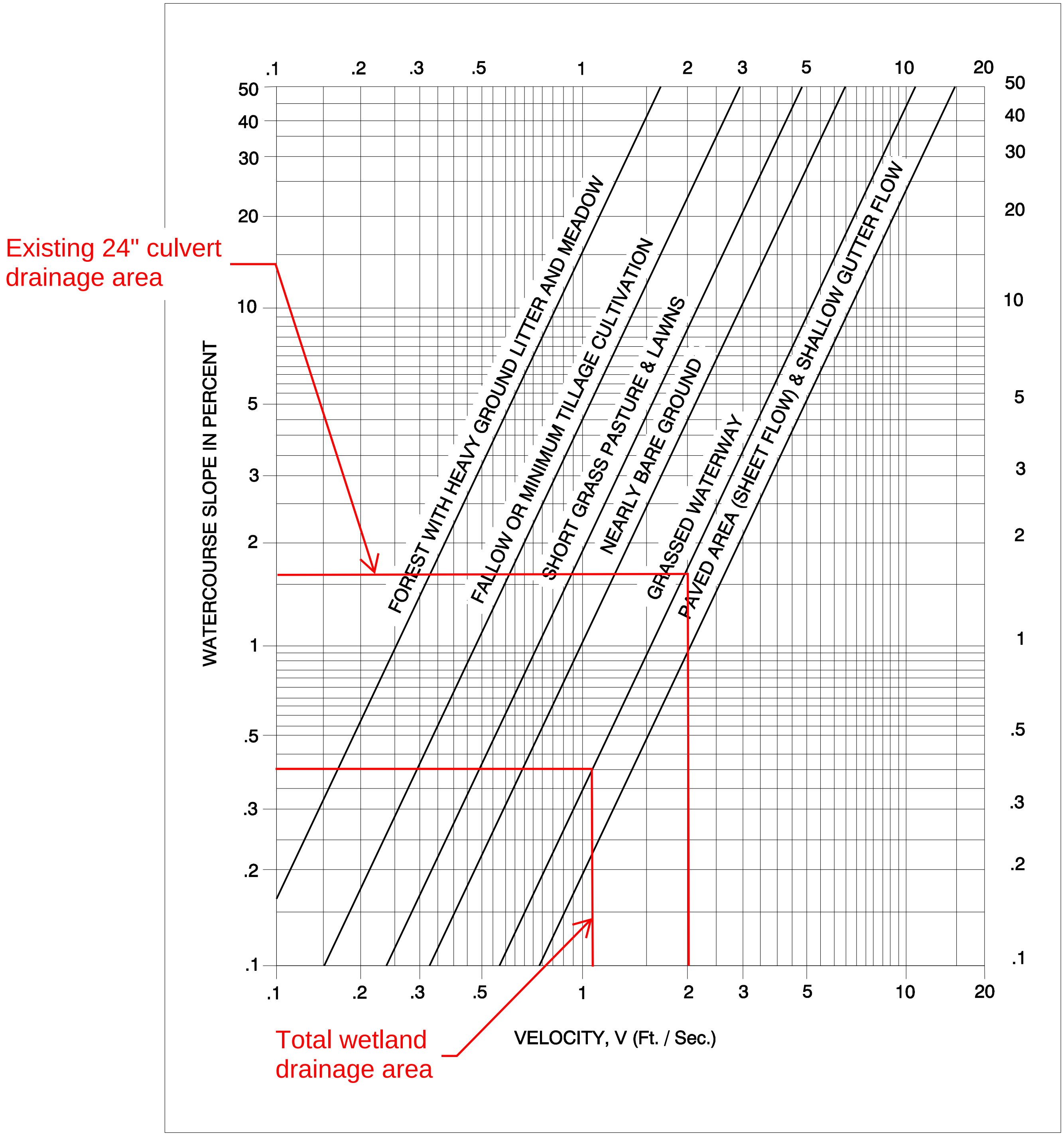
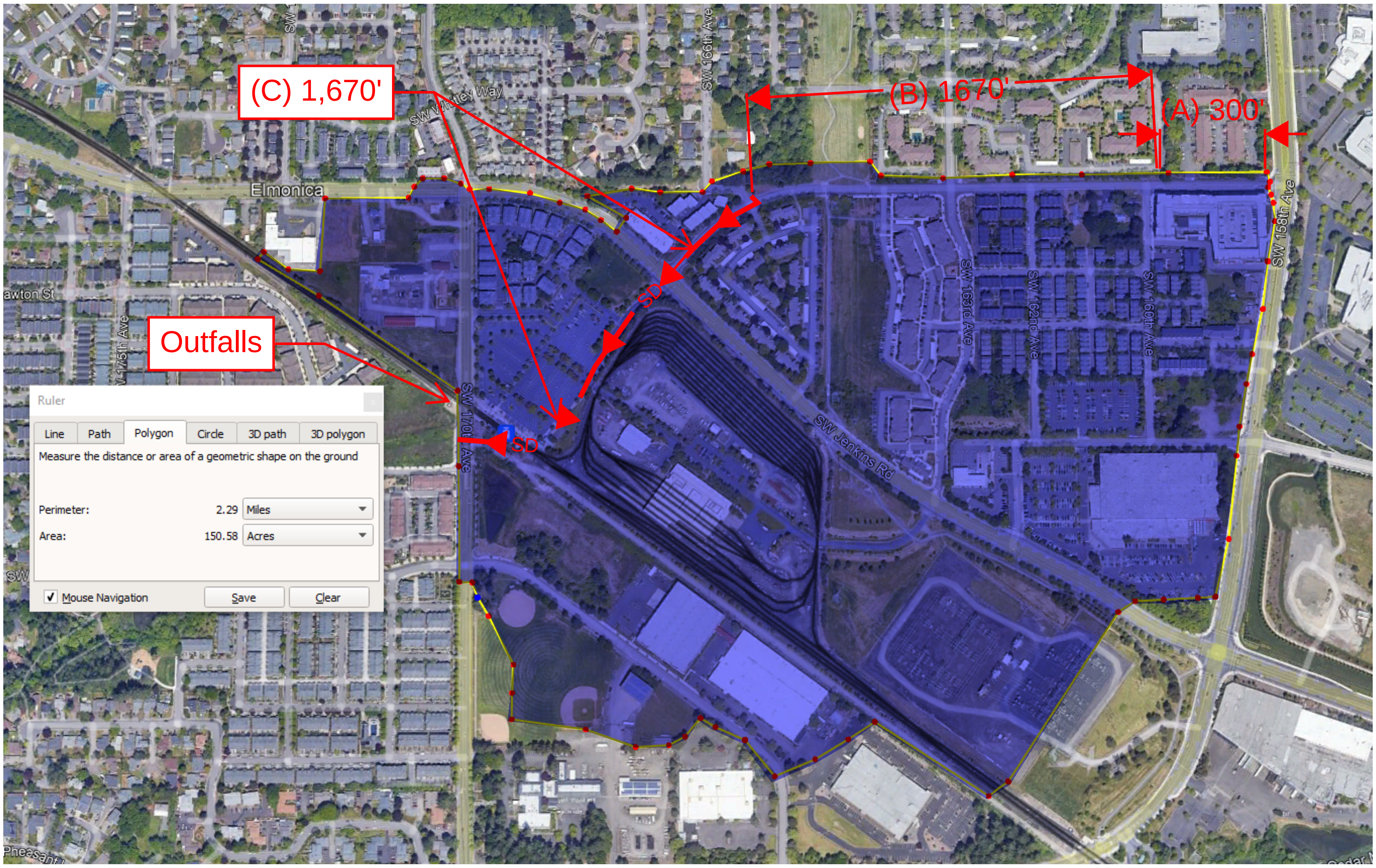


Figure 1 Shallow Concentrated Flow Velocities

ATTACHMENT E

Attachment E
Wetland Total Upstream Drainage Area & Time of Concentration



*Time of concentration calculated using methods from Appendix F of the ODOT Hydraulics Manual.

(A) Overland Sheet Flow

$$T_{osf} = \frac{0.93(L^{0.6}n^{0.6})}{(i^{0.4}S^{0.3})}$$

L	300	ft	
n	0.15		
i	0.74	(Based on ODOT, Zone 7 , 25yr 54min Tc)	
S	0.004	ft/ft	
T_{osf}	**54**	min	

(B) Shallow Concentrated Flow (in grass swale)

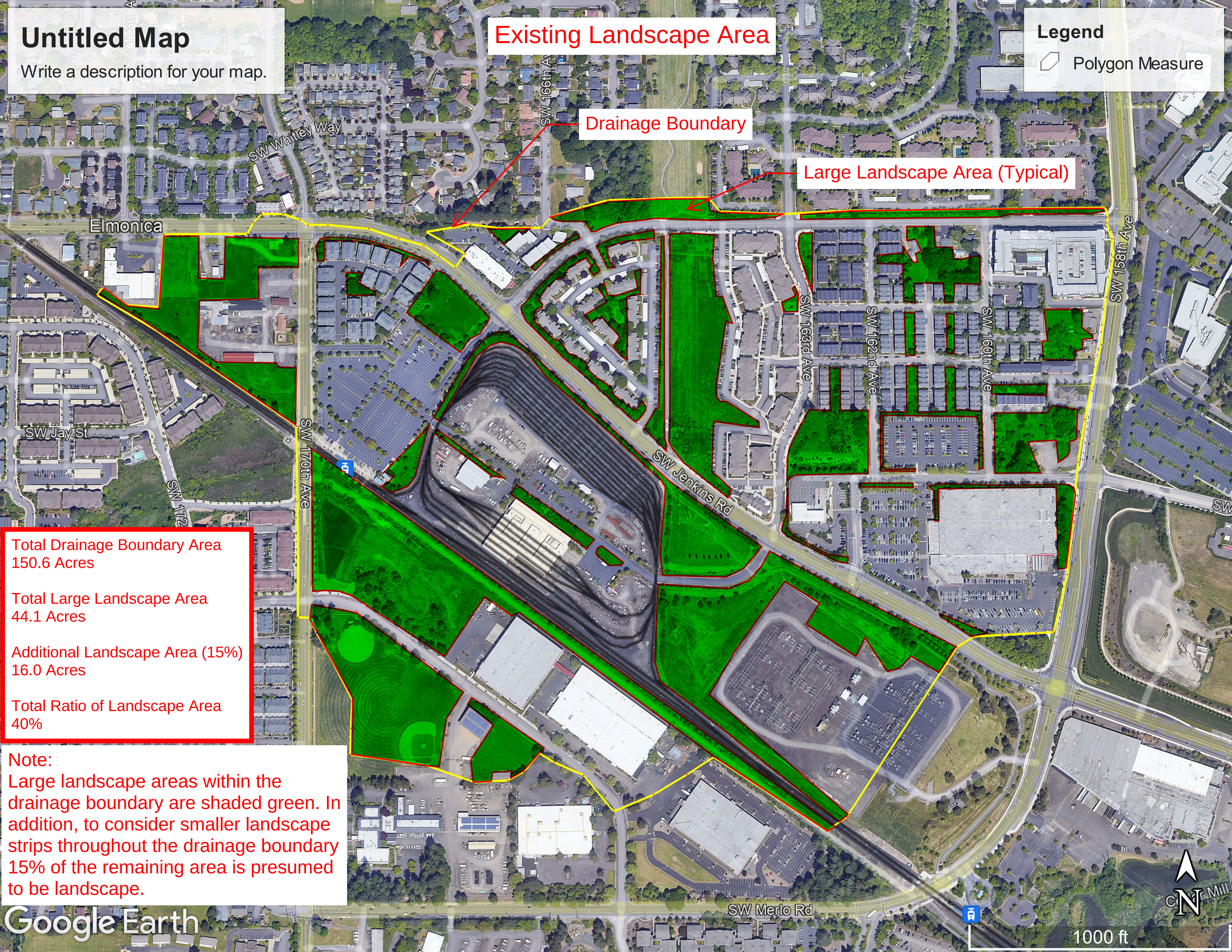
$$T_{scf} = \frac{L}{60V}$$

L	1670	ft	
V	1.2	ft/sec	Grassed Waterway (0.4%)
T_{scf}	**23**	min	

(C) Shallow Concentrated Flow (in pipe)

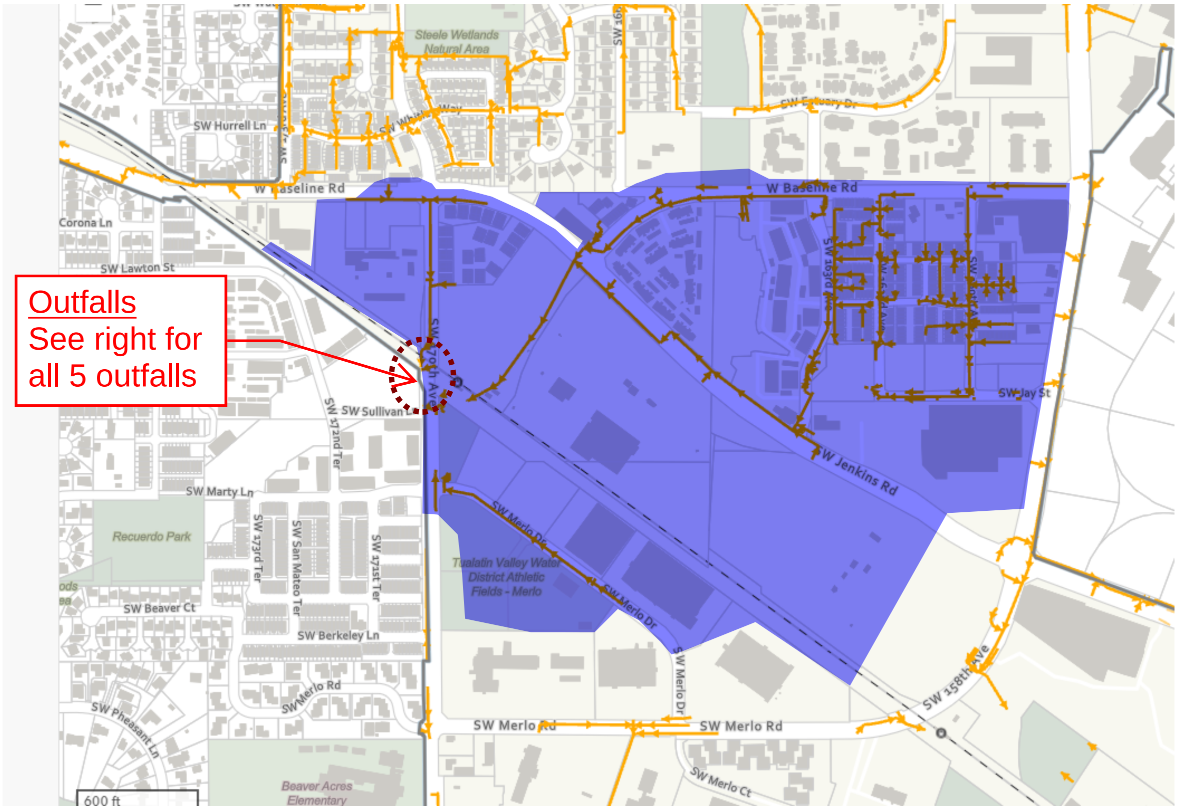
$$T_{scf} = \frac{L}{60V}$$

L	1673	ft	Length of travel in stormdrain pipe
V	2	ft/sec	(Assumed existing pipe velocity)
T_{scf}	**14**	min	


Untitled Map
Write a description for your map.
Existing Landscape Area
Legend
Polygon Measure
Drainage Boundary
Large Landscape Area (Typical)
Elmonica
SW Whitley Way
SW 166th Ave
SW 163rd Ave
SW 162nd Ave
SW 160th Ave
SW 158th Ave
SW 170th Ave
SW Jay St
SW Jenkins Rd
SW Merlo Rd
Total Drainage Boundary Area
150.6 Acres
Total Large Landscape Area
44.1 Acres
Additional Landscape Area (15%)
16.0 Acres
Total Ratio of Landscape Area
40%
Note:
Large landscape areas within the drainage boundary are shaded green. In addition, to consider smaller landscape strips throughout the drainage boundary 15% of the remaining area is presumed to be landscape.
Google Earth
1000 ft
N

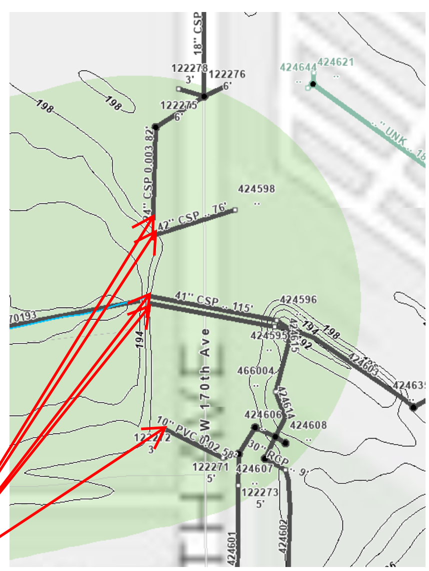
Attachment E
Wetland Upstream Drainage Area Pipe Network & Outfalls

Existing Stormwater Network From



* Image is taken from Beaverton Utilities Vewer arcgis

Existing Stormwater Outfalls x5



Outfalls

* Image is taken from Hydromodification Planning tool arcgis

ATTACHMENT F

REPORT OF GEOTECHNICAL ENGINEERING SERVICES

Elmonica
17160 West Baseline Road
Beaverton, Oregon

For
Rembold Properties, LLC
January 12, 2022

Project: Rembold-15-01

January 12, 2022

Rembold Properties, LLC
10305 SW Park Way, Suite 204
Portland, OR 97225

Attention: Chad Fackler

Report of Geotechnical Engineering Services
Elmonica
17160 West Baseline Road
Beaverton, Oregon
Project: Rembold-15-01

NV5 is pleased to submit this report of geotechnical engineering services for the Elmonica development located at 17160 West Baseline Road in Beaverton, Oregon. Our services for this project were conducted in accordance with our proposal dated December 6, 2021.

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

Sincerely,

NV5

Shawn M. Dimke, P.E., G.E.
Principal Engineer

cc: Kali Bader, Rembold Properties, LLC (via email only)

SMD:kt
Attachments
One copy submitted (via email only)
Document ID: Rembold-15-01-011222-geor.docx

EXECUTIVE SUMMARY

We understand the proposed development will include three five-story, wood-framed residential buildings; a one-story clubhouse building; and associated infrastructure. The following provides a summary of pertinent geotechnical considerations for the project. We recommend that the report be referenced for a thorough description of the subsurface conditions and geotechnical recommendations for the project.

- The proposed structures can be supported on conventional spread footings founded on minimum 6-inch-thick granular pads. The granular pads should be founded on firm, undisturbed native soil or on structural fill.
- We estimate up to 1 inch of cyclic softening-induced settlement is possible at the ground surface as a result of a design-level seismic event and differential settlement will be up to one-half of the total settlement.
- The undeveloped areas of the site have been used for agricultural purposes, with the surficial soil being tilled and possibly fertilized. Undocumented fill was also encountered to depths between 2.5 and 3.5 feet BGS in test pits TP-2 through TP-6 and to a depth of 5 feet BGS in prior boring B-2. We recommend removing all soft undocumented fill if present in structural areas. Where the tilled zone and undocumented fill will not be removed by cuts required for site grading and to remove soft fill, we recommend improving the upper 12 inches of subgrade by removing and replacing with granular structural fill or scarifying and re-compacting the on-site soil to structural fill requirements. Cement amendment is also a good option for improvement and adds the benefit of wet weather subgrade protection. The cost of improving subgrades for pavement and building slab areas should be included in the project budget.
- The fine-grained soil present on this site is easily disturbed during the wet season and after the removal of overlying hardscapes. If not carefully executed, site earthwork can create extensive soft areas and significant repair costs can result. Subgrade protection will be required when the subgrade is wet or above the optimum moisture content for compaction.
- The moisture content of the soil encountered at the site is above that required for compaction; drying will likely be required for use as structural fill. The on-site soil will only be suitable as structural fill during dry summer months.
- Due to shallow groundwater and very low tested infiltration rates, stormwater infiltration is not considered feasible as the primary means for managing stormwater from the site. We recommend infiltration systems, if used, have a redundant overflow system.
- Foundation drains are recommended around the perimeter of the proposed buildings due to relatively shallow perched groundwater conditions observed at the site.

TABLE OF CONTENTS **PAGE NO.**

TABLE OF CONTENTS **PAGE NO.**

ACRONYMS AND ABBREVIATIONS

AC	asphalt concrete
ACP	asphalt concrete pavement
ASCE	American Society of Civil Engineers
ASTM	American Society for Testing and Materials
BGS	below ground surface
CPT	cone penetration test
DEQ	Oregon Department of Environmental Quality
ECSI	Environmental Cleanup Site Information
g	gravitational acceleration (32.2 feet/second^2)
H:V	horizontal to vertical
MCE	maximum considered earthquake
OSHA	Occupational Safety and Health Administration
OSSC	Oregon Standard Specifications for Construction (2021)
pcf	pounds per cubic foot
pci	pounds per cubic inch
psi	pounds per square inch
PG	performance grade
psf	pounds per square foot
µm	micrometer

1.0 INTRODUCTION

NV5 is pleased to submit this report of geotechnical engineering services for the proposed Elmonica development located at 17160 West Baseline Road in Beaverton, Oregon. The site location relative to surrounding physical features is shown on Figure 1. Acronyms and abbreviations used herein are defined above, immediately following the Table of Contents.

Based on preliminary concept information and plans provided by Ankrom Moisan Architects, we understand the proposed development will include three five-story, wood-framed residential buildings; a one-story clubhouse building; and associated infrastructure. Building loads were unknown at the time of this report. We estimate column loads may range up to 300 kips and perimeter footing loads for walls may range up to 6 kips per lineal foot. Floor slab loads are expected to be less than 100 psf. The site slopes gently from the northern end down to the southeastern corner. Considering the gently sloping site grades and proposed development, we anticipate cuts and fills will be less than 5 feet each.

2.0 SCOPE OF SERVICES

The purpose of our services was to explore subsurface conditions at the site and provide geotechnical engineering recommendations for design and construction of the proposed development. Specifically, we completed the following scope of services:

- Reviewed readily available, published geologic data and our in-house files for existing information on subsurface conditions at the site.
- Coordinated and managed the field explorations, including public utility locates and scheduling subcontractors and NV5 staff.
- Completed the following subsurface explorations at the site:
  - Excavated six test pits (TP-1 through TP-6) to a depth of 10.5 feet BGS. The logs of the test pits are presented in Appendix A.
  - Performed infiltration tests in two test pits (TP-2 and TP-6) at a depth of 4 feet BGS.
- Collected disturbed soil samples for laboratory testing and maintained a log of encountered soil and groundwater conditions in the test pits.
- Completed the following laboratory testing on select soil samples:
  - Eleven moisture content determinations in general accordance with ASTM D2216
  - Two particle-size analyses in general accordance with ASTM D1140
  - One Atterberg limits test in general accordance with ASTM D4318
- Provided recommendations for site preparation, grading and drainage, stripping depths, fill type for imported material, compaction criteria, trench excavation and backfill, use of on-site soil, and wet/dry weather earthwork.
- Provided geotechnical engineering recommendations for design and construction of shallow spread foundations, including allowable design bearing pressure and minimum footing depth and width.
- Provided recommendations for preparation of the subgrade for floor slabs.
- Provided design criteria recommendations for conventional retaining structures, including lateral earth pressures, backfill, compaction, and drainage.
- Provided seismic coefficients in accordance with ASCE 7-16.

- Provided recommendations for managing identified groundwater conditions that may affect the performance of structures or pavement.
- Provided recommendations for construction of AC pavement for on-site drive aisles and parking areas, including subbase, base course, and AC paving thickness.
- Prepared this geotechnical engineering report that presents our findings, conclusions, and recommendations.

3.0 BACKGROUND

NV5 (formerly GeoDesign, Inc.) advanced direct-push borings at the site in 2007 for environmental sampling and is currently preparing a Phase I Environmental Site Assessment. We also reviewed a draft geotechnical report (GRI, 2020) and an environmental site assessment report (Alpha Environmental, 2020) for the site. The results of borings B-1 and B-2 from the draft geotechnical report are presented in Appendix B, and the results of CPT probes CPT-1 through CPT-4 from the draft geotechnical report are presented in Appendix C. After completion of the prior environmental site assessment, DEQ issued a No Further Action determination for ECSI File No. 1008 for the site in June 2020.

4.0 SITE CONDITIONS

4.1 GEOLOGIC SETTING

The site is located in the Tualatin Basin of the Puget Sound-Willamette Valley physiographic province, a tectonically active lowland located along the convergent Cascadia margin (Orr and Orr, 1999). The Tualatin Basin is formed by a gentle syncline between the uplifted Coast Ranges to the west, the Chehalem Mountains to the south, and the Tualatin Mountains to the north and east. The Tualatin Mountains have been uplifted along northwesterly oriented faults, including the steeply dipping Portland Hills fault located along the eastern flank of the mountains.

The near-surface geologic unit mapped at the site is the fine-grained facies of the Quaternary flood deposits (Madin, 1990). The unit consists of unconsolidated silt and sand deposited by catastrophic floods associated with the sudden release of waters from glacial Lake Missoula during the late Pleistocene. Many dozens of these Missoula floods occurred between approximately 15,500 and 12,500 years ago (and perhaps during earlier glaciations). Flood waters several hundred feet deep swept out of the Columbia Gorge and over the lowlands of the Portland area. The thickness of the flood deposits in the site vicinity is approximately 30 to 50 feet.

The Hillsboro Formation, aka the Sandy River Mudstone equivalent, underlies the Quaternary flood deposits. The thickness of the unit in the site vicinity is approximately 250 feet (Madin, 1990). The Hillsboro Formation is typically comprised of stiff, gray to brown, silty clay (Madin, 2009).

Basement rocks underlying the Sandy River Mudstone equivalent in the site vicinity consist of the Miocene Columbia River Basalts, emplaced approximately 17 million to 6 million years ago in the Portland area (Madin, 1990). The Columbia River Basalts are exposed in the Tualatin Basin in the highlands surrounding the valley and in a group of mountains south of the site, which include Cooper Mountain and Bull Mountain. The Columbia River Basalts consist of thick flows of basalt erupted from fissures in eastern Oregon, Washington, and western Idaho that traveled down the ancient Columbia River Gorge to fill the lowland areas around Portland.

4.2 SURFACE CONDITIONS

The approximately 5.43-acre site is bound by West Baseline Road and a vacant lot with a slab from a former building to the north, SW 170th Avenue to the east, the Tri-Met Westside Light Rail line to the south, and to an industrial/office building and parking lot to the west. The western and southern ends of the site are vacant fields. The fields are vegetated with grass, except for a short stockpile that is overgrown with blackberry bushes. The central and eastern portions of the site are developed with two wood-framed buildings, three one-story warehouses, a parking lot, and associated smaller structures and pavement. The north-central portion of the site is developed with a wood-framed residence, two associated garages, and a barn. The site slopes gently from an elevation of approximately 215 feet at the northern end to 196 feet at the southern end.

4.3 SUBSURFACE CONDITIONS

4.3.1 General

We explored subsurface conditions at the site by excavating six test pits (TP-1 through TP-6) to a depth of 10.5 feet BGS. Figure 2 shows the approximate exploration locations. A description of the test pit explorations and laboratory testing program, the test pit logs, and the results of laboratory testing are presented in Appendix A. The boring logs and results of laboratory testing are presented in Appendix B. A description of the CPT program and the results of the CPT probes are presented in Appendix C.

4.3.2 Root Zone, Tilled Zone, AC, and Undocumented Fill

An approximately 6- to 12-inch-thick tilled zone and 3- to 6-inch-thick root zone was encountered in all of the test pits, which were excavated in the vacant field areas of the site. Prior boring B-1 encountered a 1.5-inch-thick AC layer over 2 inches of base rock at the site.

Undocumented fill was encountered to depths between 2.5 and 3.5 feet BGS in test pits TP-2 through TP-6 and to a depth of 5 feet BGS in prior boring B-2. The undocumented fill generally consists of medium stiff to very stiff silt. A layer of dense, black gravel with asphalt pieces was also encountered from 0.8 to 1 foot BGS in test pit TP-4 and a buried AC layer was also encountered from 1 foot to 1.5 feet BGS in test pit TP-5.

4.3.3 Silt and Clay

Soil conditions generally consist of fine-grained silt and clay with variable sand content to the depths explored. The prior borings indicate the clay content generally increases with depth and the soil transitions to a silty clay to clayey silt at 40 feet BGS. Based on our test pits and the prior borings, the silt and clay ranges from very soft to very stiff. The prior CPT probes indicate there are some thin lenses or layers of hard, fine-grained soil between 15 and 30 feet BGS and the lowest tip resistance in the soil profile is near 10 feet BGS. Results of an Atterberg limits test indicate the silt exhibits low plasticity. Laboratory testing indicates that the moisture content of the silt ranged from 22 to 39 percent at the time of our explorations.

4.3.4 Groundwater

Groundwater seepage was encountered at depths between 2 and 10 feet BGS in test pits TP-2 through TP-6 and groundwater seepage was not encountered to a depth of 10.5 feet BGS in test pit TP-1. Based on Google Earth, there is a drainage and wetland area with a low elevation of approximately 192 feet located 200 feet south of the site. Considering the results of our explorations and observations, we expect perched groundwater at shallow depths at the site during the wet season. Pore pressure dissipation testing in the prior CPT probes indicated a groundwater table between approximately 7 and 15 feet BGS. The depth to groundwater may fluctuate in response to seasonal changes, changes in surface topography, and other factors not observed during our explorations.

4.4 SEISMIC HAZARDS

4.4.1 Liquefaction and Cyclic Softening

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 interparticle 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, sandy silt may be moderately susceptible to liquefaction under relatively high levels of ground shaking. Non-plastic and low plasticity, fine-grained material may be subject to cyclic softening from an increase in pore water pressure and a reduction in strength during seismic shaking; however, the relatively poor drainage characteristics of silt deposits inhibit the occurrence of a rapid decrease in volume.

We performed a liquefaction analysis using the results of the CPT explorations. Our analysis indicates thin layers or lenses of low plasticity silt below the groundwater table may be prone to cyclic softening. We estimate up to 1 inch of cyclic softening-induced settlement at the ground surface as a result of a design-level seismic event. We estimate differential settlement will be up to one-half of the total settlement over a distance of 50 feet. The estimated settlement is typically within allowable design tolerances for structures. However, if the estimated settlements are greater than allowable, we can be contacted to provide alternate intermediate foundation recommendations and/or further evaluate the liquefaction/cyclic softening potential at the site.

4.4.2 Lateral Spreading

Lateral spreading is a liquefaction-related seismic hazard. Areas subject to lateral spreading are typically gently sloping or flat sites underlain by liquefiable sediment adjacent to an open face, such as a riverbank. Since there are no open faces in the site vicinity, the potential for lateral spreading at the site is low.

4.5 INFILTRATION TESTING

Infiltration testing was performed in two test pits (TP-2 and TP-6). The infiltration tests were completed at a depth of 4 feet BGS. The infiltration testing procedures are described in Appendix A, and the results of the infiltration and laboratory testing are summarized in Table 1.

Table 1. Infiltration and Laboratory Testing Summary

Location	Depth (feet BGS)	Observed Infiltration Rate[1] (inches per hour)	Fines Content[2] (percent)	Soil Type at Test Depth
TP-2	4	0.3	96	SILT, trace sand and clay
TP-6	4	0.7	96	SILT, trace sand and clay

1. In-situ infiltration rate observed in the field
2. Fines content - material passing the U.S. Standard No. 200 sieve

5.0 DESIGN RECOMMENDATIONS

5.1 GENERAL

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

5.2 SHALLOW FOUNDATIONS

5.2.1 General

Based on the assumed foundation loads, the proposed development can be supported by shallow foundations established on minimum 6-inch-thick granular pads. The granular pads should be founded on firm, undisturbed native soil or on structural fill.

If loose or soft material, organic material, tilled soil, unsuitable fill, or prior topsoil zones are encountered for footing subgrades, over-excavation for granular pads may be necessary. Granular pads should extend 6 inches beyond the margins of the footings for every foot excavated below the footing's base grade. The granular pads should consist of imported granular material, as defined in the "Structural Fill" section. The imported granular material should be compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D1557, or until well keyed, as determined by one of our geotechnical staff. We recommend that a member of our geotechnical staff observe the prepared footing subgrade.

5.2.2 Dimensions and Capacities

Shallow footings should be proportioned for a maximum allowable soil bearing pressure of 2,500 psf. This bearing pressure is a net bearing pressure and applies to the total of dead and long-term live loads and can be increased by one-half when considering seismic or wind loads. The weight of the footing and overlying backfill can be ignored in calculating footing loads. Isolated spread footings and continuous spread footings should be at least 18 and 12 inches wide, respectively. 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 top of the floor slab/ground surface.

5.2.3 Settlement

Based on our experience with similar soil, total static post-construction settlement should be less than 1 inch, with differential settlement of approximately one-half the total over a 50-foot span.

5.2.4 Resistance to Sliding

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. We recommend a friction coefficient of 0.45 for computing the friction capacity of building foundations that bear on granular pads. Our analysis indicates that the available passive earth pressure for footings confined by structural fill or footings constructed in direct contact with the undisturbed native soil or structural fill is 350 pcf. Typically, the movement required to develop the available passive resistance may be relatively large; therefore, we recommend using a reduced passive pressure of 250 pcf equivalent fluid pressure. Adjacent floor slabs, pavement, or the upper 12-inch depth of adjacent, unpaved areas should not be considered when calculating passive resistance. In addition, in order to rely on the recommended passive resistance, the groundwater level must be below the base of the footing and a minimum of 5 feet of horizontal clearance must exist between the face of the footings and any adjacent downslopes.

5.3 FLOOR SLABS

A subgrade modulus of 150 pci can be used to design the floor slabs. A minimum 6-inch-thick layer of base rock should be placed and compacted over the prepared subgrade to assist as a capillary break. The base rock should be crushed rock or crushed gravel and sand meeting the requirements outlined in the "Structural Fill" section. The imported granular material should be placed in one lift and compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D1557. Floor slab base rock should be replaced if it becomes contaminated with excessive fines (greater than 5 percent by dry weight passing the U.S. Standard No. 200 sieve). Vapor barriers are often required by flooring manufacturers to protect flooring and flooring adhesives. Many flooring manufacturers will warrant their product only if a vapor barrier is installed according to their recommendations. Selection and design of an appropriate vapor barrier (if needed) should be based on discussions among members of the design team. We can provide additional information to assist you with your decision.

5.4 RETAINING STRUCTURES

5.4.1 Assumptions

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

5.4.2 Wall Design Parameters

Retaining structures free to rotate slightly around the base should be designed for active earth pressures using an equivalent fluid unit pressure of 35 pcf. If retaining walls are restrained against rotation during backfilling, they should be designed for an at-rest earth pressure of 55 pcf. For embedded building walls, a superimposed seismic lateral force should be calculated based on a dynamic force of $7H^2$ pounds per lineal foot of wall, where H is the height of the wall in feet, and applied at 0.6H from the base of the wall. If other surcharges (e.g., slopes steeper than 4H:1V, foundations, vehicles, etc.) are located within a horizontal distance from the back of a wall equal to twice the height of the wall, additional pressures may need to be accounted for in the wall design. Our office should be contacted for appropriate wall surcharges based on the actual magnitude and configuration of the applied loads.

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

5.4.3 Wall Drainage and Backfill

The above design parameters have been provided assuming back-of-wall drains will be installed to prevent buildup of hydrostatic pressures behind all walls. If a drainage system is not installed, our office should be contacted for revised design forces. The backfill material placed behind the walls and extending a horizontal distance of ½H, where H is the height of the retaining wall, should consist of retaining wall select backfill placed and compacted in conformance with the "Structural Fill" section.

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

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

5.5 SEISMIC DESIGN CRITERIA

Seismic design is prescribed by ASCE 7-16. Table 2 presents the design parameters prescribed by ASCE 7-16 for the site. Due to the presence of potentially liquefiable soil, the Site Class is F; however, the design parameters for Site Class D provided below can be used per ASCE 7-16, provided the fundamental period of the structure is 0.5 second or less.

Table 2. Seismic Design Parameters

Parameter	Short Period (T_s = 0.2 second)	1 Second Period (T_1 = 1.0 second)
MCE Spectral Acceleration, S	S_s = 0.891 g	S_1 = 0.413 g
Site Class	F*	
Site Coefficient, F	F_a = 1.2	F_v = 1.887
Adjusted Spectral Acceleration, S_M	S_{MS} = 1.069 g	S_{M1} = 0.779 g
Design Spectral Response Acceleration Parameters, S_D	S_{DS} = 0.713 g	S_{D1} = 0.520 g

* The above parameters provided for Site Class D can be used, provided the structure has a fundamental period of 0.5 second or less per ASCE 7-16 Section 20.3.1 and the seismic response coefficient (C_s) is determined according to the exception in ASCE 7-16 Section 11.4.8 or else a site-specific response analysis will be required.

5.6 PAVEMENT

New pavement should be constructed on competent subgrade or new engineered fill prepared in conformance with the “Site Preparation” and “Structural Fill” sections. We do not have specific information on the frequency and type of vehicles expected at the site. Based on our experience with similar projects, we anticipate that traffic will consist mainly of passenger vehicles and occasional two-axle trucks, such as delivery trucks or garbage trucks. We have assumed that all roads are private. Design for city or county roads may require thicker pavement sections than those provided in this report.

Our pavement recommendations are based on the following assumptions:

- A resilient modulus of 20,000 psi was estimated for the aggregate base.
- A resilient modulus of 3,500 psi for subgrades prepared as recommended in this report.
- Initial and terminal serviceability indices of 4.2 and 2.5, respectively.
- Reliability of 75 percent and standard deviation of 0.45.
- Structural coefficients of 0.42 and 0.14 for the AC and aggregate base, respectively.
- The design life of the pavement is 20 years.
- Zero growth over the design life.
- Trucks will have two axles.

If any of these assumptions vary from project design values, our office should be contacted with the appropriate information so that the pavement designs can be revised. Our pavement design recommendations and assumed traffic scenarios are summarized in Table 3. The design team should select the most appropriate design traffic level. The recommended pavement sections are capable of supporting an occasional 75,000-pound fire truck.

Table 3. Pavement Section Thickness

Traffic Levels		Pavement Section Thicknesses on On-Site Subgrade[1] (inches)		Pavement Section Thicknesses on Cement-Amended Subgrade[2] (inches)	
Cars/Day	Trucks/Day	AC	Base Rock	AC	Base Rock
200	0	2.5	8.0	2.5	4.0
400	5	3.0	9.0	3.0	4.0
400	10	3.0	11.0	3.0	4.0

1. All thicknesses are intended to be the minimum acceptable values.
2. Compressive strength of cement-amended soil should be at least 100 psi.

The AC, aggregate base, and cement amendment should meet the requirements outlined in the "Materials" section. Our pavement design assumes construction will be completed during an extended period of dry weather. Wet weather construction could require an increased thickness of aggregate base.

Construction traffic should be limited to non-structural portions of the site or haul roads. Construction traffic should not be allowed on new pavement. 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.

5.7 DRAINAGE CONSIDERATIONS

5.7.1 Temporary

During earthwork at the site, the contractor should be made responsible for temporary drainage of surface water as necessary to prevent standing water and/or erosion at the working surface.

5.7.2 Site Drainage

We recommend that all roof drains be connected to a tightline leading to storm drain facilities. Pavement surfaces and open space areas should be sloped such that surface water runoff is collected and routed to suitable discharge points. We also recommend that ground surfaces adjacent to the buildings be sloped away from the buildings to facilitate drainage away from the buildings. Trapped planter areas should not be created adjacent to pavement and structures without providing means for positive drainage (e.g., swales or catch basins).

5.7.3 Foundation Drains

Foundation drains are recommended for the proposed buildings due to the relatively shallow perched groundwater and relatively impervious fine-grained soil at the site. Foundation drains should be constructed at a minimum slope of approximately ½ percent and pumped or drained by gravity to a suitable discharge. The perforated drainpipe should not be tied to a stormwater drainage system without backflow provisions. Foundation drains should consist of 4-inch-diameter, perforated drainpipe embedded in a minimum 2-foot-wide zone of crushed drain rock wrapped in drainage geotextile that extends to within 12 inches of the ground surface. The invert elevation of the drainpipe should be installed at least 18 inches below the elevation of the floor slab. The drain rock and geotextile should meet the requirements specified in the "Materials" section.

5.8 INFILTRATION SYSTEMS

The results of our infiltration testing indicate that the on-site soil has very low infiltration capacity. In addition, groundwater seepage was observed at depths between 2 and 10 feet BGS in test pits TP-2 through TP-6.

The infiltration rates shown in Table 1 are short-term field rates and factors of safety have not been applied. If infiltration facilities will be used, correction factors should be applied to the measured infiltration rates by the civil engineer to account for soil variations and the potential for long-term clogging due to siltation and buildup of organic material, depending on the proposed length, location, and type of infiltration facility. We recommend a minimum factor of safety of at least 2 be applied to the field infiltration values.

The actual depths and estimated infiltration rates can vary significantly from these values. Based on the shallow depth to groundwater and very low tested infiltration rates, we recommend any infiltration system at the site not be the sole source for stormwater infiltration and facilities have a redundant overflow system. We recommend the installation of stormwater facilities be observed by a qualified geotechnical engineer or representative under their supervision to evaluate if soil conditions are consistent with subsurface conditions encountered during our explorations.

5.9 PERMANENT SLOPES

Permanent cut and fill slopes should not exceed 2H:1V. Fill slopes should be over-built by at least 12 inches and trimmed back to the required slope to maintain a firm face. Access roads and pavement should be located at least 5 feet from the top of cut and fill slopes. The setback should be increased to 10 feet for buildings.

The slopes should be planted with appropriate vegetation to provide protection against erosion as soon as possible after grading. Surface water runoff should be collected and directed away from slopes to prevent water from running down the face of the slope.

6.0 CONSTRUCTION RECOMMENDATIONS

6.1 SITE PREPARATION

6.1.1 Demolition

Demolition includes removal of abandoned utilities and any subsurface elements from previous on-site improvements throughout the proposed building footprint. Demolished material should be transported off site for disposal. Excavations remaining from site preparation activities should be backfilled with structural fill where below planned site grades. The base of excavations should be excavated to expose firm subgrade before filling. Utility lines abandoned under new structural elements should be completely removed and backfilled with structural fill in accordance with the recommendations in the "Structural Fill" section. Soft soil encountered in utility line excavations should be removed and replaced with structural fill. Concrete debris, AC pavement, and base rock can be used as structural fill, provided it is processed to meet the requirements in "Recycled Material" section.

6.1.2 Stripping and Grubbing

The near-surface root zone should be stripped and removed from the site in all proposed building and pavement areas and for a 5-foot margin around such areas. Based on our explorations, the depth of stripping will average approximately 4 inches. Additional areas surrounding larger trees will also require greater stripping depths 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. Trees and their root balls should be grubbed to the depth of the roots, which could exceed 5 feet BGS. Depending on the methods used to remove this material, considerable disturbance and loosening of the subgrade could occur. We recommend that disturbed soil be removed to expose stiff native soil. The resulting excavations should be backfilled with structural fill.

6.1.3 Tilled Zone Improvement

We observed an approximately 6- to 12-inch-thick tilled zone in the field areas at the site, and undocumented fills ranging up to 5 feet BGS were encountered at the site. The tilled zone is generally weaker, contains trace to minor organics and sand, and will provide inadequate support for structures. Undocumented fill is also typically weaker and has unpredictable properties. We recommend removing all soft undocumented fill where present in structural areas. We also recommend improving a minimum of the upper 12 inches of subgrade in structural areas where the tilled zone or undocumented fill is not removed from cuts required for site grading. Improvement can be accomplished by replacing the topsoil zone with imported structural fill or by scarifying and re-compacting the existing topsoil zone soil after sufficient stripping. Scarification is typically performed by ripping with agricultural discs. Considerable soil processing, including moisture conditioning, may be required to use the excavated material as structural fill. If the soil cannot be properly moisture conditioned, the subgrade should be removed and replaced with granular fill or cement amended as described in the "Materials" section. Scarifying and re-compacting will not be a viable option during extended periods of wet weather.

6.1.4 Subgrade Evaluation

A member of our geotechnical staff should observe the exposed subgrade after stripping, demolition, and site cutting have been completed to determine if there are areas of unsuitable or unstable soil. The subgrade should be proof rolled with a fully loaded dump truck or similar heavy, rubber tire construction equipment to identify soft, loose, or unsuitable areas after subgrade compaction is complete. Proof rolling should be observed by a qualified geotechnical engineer or their representative. 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.

6.1.5 Test Pit Locations

The test pit excavations were backfilled using relatively minimal compactive effort of tamping with the hoe bucket; therefore, soft spots can be expected at these locations. We recommend removing the relatively uncompacted soil from the test pits to a depth of 3 feet below finished subgrade. If a test pit is located within 5 feet of a footing, we recommend full-depth removal of the uncompacted soil. The resulting excavation should be brought back to grade with structural fill.

6.2 CONSTRUCTION CONSIDERATIONS

The fine-grained soil present on this site is easily disturbed. If not carefully executed, site preparation, utility trench work, and roadway excavation 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.

As an alternative to thickened crushed rock sections, haul roads and utility work zones may be constructed using cement-amended subgrades overlain by a crushed rock wearing surface. If this approach is used, the thickness of granular material in staging areas and along haul roads can typically be reduced to between 6 and 9 inches. This recommendation is based on an assumed minimum unconfined compressive strength of 100 psi for subgrade amended to a depth of 12 to 16 inches. The actual thickness of the amended material and imported granular material will depend on the contractor's means and methods and, accordingly, should be the contractor's responsibility. Cement amendment is discussed in the "Materials" section.

6.3 EXCAVATION

6.3.1 General

Conventional earthmoving equipment in proper working condition should be capable of making necessary excavations for pavement, foundations, and utilities. We recommend that excavation be performed by a track-mounted excavator using a smooth-blade bucket.

6.3.2 Trench Cuts and Trench Shoring

Trench cuts should stand vertical to a depth of approximately 4 feet, provided groundwater seepage does not occur. Open excavation techniques may be used to excavate trenches with depths between 4 and 10 feet BGS, provided the walls of the excavation are cut at a slope of 1H:1V and groundwater seepage is not present. Sloughing and caving will likely occur if the excavation extends below the groundwater table or if seepage is present. The walls of the trench should be flattened or braced for stability if excessive sloughing occurs, and the area dewatered if seepage is encountered.

Excavations should not undermine adjacent utilities, foundations, walkways, streets, or other hardscapes, unless special shoring or underpinned support is provided. Unsupported excavations should not be conducted within a downward and outward projection of a 1H:1V line from 5 feet outside the edge of an adjacent structural feature.

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

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

6.3.3 Dewatering

We anticipate that a sump located within the trench excavation likely will be sufficient to remove accumulated water, depending on the amount and persistence of water seepage and the length of time the trench is left open. Flow rates for dewatering are likely to vary depending on location, soil type, and the season during which the excavation occurs. Dewatering systems should be capable of adapting to variable flows. If groundwater and fine-grained soil are present in the base of the utility trench excavation, we recommend over-excavating the trench by 12 to 18 inches and placing trench stabilization material in the base.

6.3.4 Safety

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

6.4 MATERIALS

6.4.1 Structural Fill

6.4.1.1 General

Structural fill includes fill beneath foundations, slabs, pavement, any other areas intended to support structures, or within the influence zones of structures. Structural fill should be free of organic material and other deleterious material and, in general, should consist of particles no larger than 4 inches in diameter. A brief characterization of some of the acceptable materials and our recommendations for their use as structural fill are provided below.

6.4.1.2 On-Site Soil

The near-surface soil at the site is primarily fine grained. This soil can be used for structural fill, provided it can be adequately moisture conditioned. The site soil is sensitive to small changes in moisture content and is highly susceptible to disturbance when wet. Use of the on-site material as structural fill will not be possible during the wet season, which typically extends from mid-October to early June.

We estimate the optimum moisture content for compaction to be approximately 14 to 18 percent for the on-site soil. Optimum compaction typically occurs within 3 percent of optimum moisture. Typically, the moisture content for the on-site soil will be greater than the anticipated optimum moisture content required for adequate compaction. It is likely that even during the dry season, drying will be required to achieve adequate compaction. We recommend using imported granular material for structural fill if the on-site material cannot be properly moisture conditioned.

When used as structural fill, the on-site soil should be placed in lifts with a maximum uncompacted thickness of 6 to 8 inches. The soil should be compacted to not less than 92 percent of the maximum dry density, as determined by ASTM D1557.

6.4.1.3 Imported Granular Material

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

When used as structural fill, imported granular material should be placed in lifts with a maximum uncompacted thickness of 12 inches and compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D1557.

6.4.1.4 Stabilization Material

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

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

6.4.1.5 Trench Backfill

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

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

6.4.1.6 Aggregate Base Rock

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

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

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

6.4.1.8 Drain Rock Material

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

6.4.1.9 Recycled Material

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

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

6.4.2 Geotextile Fabric

6.4.2.1 Separate Geotextile Fabric

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

6.4.2.2 Drainage Geotextile Fabric

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

6.4.3 AC

The AC should be Level 2, ½-inch, dense ACP as described in OSSC 00744 (Asphalt Concrete Pavement) and compacted to 91 percent of the specific gravity of the mix, as determined by ASTM D2041. The minimum and maximum lift thickness is 2.0 and 3.0 inches, respectively, for ½-inch ACP. Asphalt binder should be performance graded and conform to PG 64-22 or better.

6.4.4 Cement Amendment

6.4.4.1 General

As an alternative to the use of imported granular material for wet weather structural fill, an experienced contractor may be able to amend the on-site soil with portland cement to obtain suitable support properties. Successful use of soil amendment depends on the use of correct mixing techniques, soil moisture content, and amendment quantities.

6.4.4.2 Subbase Stabilization

Specific recommendations based on exposed site conditions for soil amending can be provided if necessary. However, for preliminary design purposes, we recommend a target strength for cement-amended subgrade for building and pavement subbase (below aggregate base) soil of 100 psi. Successful use of soil amendment depends on use of correct techniques and equipment, soil moisture content, and the amount of cement added to the soil. The recommended percentage of cement is based on soil moisture contents at the time of placing the structural fill. Based on our experience, 5 percent cement by weight of dry soil is generally satisfactory when the soil moisture content does not exceed approximately 25 percent. If the soil moisture content is in the range of 25 to 35 percent, 6 to 8 percent by weight of dry soil is recommended. It is difficult to accurately predict field performance due to the variability in soil response to cement amendment. The amount of cement added to the soil may need to be adjusted based on field observations and performance. Moreover, depending on the time of year and moisture content levels during amendment, water may need to be applied during tilling to appropriately condition the soil moisture content. The amount of cement used during amendment should be based on an assumed soil dry unit weight of 100 pcf. For preliminary design purposes, we recommend a minimum of 6 percent cement. It is not possible to amend soil during heavy or continuous rainfall. Work should be completed during suitable conditions

We recommend cement-spreading equipment be equipped with balloon tires to reduce rutting and disturbance of the fine-grained soil. A static sheepsfoot or segmented pad roller with a minimum static weight of 40,000 pounds should be used for initial compaction of the fine-grained soil. A smooth-drum roller with a minimum applied linear force of 700 pounds per inch should be used for final compaction. The amended soil should be compacted to at least 92 percent of the achievable dry density at the moisture content of the material, as defined in ASTM D1557.

A minimum curing time of four days is required between amendment and construction traffic access. Construction traffic should not be allowed on unprotected, cement-amended subgrade. To protect the cement-amended surfaces from abrasion or damage, the finished surface should be covered with 4 to 6 inches of imported granular material.

Amendment depths for building/pavement, haul roads, and staging areas are typically on the order of 12, 16, and 12 inches, respectively. The crushed rock typically becomes contaminated with soil during construction. Contaminated base rock should be removed and replaced with clean rock in pavement areas. The actual thickness of the amended material and imported granular material for haul roads and staging areas will depend on the anticipated traffic, as well as the contractor's means and methods and, accordingly, should be the contractor's responsibility. Cement amendment should not be attempted when air temperature is below 40 degrees Fahrenheit or during moderate to heavy precipitation. Cement should not be placed when the ground surface is saturated or standing water exists.

6.4.4.3 Cement-Amended Structural Fill

On-site silt/clay soil that is not suitable for structural fill due to high moisture content may be amended and placed as fill over a subgrade prepared in conformance with the "Site Preparation" section. Cement-amended fill lift thicknesses should be limited to 12 inches. The cement ratio for general cement-amended fill can generally be reduced by 1 percent (by dry weight). Typically, a minimum curing time of four days is required between amendment and construction traffic access. Consecutive lifts of fill may be amended immediately after the previous lift has been amended and compacted (e.g., the four-day wait period does not apply). However, where the final lift of fill is a building or roadway subgrade, the four-day wait period is in effect for the final lift of cement-amended soil.

6.4.4.4 Other Considerations

Portland cement-amended soil is hard and has low permeability. This soil does not drain well and it is not suitable for planting. Future planted areas should not be cement amended, if practical, or accommodations should be made for drainage and planting. Moreover, cement amending soil within building areas must be done carefully to avoid trapping water under floor slabs. We should be contacted if this approach is considered. Cement amendment should not be used if runoff during construction cannot be directed away from adjacent wetlands (if any). Cement amendment runoff should be collected, monitored, and treated in accordance with DEQ requirements prior to being discharged.

6.4.4.5 Specification Recommendations

We recommend that the following comments be included in the specifications for the project:

- In general, cement amendment is not recommended during the cold weather (temperatures less than 40 degrees Fahrenheit) or during rainfall.
- Mixing Equipment
    - Use a pulverizer/mixer capable of uniformly mixing the cement into the soil to the design depth. Blade mixing will not be allowed.
    - Pulverize the soil-cement mixture such that 100 percent by dry weight passes a 1-inch sieve and a minimum of 70 percent passes a No. 4 sieve, exclusive of gravel or stone retained on these sieves. If water is required, the pulverizer should be equipped to inject water to a tolerance of ¼ gallon per square foot of surface area.
    - Use machinery that will not disturb the subgrade, such as using low-pressure "balloon" tires on the pulverizer/mixer vehicle. If subgrade is disturbed, the tilling/amendment depth shall extend the full depth of the disturbance.
    - Multiple "passes" of the tiller may be required to adequately blend the cement and soil mixture.
- Spreading Equipment
    - Use a spreader capable of distributing the cement uniformly on the ground to within 5 percent variance of the specified application rate.
    - Use machinery that will not disturb the subgrade, such as using low-pressure "balloon" tires on the spreader vehicle. If subgrade is disturbed, the tilling/amendment depth shall extend the full depth of the disturbance.
- Compaction Equipment
    - Use a static, sheepsfoot or segmented pad roller with a minimum static weight of 40,000 pounds for initial compaction of fine-grained soil (silt and clay) or an alternate approved by the geotechnical engineer.

6.5 EROSION CONTROL

The site soil is moderately susceptible to erosion; therefore, erosion control measures should be carefully planned and in place before construction begins. Surface water runoff should be collected and directed away from slopes to prevent water from running down the slope face. Erosion control measures (such as straw bales, sediment fences, and temporary detention and settling basins) should be used in accordance with local and state ordinances.

7.0 OBSERVATION OF CONSTRUCTION

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

We recommend that NV5 be retained to observe earthwork activities, including stripping, proof rolling of the subgrade and repair of soft areas, footing subgrade preparation, final proof rolling of the pavement subgrade and base rock, and AC placement and compaction, and performing laboratory compaction and field moisture-density tests.

8.0 LIMITATIONS

We have prepared this report for use by Rembold Properties and other members of the design and construction team 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 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 was conceptual at the time this report was prepared. When the design has been finalized and if there are changes in the site grades, design traffic, or type of construction, 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 of our services does not include services related to construction safety precautions, and our recommendations are not intended to direct the contractor's methods, techniques, sequences, or procedures, except as specifically described in this report for consideration in design.

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

♦ ♦ ♦

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,

NV5

Shawn M. Dimke, P.E., G.E.
Principal Engineer

REFERENCES

Alpha Environmental, 2020. *Environmental Site Assessment Report; ECSI #1008; Pacific Crest Supply; 1065 SW 170th Avenue; Beaverton, Oregon 97003*, dated May 28, 2020.

GRI, 2020. DRAFT *Geotechnical Investigation; Proposed New Apartments and Parking Garage; 17160 W Baseline Road; Beaverton, Oregon*, dated February 7, 2020.

Madin, Ian P., 1990. Earthquake-Hazard Geology Maps of the Portland Metropolitan Area, Oregon: Text and Map Explanation, Oregon Department of Geology and Mineral Industries, Open-File Report O-90-2, 21p., 8 plates, scale 1:24,000.

Madin, Ian P., 2009. Portland, Oregon geology by tram, train, and foot, Oregon Geology, Volume 69, No. 1, 2009, pp 73-92.

Orr, E.L. and W.N. Orr, 1999. Geology of Oregon. Kendall/Hunt Publishing Company, Iowa: 254 p.

FIGURES

NV5	REMBOLD-15-01	VICINITY MAP	
	JANUARY 2022	ELMONICA BEAVERTON, OR	FIGURE 1

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LEGEND:

— — —	SITE BOUNDARY
TP-1	TEST PIT
B-1	PRIOR BORING (GRI, AUGUST 2019)
CPT-1	PRIOR CPT (GRI, JUNE/NOVEMBER 2019)

N

0 100 200

(SCALE IN FEET)

NOTES:

1. SITE PLAN BASED ON IMAGE OF OPTION B - SITE PLAN - OVERALL DATED OCTOBER 27, 2021 PREPARED BY ANKROM MOISAN INC.
2. AERIAL PHOTOGRAPH OBTAINED FROM GOOGLE EARTH PRO JANUARY 6, 2022.

NV5

REMBOLD-15-01

JANUARY 2022

SITE PLAN

ELMONICA
BEAVERTON, OR

FIGURE 2

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APPENDIX A

APPENDIX A

FIELD EXPLORATIONS

GENERAL

We explored subsurface conditions at the site by excavating six test pits (TP-1 through TP-6) to a depth of 10.5 feet BGS. The test pits were excavated by Dan J. Fischer Excavating, Inc. of Forest Grove, Oregon, on January 3, 2022, using a mini excavator. The explorations were completed under the supervision of NV5 personnel. The exploration logs are presented in this appendix.

The locations of the explorations were determined in the field by pacing from existing site features. This information should be considered accurate to the degree implied by the method used.

SOIL SAMPLING

Representative disturbed samples of soil observed in the test pit explorations were collected from the test pit walls and base using the excavator bucket. Sampling methods and intervals are shown on the exploration logs.

SOIL CLASSIFICATION

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

INFILTRATION TESTING

Infiltration tests were conducted in test pits TP-2 and TP-6 at a depth of 4 feet BGS. The underlying soil was saturated by allowing the water to infiltrate into the subsurface. The infiltration rate was measured under low-head conditions after saturated conditions had been achieved. Infiltration testing was completed using the single-ring falling head test. Infiltration testing was completed by pouring water into the holes and measuring the drop in water with respect to time. Testing was completed until consistent rates were achieved.

LABORATORY TESTING

CLASSIFICATION

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

MOISTURE CONTENT

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

PARTICLE-SIZE ANALYSIS

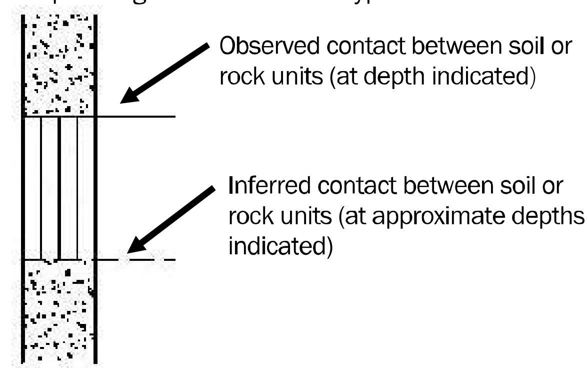
Fines content determinations were completed on select soil samples in general accordance with ASTM D1140 (percent passing the U.S. Standard No. 200 sieve). This test determines the fraction of the soil particles in a sample that are finer than 75 μm expressed as percentage of its dry weight. The test results are presented in this appendix.

ATTERBERG LIMITS

The plastic limit and liquid limit (Atterberg limits) of a select soil sample was determined in accordance with ASTM D4318. The Atterberg limits and the plasticity index were completed to aid in the classification of the soil and evaluation of liquefaction susceptibility. The plastic limit is defined as the moisture content (in percent) where the soil becomes brittle. The liquid limit is defined as the moisture content where the soil begins to act similar to a liquid. The plasticity index is the difference between the liquid and plastic limits. The test results are presented in this appendix.

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

Graphic Log of Soil and Rock Types



GEOTECHNICAL TESTING EXPLANATIONS

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

ENVIRONMENTAL TESTING EXPLANATIONS

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

NV5

EXPLORATION KEY | **TABLE A-1**

RELATIVE DENSITY - COARSE-GRAINED SOIL

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

CONSISTENCY - FINE-GRAINED SOIL

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

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

MOISTURE CLASSIFICATION

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

ADDITIONAL CONSTITUENTS

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

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

NV5	SOIL CLASSIFICATION SYSTEM	TABLE A-2

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	● MOISTURE CONTENT % (0 50 100)	COMMENTS
0.0		Medium stiff, brown SILT (ML), minor clay and sand, trace organics; moist to wet (9-inch-thick tilled zone, 4-inch-thick root zone).		PP			PP = 0.75 tsf
		Stiff, brown SILT (ML), trace sand and clay; moist.	0.8				
2.5				PP	☒	●	PP = 1.5 tsf
5.0		some clay at 6.0 feet					
7.5				PP	☒		PP = 1.5 tsf
10.0				PP	☒		PP = 1.5 tsf
		Exploration completed at a depth of 10.5 feet.	10.5				No groundwater seepage observed to the depth explored. No caving observed to the depth explored. Surface elevation was not measured at the time of exploration.
12.5							
15.0							

EXCAVATED BY: Dan J. Fischer Excavating, Inc.

LOGGED BY: P. Chaichi

COMPLETED: 01/03/22

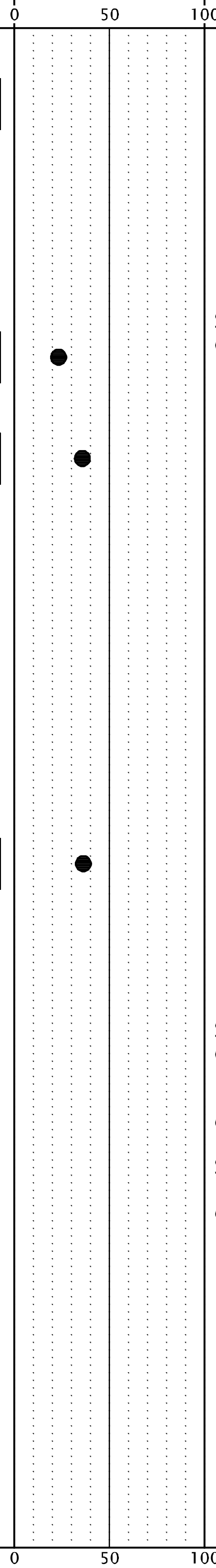
EXCAVATION METHOD: mini excavator (see document text)

NV5	REMBOLD-15-01	**TEST PIT TP-1**
	JANUARY 2022	ELMONICA BEAVERTON, OR

FIGURE A-1

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE	COMMENTS
0.0	Medium stiff, brown SILT (ML), minor clay and sand, trace organics; moist to wet (12-inch-thick tilled zone, 4-inch-thick root zone).			☒	
	Stiff, brown SILT (ML), minor clay and sand; moist, blocky structure - **FILL**.	1.0			
2.5					
	Stiff, brown SILT (ML), trace clay and sand; moist, silt has low plasticity.	3.0	PP	☒	Slow groundwater seepage observed at 3.0 feet. PP = 1.0 tsf
			P200	☒	Infiltration test at 4.0 feet. P200 = 96%
5.0					
7.5					
			ATT PP	☒	PP = 1.25 tsf LL = 37% PL = 25%
10.0					Slow groundwater seepage observed at 10.0 feet.
	Exploration completed at a depth of 10.5 feet.	10.5			No caving observed to the depth explored. Surface elevation was not measured at the time of exploration.
12.5					
15.0					

● MOISTURE CONTENT %



EXCAVATED BY: Dan J. Fischer Excavating, Inc. LOGGED BY: P. Chaichi COMPLETED: 01/03/22

EXCAVATION METHOD: mini excavator (see document text)

NV5	REMBOLD-15-01	**TEST PIT TP-2**	
	JANUARY 2022	ELMONICA BEAVERTON, OR	**FIGURE A-2**

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE	● MOISTURE CONTENT % (0 50 100)	COMMENTS
0.0		Soft, brown SILT (ML), minor clay and sand, trace organics; moist (12-inch-thick tilled zone, 3-inch-thick root zone).		PP			PP = 0.25 tsf
		Medium stiff, brown SILT (ML), minor clay and sand, trace organics (roots); moist - **FILL.**	1.0				
2.5		Stiff, brown SILT (ML), trace clay and sand; moist.	2.5	PP	☒		Slow groundwater seepage observed at 3.0 feet. PP = 1.5 tsf
5.0							Slow groundwater seepage observed at 5.0 feet.
7.5				PP	☒	●	PP = 1.0 tsf Moderate groundwater seepage observed at 8.0 feet.
10.0					☒		
		Exploration completed at a depth of 10.5 feet.	10.5				No caving observed to the depth explored. Surface elevation was not measured at the time of exploration.
12.5							
15.0							

EXCAVATED BY: Dan J. Fischer Excavating, Inc. LOGGED BY: P. Chaichi COMPLETED: 01/03/22

EXCAVATION METHOD: mini excavator (see document text)

NV5	REMBOLD-15-01	**TEST PIT TP-3**
	JANUARY 2022	ELMONICA BEAVERTON, OR — **FIGURE A-3**

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE	MOISTURE CONTENT % (0 / 50 / 100)	COMMENTS
0.0	Medium stiff, brown SILT (ML), minor sand and clay, trace organics; moist to wet (9-inch-thick tilled zone, 3-inch-thick root zone).					
	light gray, gravelly at 0.5 foot	0.8				
	Dense, black GRAVEL with sand (GP), minor debris (asphalt pieces), trace silt; moist - **FILL.**	1.0				
2.5	Very stiff, dark gray SILT (ML), minor clay and sand, trace gravel; moist - **FILL.**		PP		●	PP = 2.5 tsf
	Stiff to very stiff, brown-yellow SILT (ML), trace sand and clay; moist.	3.0				
5.0						Slow groundwater seepage observed at 5.0 feet.
7.5						
			PP		●	PP = 2.0 tsf
10.0						
	Exploration completed at a depth of 10.5 feet.	10.5				No caving observed to the depth explored. Surface elevation was not measured at the time of exploration.
12.5						
15.0						

EXCAVATED BY: Dan J. Fischer Excavating, Inc. LOGGED BY: P. Chaichi COMPLETED: 01/03/22

EXCAVATION METHOD: mini excavator (see document text)

NV5	REMBOLD-15-01	**TEST PIT TP-4**
	JANUARY 2022	ELMONICA BEAVERTON, OR — **FIGURE A-4**

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	● MOISTURE CONTENT % (0 50 100)	COMMENTS
0.0		Medium stiff, brown SILT (ML), minor sand and clay, trace organics; moist (12-inch-thick tilled zone, 6-inch-thick root zone).					
		ASPHALT CONCRETE (buried layer).	1.0				
		Stiff, brown with yellow mottled SILT (ML), minor sand and clay; moist - **FILL.**	1.5	PP		●	Slow groundwater seepage observed at 2.0 feet. PP = 1.5 tsf
2.5							
		Very stiff, yellow-brown SILT (ML), minor sand, trace clay and gravel; moist.	3.5				
5.0							
7.5							Rapid groundwater seepage observed at 9.0 feet.
10.0				PP		●	PP = 1.5 tsf
		Exploration completed at a depth of 10.5 feet.	10.5				No caving observed to the depth explored. Surface elevation was not measured at the time of exploration.
12.5							
15.0							

EXCAVATED BY: Dan J. Fischer Excavating, Inc.　　LOGGED BY: P. Chaichi　　COMPLETED: 01/03/22

EXCAVATION METHOD: mini excavator (see document text)

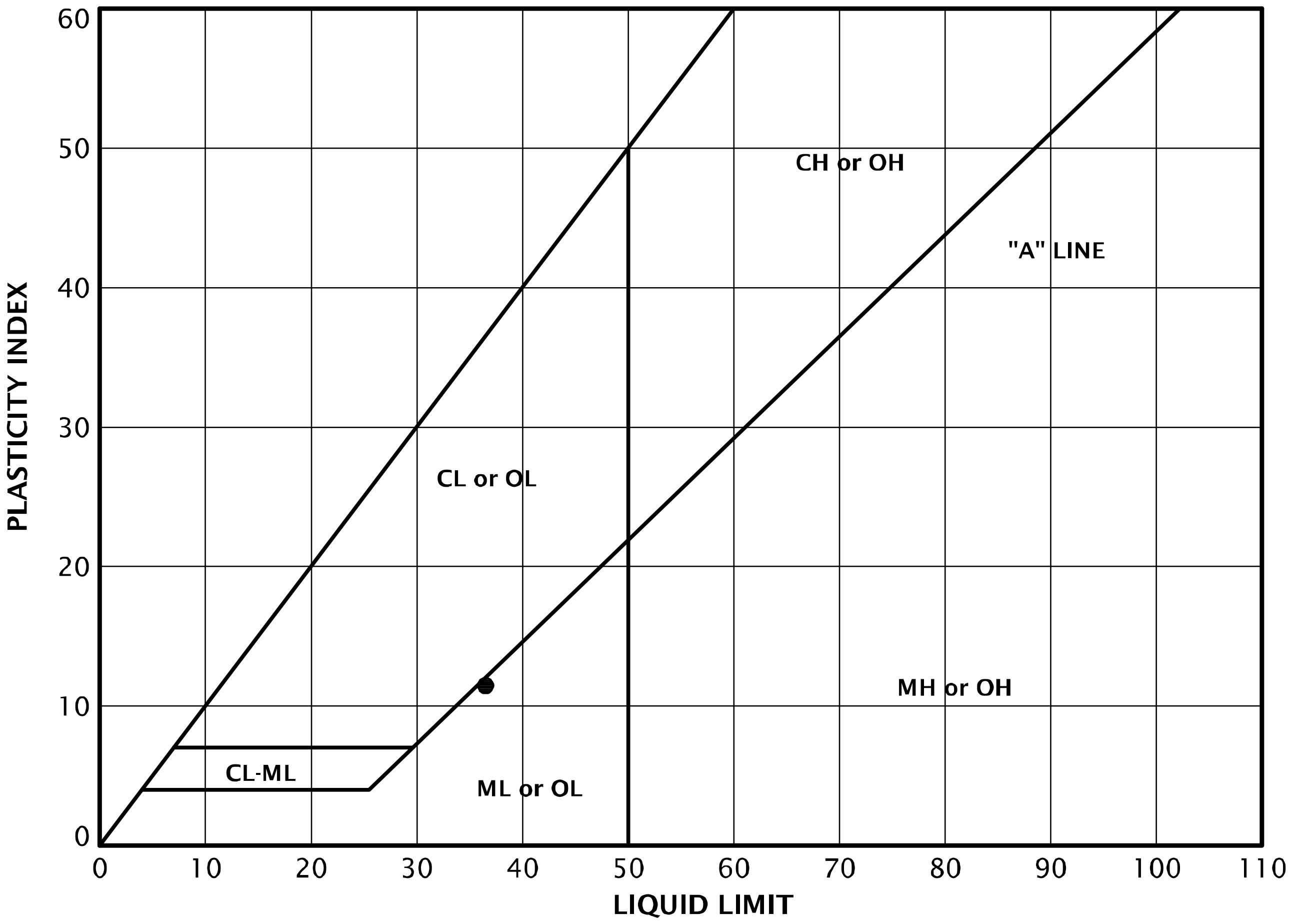
NV5	REMBOLD-15-01	**TEST PIT TP-5**	
	JANUARY 2022	ELMONICA BEAVERTON, OR	**FIGURE A-5**

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE	● MOISTURE CONTENT % (0 50 100)	COMMENTS
0.0		Soft, brown SILT (ML), minor sand and clay, trace organics; moist (6-inch-thick tilled zone, 4-inch-thick root zone).					
		Medium stiff to stiff, brown SILT (ML), minor sand and clay; moist, blocky structure - **FILL**.	0.5		☒		
2.5					☒	●	
		Stiff, yellow-brown SILT (ML), trace sand and clay; moist.	3.0				
				P200	☒	●	Infiltration test at 4.0 feet. P200 = 96%
5.0							
7.5							
		medium stiff to stiff at 8.0 feet		PP	☒		PP = 1.0 tsf
10.0					☒		Slow groundwater seepage observed at 10.0 feet.
		Exploration completed at a depth of 10.5 feet.	10.5				No caving observed to the depth explored. Surface elevation was not measured at the time of exploration.
12.5							
15.0							

EXCAVATED BY: Dan J. Fischer Excavating, Inc. LOGGED BY: P. Chaichi COMPLETED: 01/03/22

EXCAVATION METHOD: mini excavator (see document text)

NV5	REMBOLD-15-01	**TEST PIT TP-6**	
	JANUARY 2022	ELMONICA BEAVERTON, OR	**FIGURE A-6**



KEY	EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	MOISTURE CONTENT (PERCENT)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
●	TP-2	8.0	36	37	25	12

_ATTERBERG_LIMITS 7 REMBOLD-15-01-TP1_6.GPJ GEODESIGN.GDT PRINT DATE: 1/12/22:KT

NV5

REMBOLD-15-01

JANUARY 2022

ATTERBERG LIMITS TEST RESULTS

ELMONICA
BEAVERTON, OR

FIGURE A-7

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
TP-1	3.0		32							
TP-2	3.0		23							
TP-2	4.0		36				96			
TP-2	8.0		36					37	25	12
TP-3	7.0		34							
TP-4	2.0		22							
TP-4	9.0		38							
TP-5	2.0		27							
TP-5	10.0		39							
TP-6	2.0		29							
TP-6	4.0		39				96			

APPENDIX B

APPENDIX B

GRI 2019 BORING LOGS AND LABORATORY RESULTS

GRI drilled two borings (B-1 and B-2) at the site on August 5, 2019. The boring logs and associated laboratory testing results are presented in this appendix. The approximate exploration locations are shown on Figure 2.

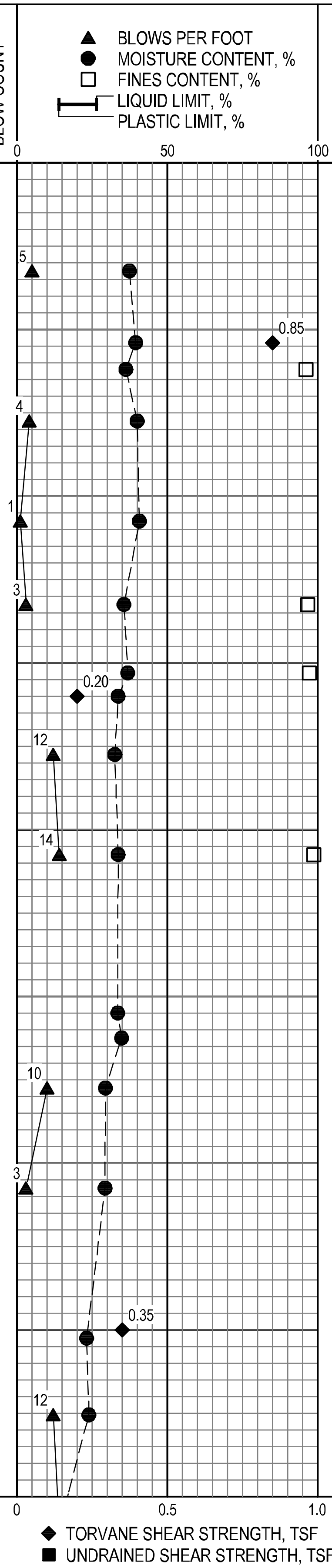
Table 3A

SUMMARY OF LABORATORY RESULTS

Sample Information						Atterberg Limits			
Location	Sample	Depth, ft	Elevation, ft	Moisture Content, %	Dry Unit Weight, pcf	Liquid Limit, %	Plasticity Index, %	Fines Content, %	Soil Type
B-1	S-1	2.5	--	37	--	--	--	--	SILT
	S-2	5.4	--	39	82	--	--	--	SILT
	S-2	6.2	--	36	--	--	--	96	SILT
	S-3	7.0	--	40	--	--	--	--	SILT
	S-4	10.0	--	41	--	--	--	--	SILT
	S-5	12.5	--	36	--	--	--	97	SILT
	S-6	15.3	--	37	--	--	--	97	SILT
	S-6	16.0	--	34	86	--	--	--	SILT
	S-7	17.0	--	32	--	--	--	--	SILT
	S-8	20.0	--	34	--	--	--	99	SILT
	S-9	25.5	--	33	89	--	--	--	SILT
	S-9	26.0	--	35	--	--	--	--	SILT
	S-10	27.0	--	29	--	--	--	--	SILT
	S-11	30.0	--	29	--	--	--	--	SILT
	S-12	35.0	--	23	105	--	--	--	Clayey SILT
	S-13	36.8	--	24	--	--	--	--	Clayey SILT
	S-14	40.0	--	15	--	--	--	--	Silty CLAY
	S-15	45.0	--	25	--	--	--	--	Silty CLAY
	S-16	50.0	--	23	--	--	--	--	Silty CLAY
B-2	S-1	2.5	--	23	--	--	--	--	SILT
	S-2	5.0	--	33	--	--	--	--	SILT
	S-3	7.5	--	40	--	--	--	--	SILT
	S-4	10.5	--	38	84	--	--	--	SILT
	S-4	11.3	--	38	--	--	--	--	SILT
	S-5	12.0	--	41	--	--	--	--	SILT
	S-6	15.0	--	38	--	--	--	--	SILT
	S-7	20.5	--	33	92	--	--	--	SILT
	S-7	21.0	--	32	--	--	--	--	SILT
	S-8	22.0	--	32	--	--	--	98	SILT
	S-9	25.0	--	35	--	--	--	98	SILT
	S-10	30.2	--	35	--	--	--	--	SILT
	S-10	30.8	--	32	80	--	--	--	SILT
	S-11	32.0	--	33	--	--	--	--	SILT
	S-12	35.0	--	30	--	--	--	--	SILT
	S-13	40.0	--	22	--	--	--	--	Silty CLAY
	S-14	45.0	--	23	--	--	--	--	Silty CLAY
	S-15	50.0	--	25	--	--	--	--	Silty CLAY
	S-16	55.0	--	24	--	--	--	--	Silty CLAY
	S-17	60.0	--	25	--	--	--	--	Silty CLAY
	S-18	65.0	--	21	--	--	--	--	Silty CLAY

BORING B-1

DEPTH, FT	CLASSIFICATION OF MATERIAL	DEPTH, FT	SAMPLE NO.	BLOW COUNT	COMMENTS AND ADDITIONAL TESTS
	Surface Elevation: Not Available				
0	Asphalt concrete PAVEMENT (1.5 in.) over crushed rock BASE COURSE (2 in.)	0.3			
	SILT, trace clay and fine-grained sand, light brown, medium stiff (Willamette Silt)		S-1	2 3 2	
5	---stiff at 5 ft		S-2		Dry Density = 82 pcf
	---trace to some sand, soft to medium stiff, contains woody debris at 7 ft		S-3	1 1 3	
10	---very soft at 10 ft		S-4	0 0 1	
	---soft below 12.5 ft		S-5	1 1 2	
15			S-6		Dry Density = 86 pcf
	---stiff below 17 ft		S-7	1 5 7	
20	---gray below 21.5 ft		S-8	2 6 8	Sample S-8 includes gray material at tip of sampler
25			S-9		Dry Density = 89 pcf
	---some sand and clay below 27 ft		S-10	2 4 6	
30	---some clay to clayey, trace sand, soft below 30 ft		S-11	0 0 3	
35	Clayey SILT, trace fine-grained sand, gray mottled brown, medium stiff (Hillsboro Formation) ---stiff below 36.8 ft	35.0	S-12		Dry Density = 105 pcf
			S-13	3 6 6	
40					



(CONTINUED NEXT PAGE)

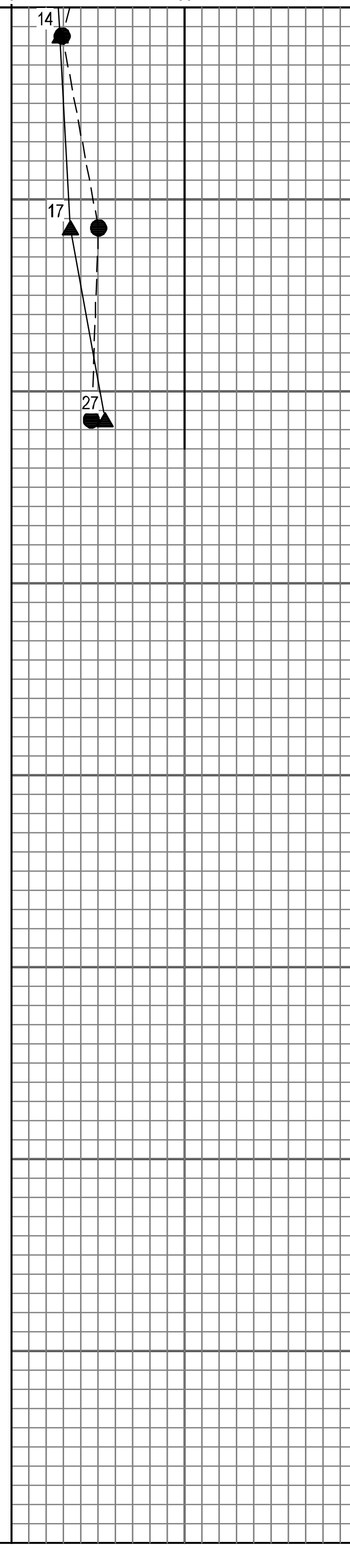
Logged By: N. Utevsky	**Drilled by:** Western States Soil Conservation, Inc.
Date Started: 8/5/19	**GPS Coordinates:** 45.51192° N -122.8526° W (WGS 84)
Drilling Method: Mud Rotary **Equipment:** CME 55 HT Track-Mounted Drill Rig **Hole Diameter:** 5 in.	**Hammer Type:** Auto Hammer **Weight:** 140 lb **Drop:** 30 in. **Energy Ratio:**
Note: See Legend for Explanation of Symbols	

GRI

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL **Surface Elevation:** Not Available	DEPTH, FT	INSTALLATION	SAMPLE NO.	SAMPLE TYPE	BLOW COUNT	COMMENTS AND ADDITIONAL TESTS
		Silty CLAY to clayey SILT, up to trace fine- to coarse-grained sand, light gray mottled rust, stiff (Hillsboro Formation)			S-14		4 6 8	
45		---very stiff below 45 ft			S-15		4 7 10	
50					S-16		7 12 15	
		(8/5/2019)	51.5					
55								
60								
65								
70								
75								
80								

▲ BLOWS PER FOOT
● MOISTURE CONTENT, %
□ FINES CONTENT, %
LIQUID LIMIT, %
PLASTIC LIMIT, %

0 50 100



0 0.5 1.0

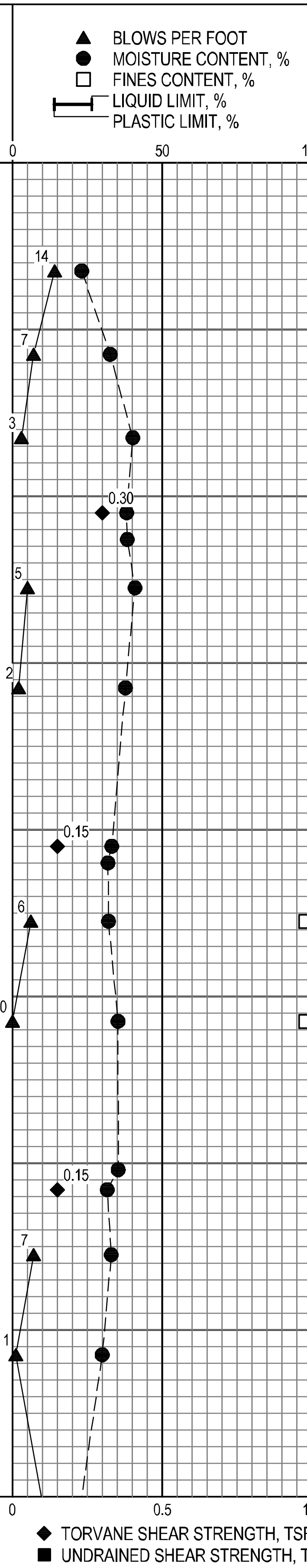
◆ TORVANE SHEAR STRENGTH, TSF
■ UNDRAINED SHEAR STRENGTH, TSF

GRI BORING LOG (GPS) - NO ELEV GRI DATA TEMPLATE.GDT 1/20/20

GRI

BORING B-1

DEPTH, FT	CLASSIFICATION OF MATERIAL	DEPTH, FT	SAMPLE NO.	BLOW COUNT	COMMENTS AND ADDITIONAL TESTS
	Surface Elevation: Not Available				
0	SILT, some clay to clayey, trace fine- to coarse-grained sand, brown mottled rust, stiff, contains roots, grass at ground surface (Possible Fill)		S-1	5 7 7	
5	SILT, trace clay and fine-grained sand, brown, medium stiff (Willamette Silt)	5.0	S-2	3 3 4	
	---soft, contains woody debris at 7.5 ft		S-3	2 1 2	
10			S-4		Dry Density = 84 pcf
			S-5	2 2 3	
15	---very soft to soft below 15 ft		S-6	0 1 1	
20			S-7		Dry Density = 92 pcf
	---medium stiff at 22 ft, clay not observed, gray below 22 ft		S-8	2 3 3	
25	---very soft at 25 ft		S-9	1 0 0	
30			S-10		Dry Density = 80 pcf
	---trace to some sand, trace clay, medium stiff below 32 ft		S-11	2 3 4	
35	---very soft below 35 ft		S-12	0 0 1	
40					



(CONTINUED NEXT PAGE)

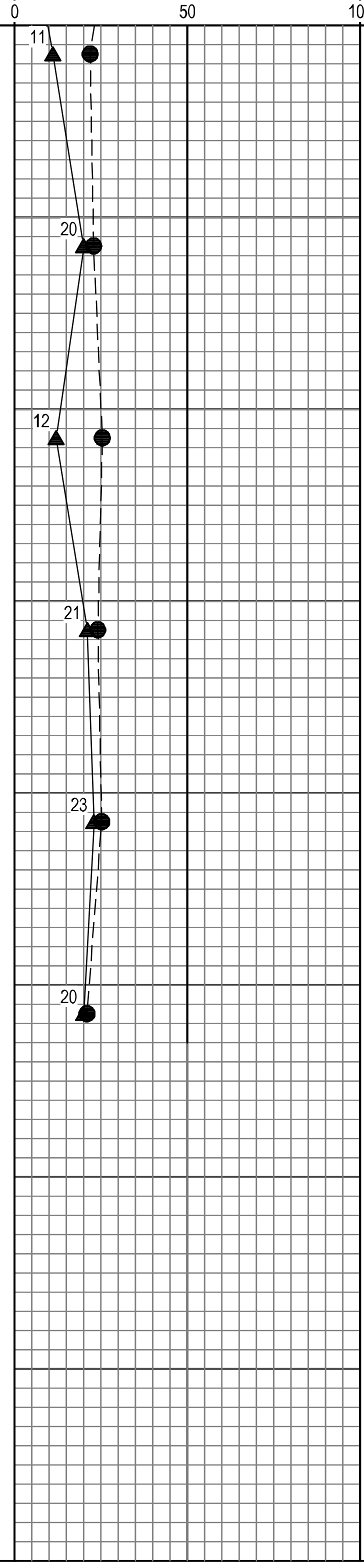
Logged By: N. Utevsky	**Drilled by:** Western States Soil Conservation, Inc.
Date Started: 8/5/19	**GPS Coordinates:** 45.51081° N -122.85253° W (WGS 84)
Drilling Method: Mud Rotary **Equipment:** CME 55 HT Track-Mounted Drill Rig **Hole Diameter:** 5 in.	**Hammer Type:** Auto Hammer **Weight:** 140 lb **Drop:** 30 in. **Energy Ratio:**
Note: See Legend for Explanation of Symbols	

GRI

BORING B-2

DEPTH, FT	GRAPHIC LOG	CLASSIFICATION OF MATERIAL **Surface Elevation:** Not Available	DEPTH, FT	INSTALLATION	SAMPLE NO.	SAMPLE TYPE	BLOW COUNT	COMMENTS AND ADDITIONAL TESTS
		Silty CLAY to clayey SILT, up to trace fine- to coarse-grained sand, gray mottled rust, stiff (Hillsboro Formation)			S-13		3 6 5	
45		---light gray mottled rust, very stiff below 45 ft			S-14		6 8 12	
50		---light brown mottled gray, stiff at 50 ft			S-15		4 5 7	
55					S-16		6 10 11	
60					S-17		5 8 15	
65					S-18		5 10 10	
		(8/5/2019)	66.5					
70								
75								
80								

▲ BLOWS PER FOOT
● MOISTURE CONTENT, %
□ FINES CONTENT, %
LIQUID LIMIT, %
PLASTIC LIMIT, %

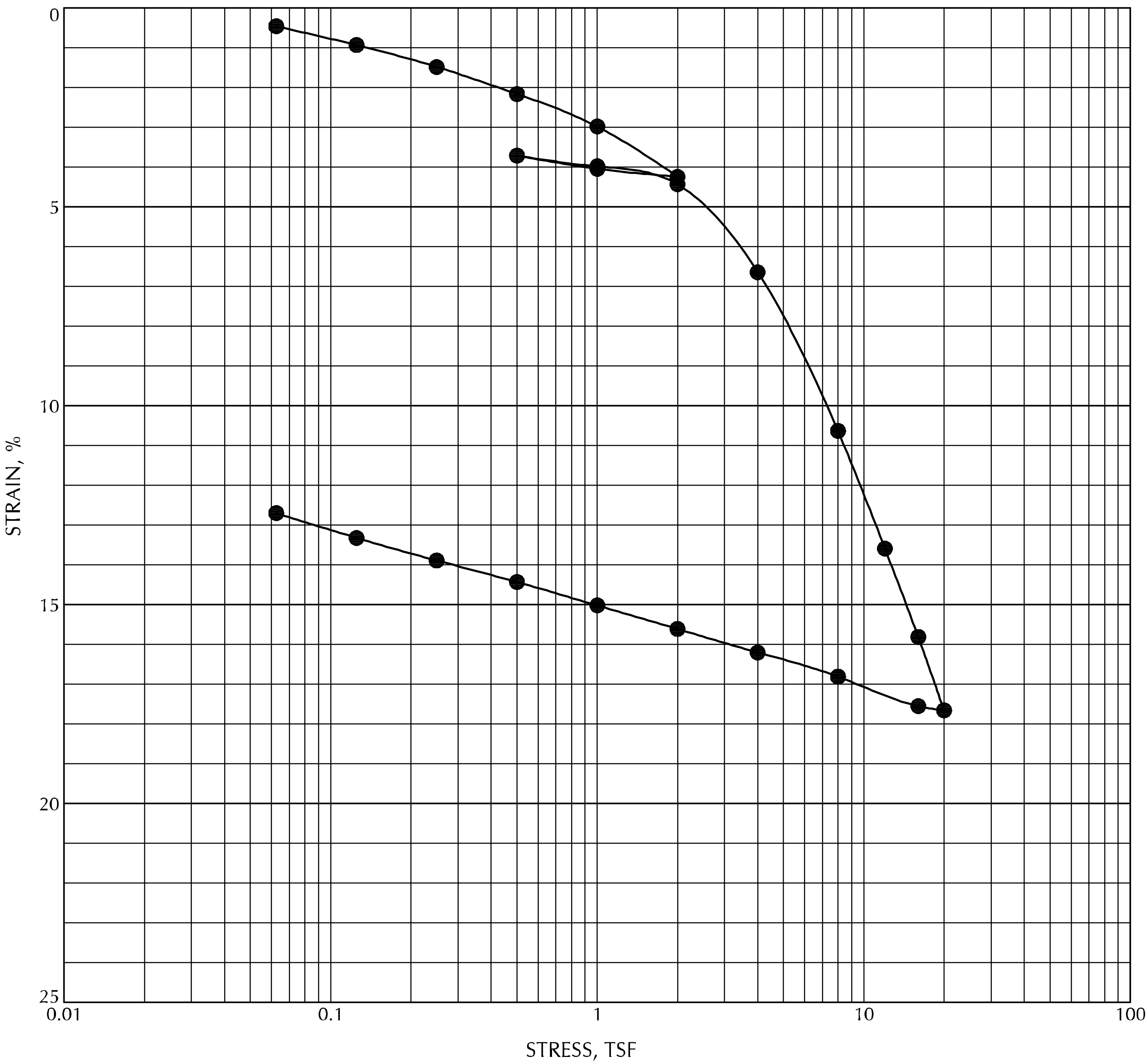


◆ TORVANE SHEAR STRENGTH, TSF
■ UNDRAINED SHEAR STRENGTH, TSF

GRI

BORING B-2

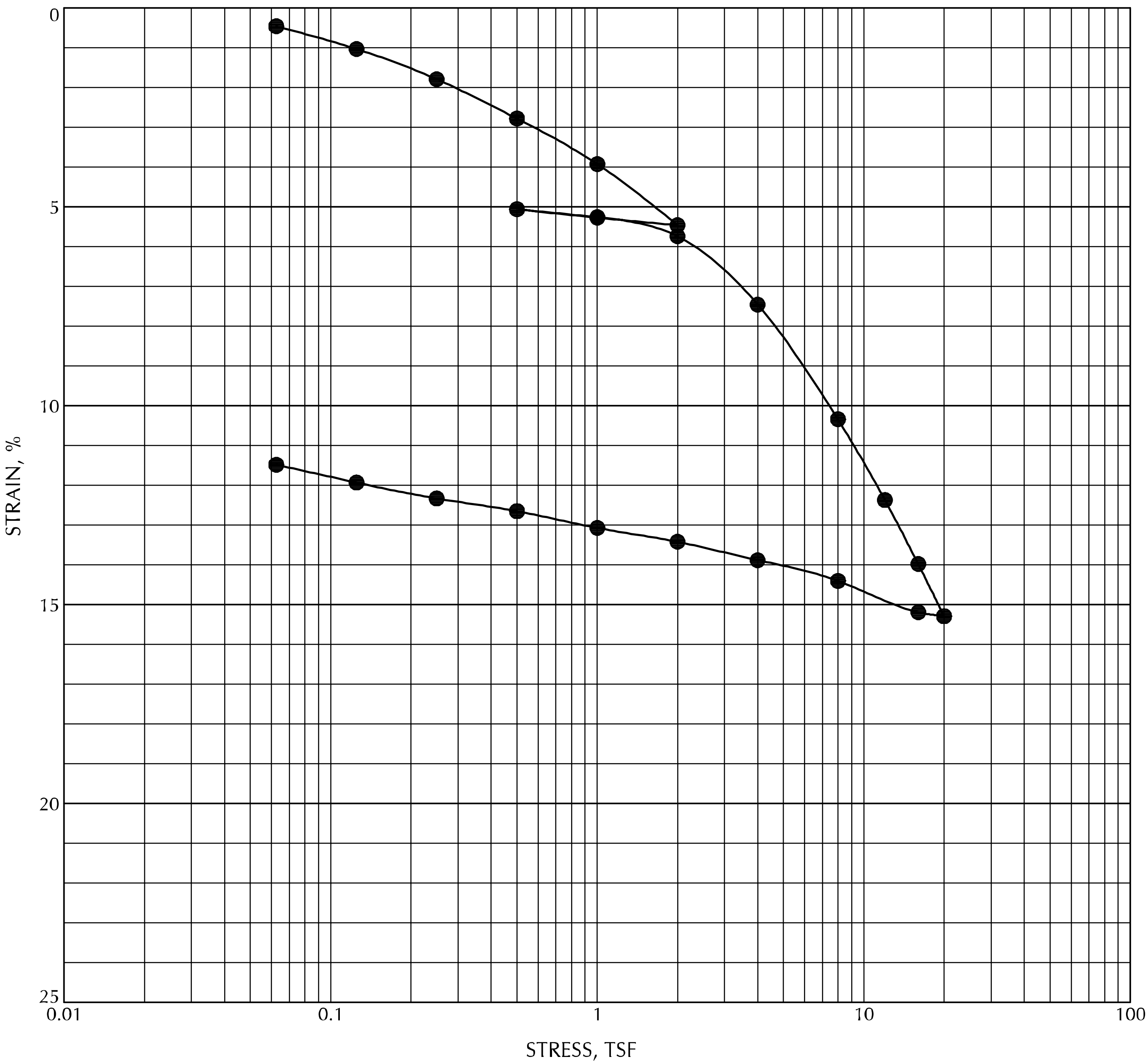
FEB. 2020 JOB NO. 6270 FIG. 2A



	Location	Sample	Depth, ft	Classification	Initial γ_d, pcf	Initial MC, %
●	B-1	S-2	6.1	SILT, trace clay and fine-grained sand, light brown, stiff (Willamette Silt)	86	37

GRI

CONSOLIDATION TEST



	Location	Sample	Depth, ft	Classification	Initial γ_d, pcf	Initial MC, %
●	B-1	S-6	15.3	SILT, trace clay and fine-grained sand, light brown, soft (Willamette Silt)	82	37

GRI

CONSOLIDATION TEST

APPENDIX C

APPENDIX C

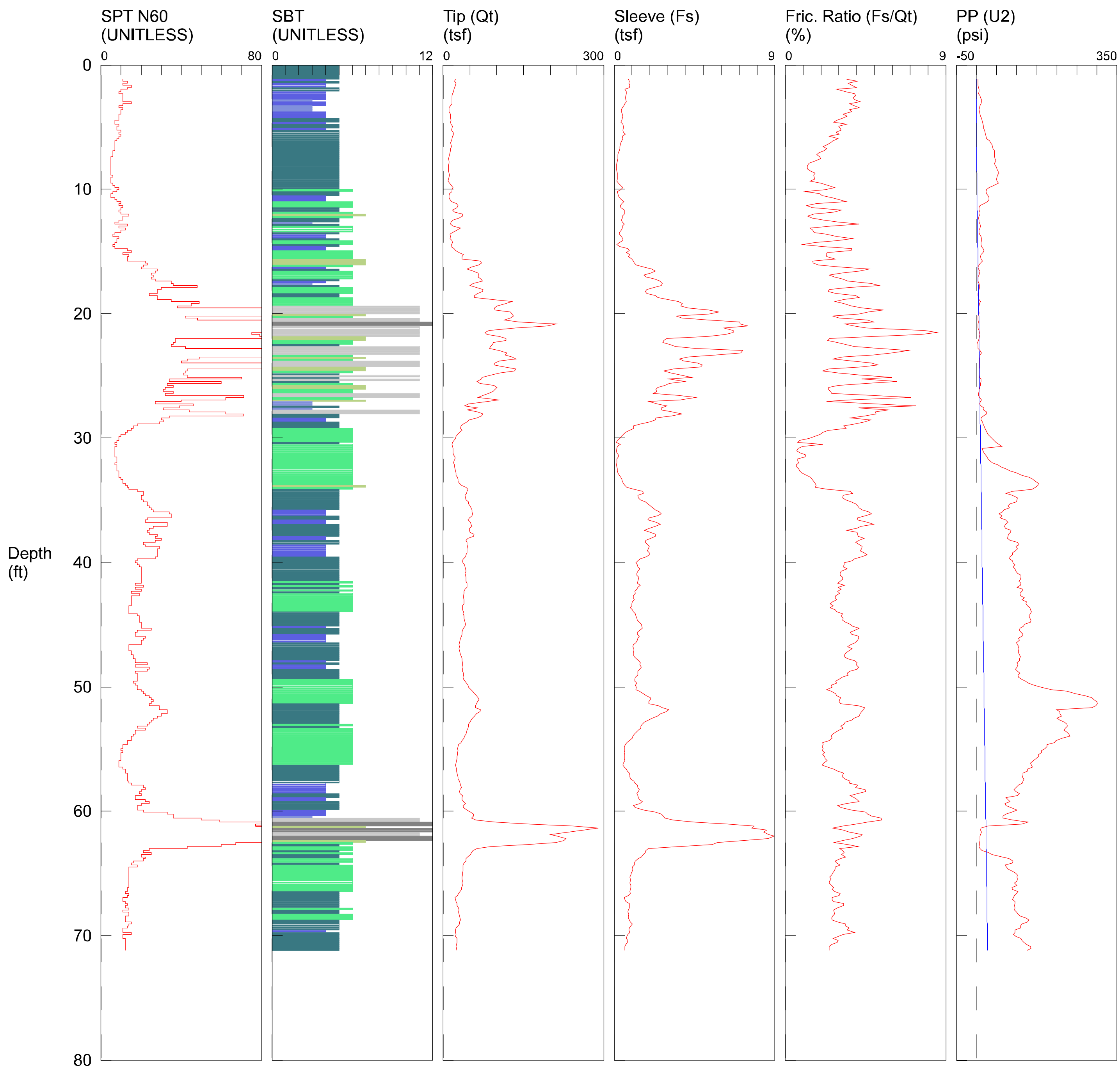
GRI 2019 CPT PROBE EXPLORATIONS

Subsurface conditions were previously explored at the site by advancing four CPT probes (CPT-1 through CPT-4) to depths between approximately 47.6 and 71.5 feet BGS. The CPT probes were performed in general accordance with ASTM D5778 by Oregon Geotechnical Explorations of Keizer, Oregon, on June 12 and November 26, 2019. The results of the CPT probes completed for this project are presented in this appendix and the approximate exploration locations are shown on Figure 2.

The CPT is an in-situ test that provides characterizes subsurface stratigraphy. The testing includes advancing a 35.6-millimeter-diameter cone equipped with a load cell and a friction sleeve through the soil profile. The cone is advanced at a rate of approximately 2 centimeters per second. Tip resistance, sleeve friction, and pore pressure at are typically recorded at 0.1-meter intervals. At select depths, the CPT advancement can be suspended and pore water dissipation rates measured. Shear wave velocity of the subsurface soil was also measured at 1-meter intervals in CPT-4.

GRI / CPT-1 / 17160 SW Baseline Rd Beaverton

OPERATOR: OGE BAK
CONE ID: DPG1386
HOLE NUMBER: CPT-1
TEST DATE: 6/12/2019 9:07:36 AM
TOTAL DEPTH: 71.194 ft

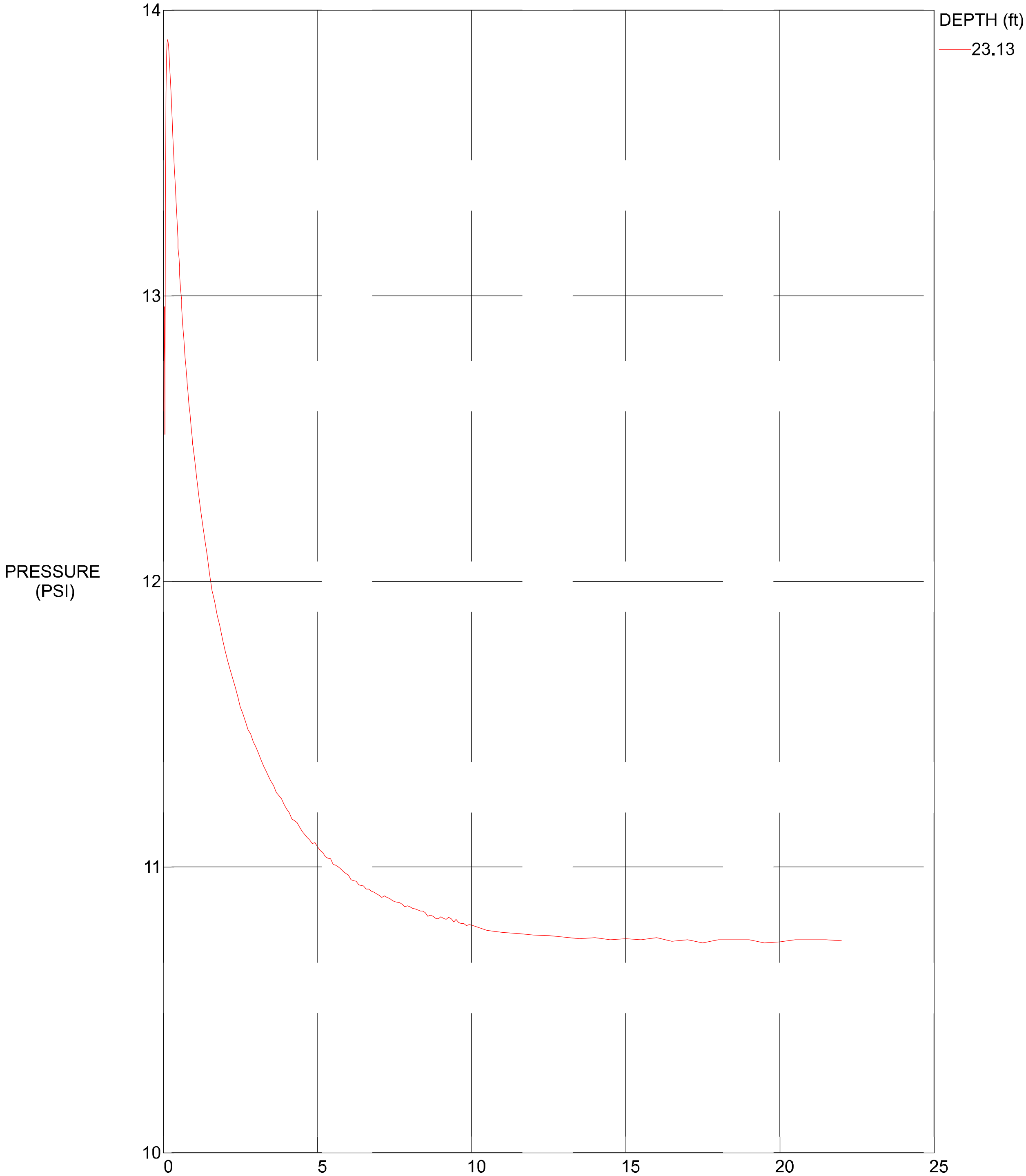


1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

*SBT/SPT CORRELATION: UBC-1983

COMMENT: GRI / CPT-1 / 17160 SW Baseline Rd Beaverton

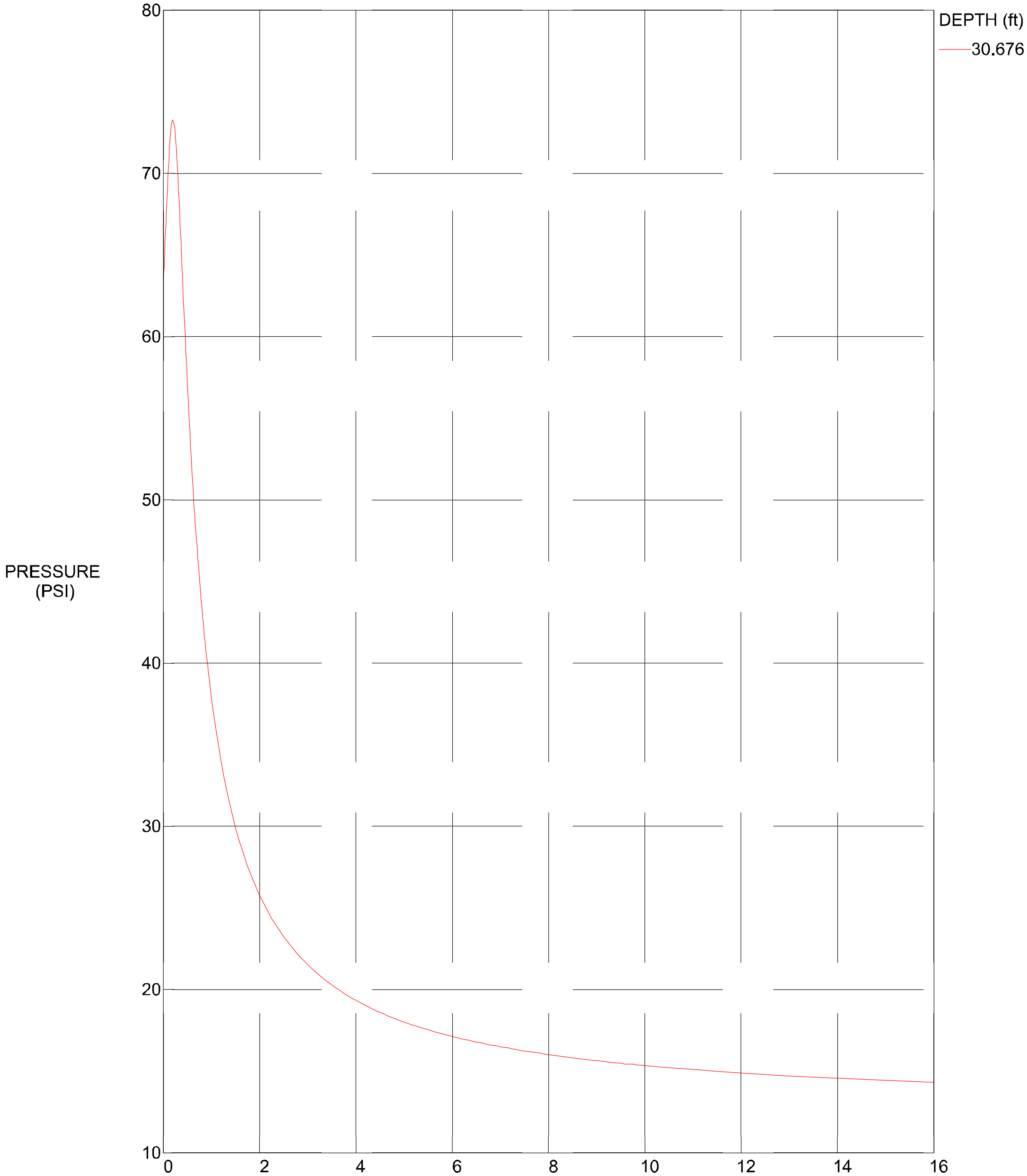
TEST DATE: 6/12/2019 9:07:36 AM



MAXIMUM PRESSURE = 13.895 (PSI) TIME: (MINUTES)
HYDROSTATIC PRESSURE = 6.996 (PSI), WATER TABLE: 6.99 ft
OPERATOR: OGE BAK

COMMENT: GRI / CPT-1 / 17160 SW Baseline Rd Beaverton

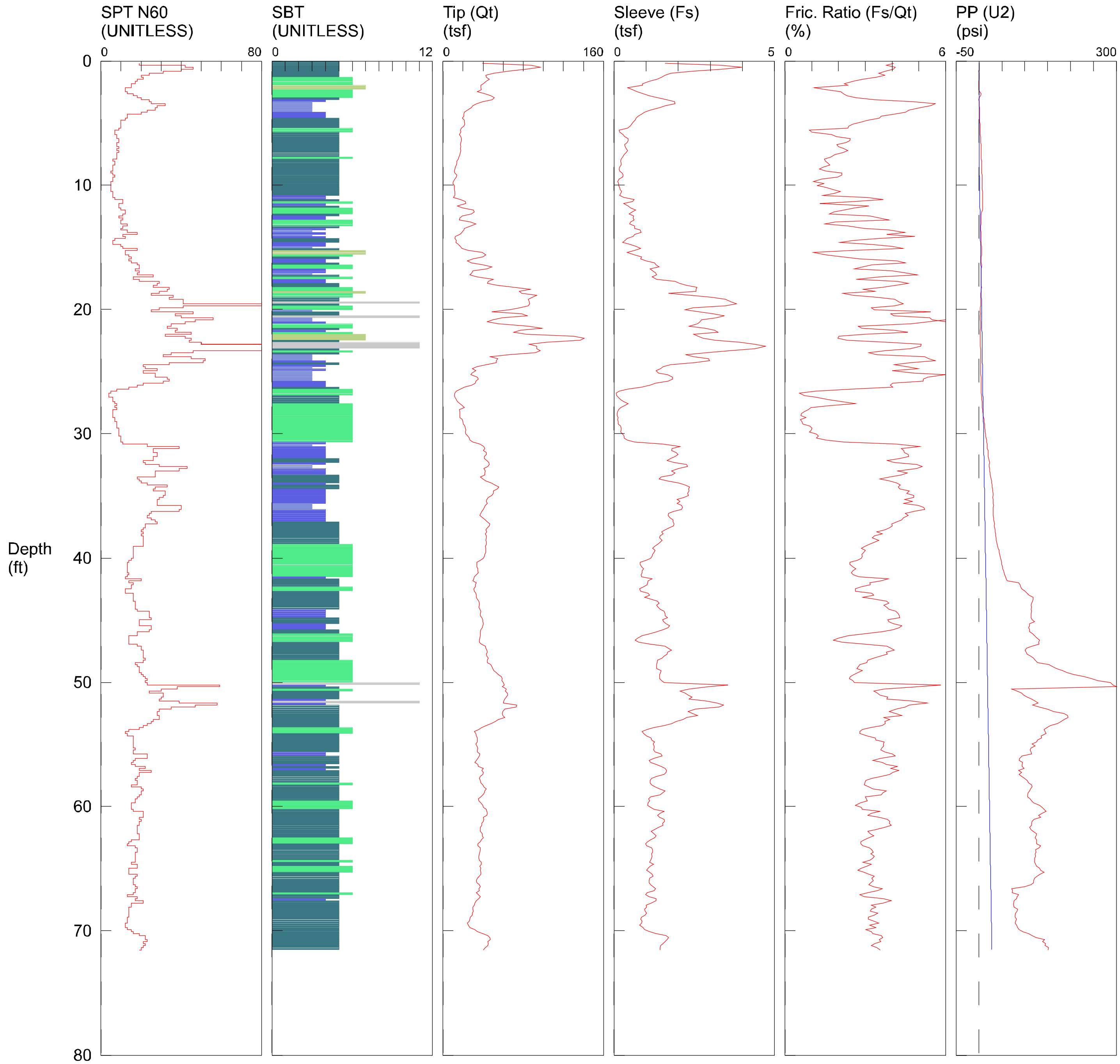
TEST DATE: 6/12/2019 9:07:36 AM



MAXIMUM PRESSURE = 73.268 (PSI) TIME: (MINUTES)
HYDROSTATIC PRESSURE = 10.266 (PSI), WATER TABLE: 6.99 ft
OPERATOR: OGE BAK

GRI / CPT-2 / 17160 SW Baseline Rd Beaverton

OPERATOR: OGE BAK
CONE ID: DPG1386
HOLE NUMBER: CPT-2
TEST DATE: 6/12/2019 10:51:22 AM
TOTAL DEPTH: 71.522 ft

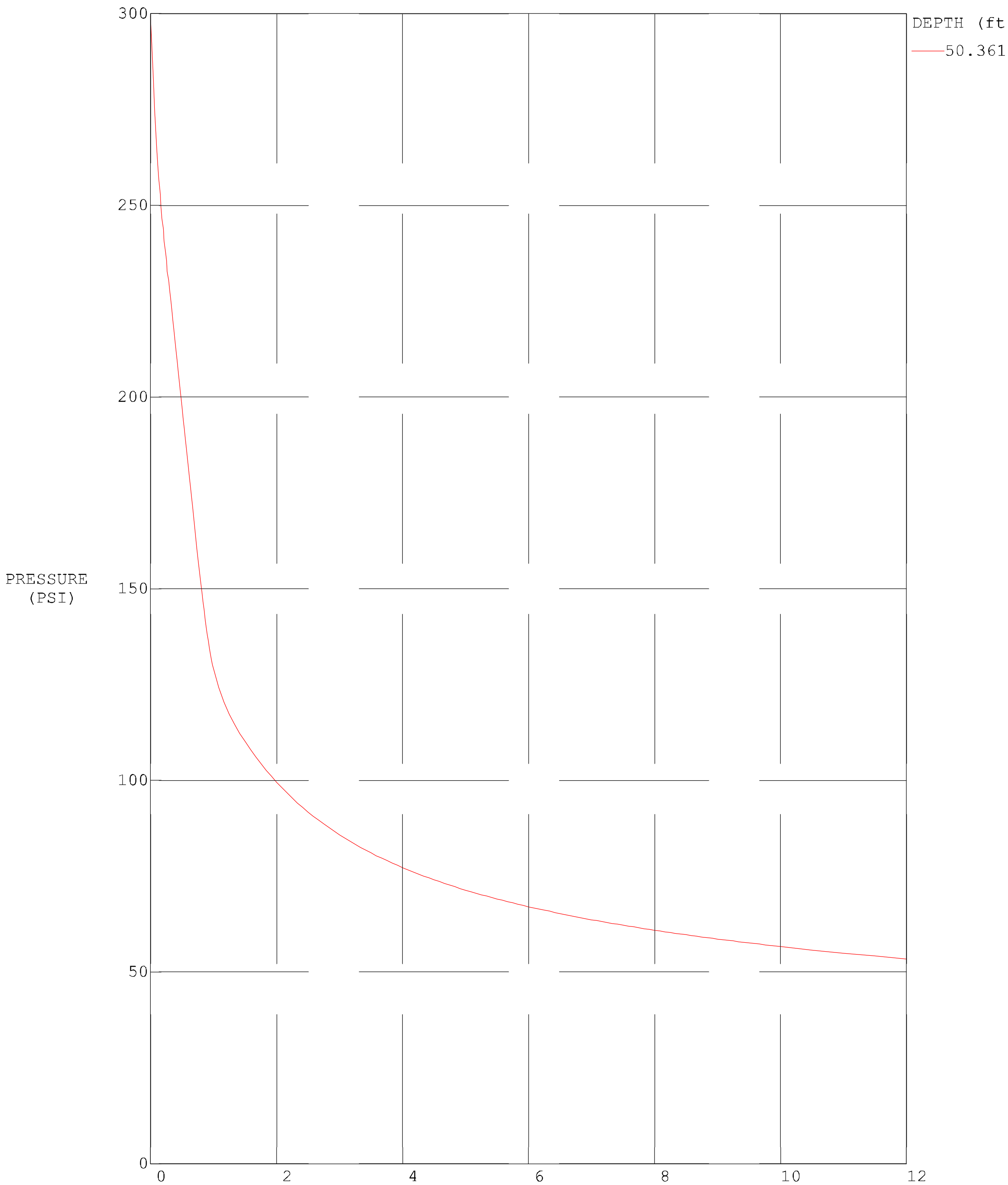


1	sensitive fine grained	4	silty clay to clay	7	silty sand to sandy silt	10	gravelly sand to sand
2	organic material	5	clayey silt to silty clay	8	sand to silty sand	11	very stiff fine grained (*)
3	clay	6	sandy silt to clayey silt	9	sand	12	sand to clayey sand (*)

*SBT/SPT CORRELATION: UBC-1983

COMMENT: GRI / CPT-2 / 17160 SW Baseline Rd Beaverton

TEST DATE: 6/12/2019 10:51:22 AM



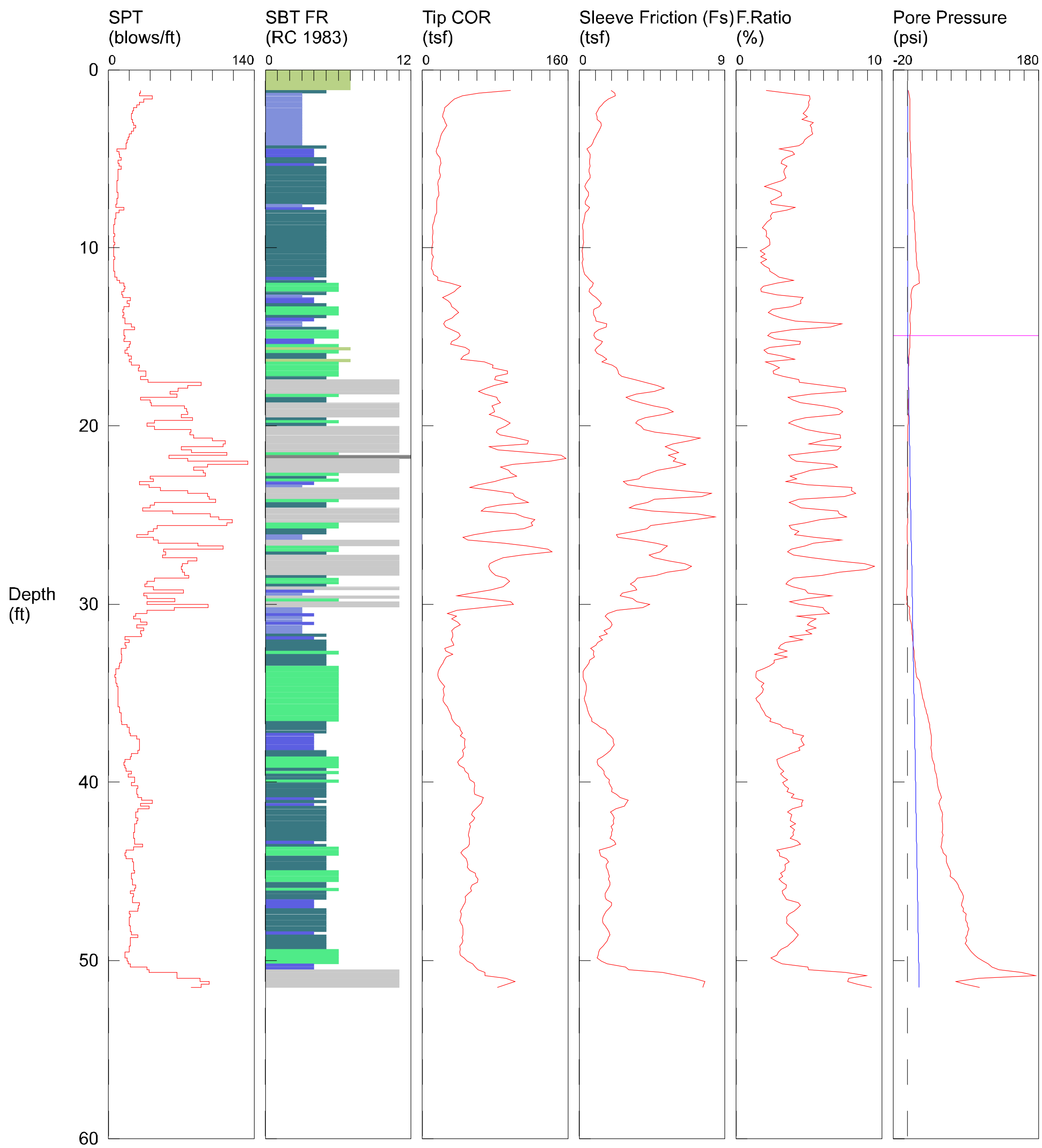
MAXIMUM PRESSURE = 299.424 (PSI)E: (MINUTES)

HYDROSTATIC PRESSURE = 18.797 (PSI), WATER TABLE: 6.99 ft

OPERATOR: OGE BAK

GRI / CPT-3 / 16791 W Baseline Rd Beaverton

OPERATOR: OGE DMM
CONE ID: DSG0707
HOLE NUMBER: CPT-3
TEST DATE: 11/26/2019 9:37:46 AM
TOTAL DEPTH: 51.509 ft

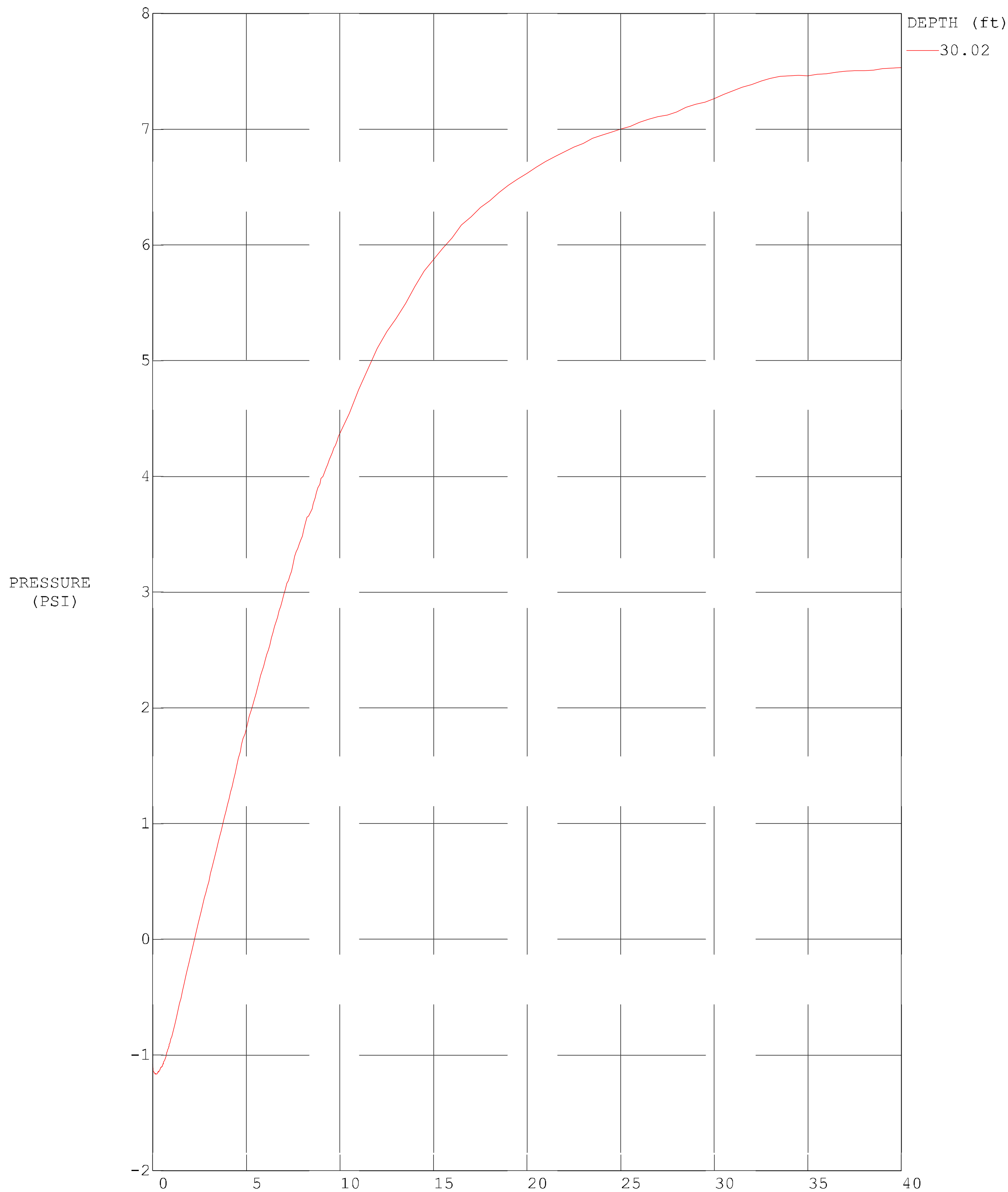


1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

*SBT/SPT CORRELATION: UBC-1983

COMMENT: GRI / CPT-3 / 16791 W Baseline Rd Beaverton

TEST DATE: 11/26/2019 9:37:46 AM

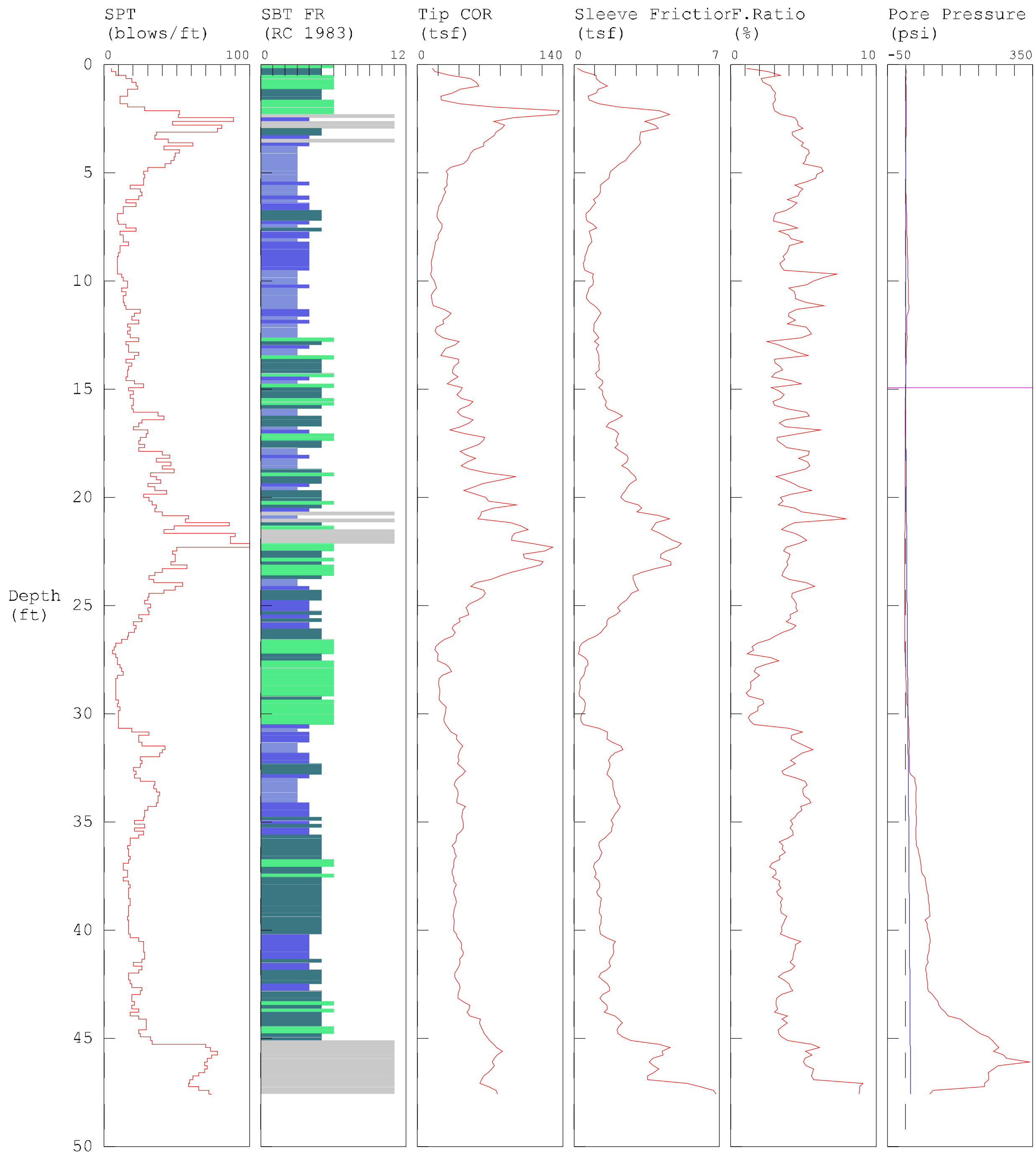


MAXIMUM PRESSURE = 7.534 (PSI)

HYDROSTATIC PRESSURE = 6.541 (PSI), WATER TABLE: 14.93 ft

GRI / CPT-4 / 16791 W Baseline Rd Beaverton

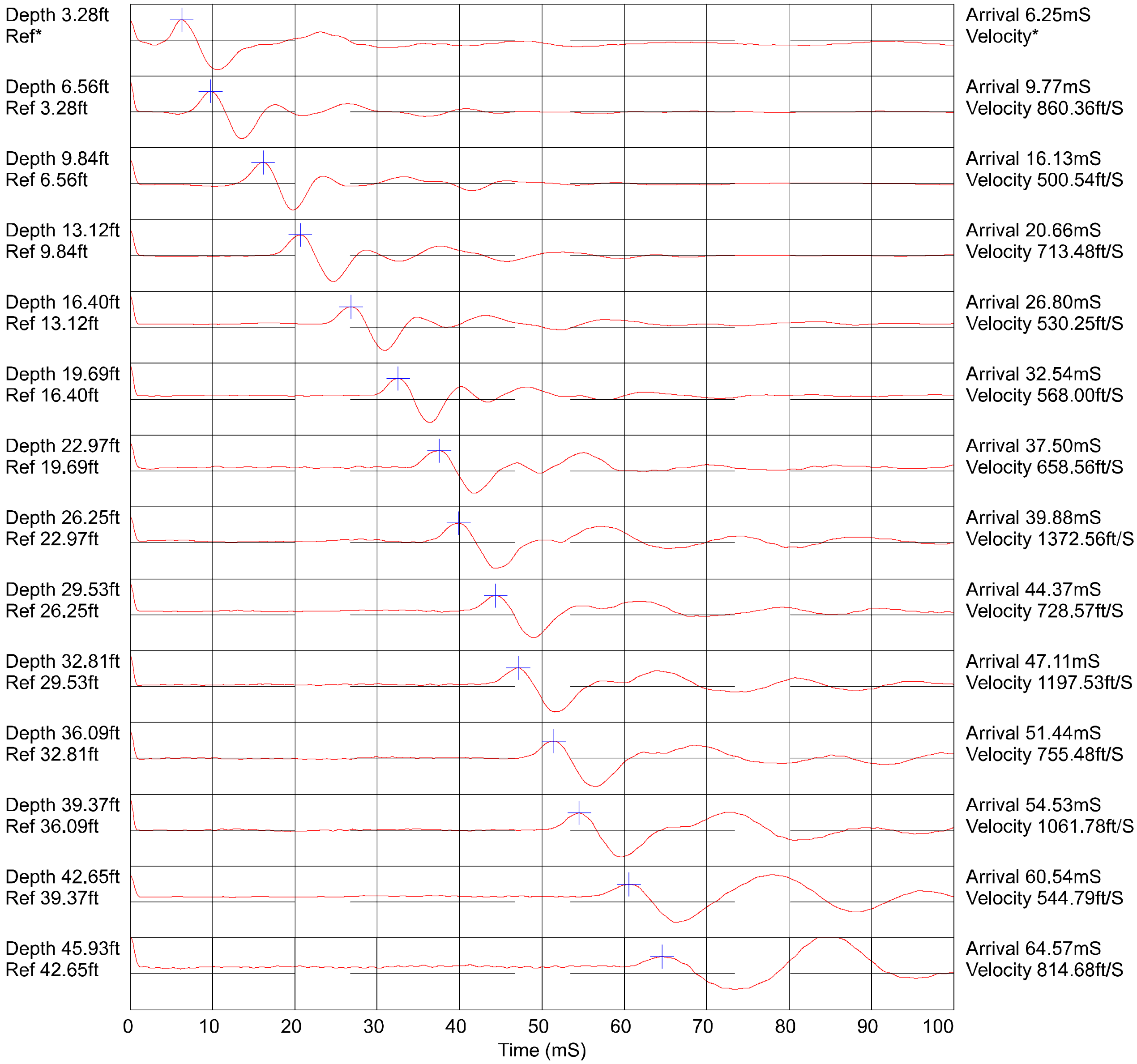
OPERATOR: OGE DMM
CONE ID: DSG0707
HOLE NUMBER: CPT-4
TEST DATE: 11/26/2019 11:26:21 AM
TOTAL DEPTH: 47.572 ft



1	sensitive fine gra	4	silty clay to cl	7	silty sand to sandy	10	gravelly sand to sand
2	organic materia	5	clayey silt to silt	8	sand to silty sa	11	very stiff fine grained (*)
3	clay	6	sandy silt to claye	9	sand	12	sand to clayey sand (*)

*SBT/SPT CORRELATION: UBC-1983

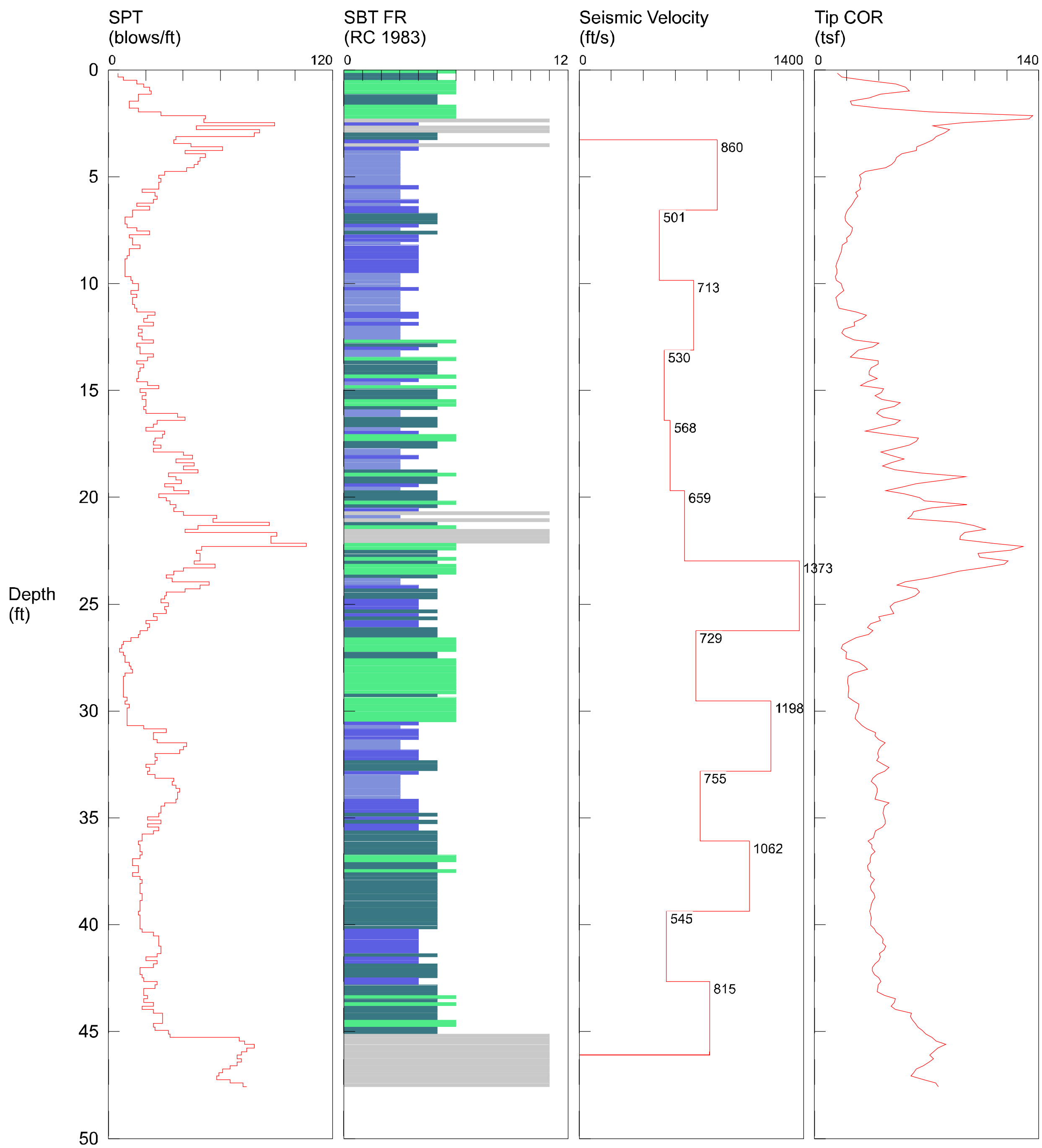
COMMENT: GRI / CPT-4 / 16791 W Baseline Rd Beaverton



Hammer to Rod String Distance (ft): 1.97
* = Not Determined

GRI / CPT-4 / 16791 W Baseline Rd Beaverton

OPERATOR: OGE DMM
CONE ID: DSG0707
HOLE NUMBER: CPT-4
TEST DATE: 11/26/2019 11:26:21 AM
TOTAL DEPTH: 47.572 ft



1	sensitive fine grained	4	silty clay to clay	7	silty sand to sandy silt	10	gravelly sand to sand
2	organic material	5	clayey silt to silty clay	8	sand to silty sand	11	very stiff fine grained (*)
3	clay	6	sandy silt to clayey silt	9	sand	12	sand to clayey sand (*)

*SBT/SPT CORRELATION: UBC-1983

Delivering Solutions
Improving Lives

ATTACHMENT G

MEMO

JTE
Janet Turner Engineering, LLC

To:	City of Beaverton
From:	Janet L. Turner, P.E.
Date:	April 10, 2023
Subject:	Land Use Review (DR2022-0139/TP2022-0015/LD2022-0018/LLD2023-0003): Stormwater Report Amendment, public right-of-way (offsite)

The proposed development project includes an amount of public right-of-way (ROW) dedication with both curb/sidewalk and utility improvements along the two frontages, which include SW Baseline Road to the north and SW 170th Avenue to the east. Though both frontages within Washington County governance, the design/configuration of the improvements and amount of dedication are also as per city of Beaverton standards.

Originally proposed as part of the public ROW improvements were vegetative stormwater management planters along the 170th Avenue frontage to meet Clean Water Services' (CWS) standards, specifically Chapter 4 criteria for all new and/or redeveloped impervious surface areas. Per the first land use review, Washington County does not typically permit LIDA facilities in the planter strip of arterial streets, thus the planters were removed from the design, leaving in-place relocated public storm inlets plumbed directly to the existing storm main with SW 170th Avenue.

Per second round land use review comments, we've learned that the design, though it can't include LIDA facilities, still needs to meet CWS standards in some manner, or proposed is the implementation of a fee-in-lieu option, requiring written justification for this approach. The following is the design team's justification for fee-in-lieu, including area take-offs of unmanaged new and modified impervious areas within each ROW fronting the development site, to assist in establishing an estimate of the cost associated with the proposed fee-in-lieu approach to meeting CWS standards.

Justification for fee-in-lieu, as proposed herein, must follow the "Order of Precedence" as listed in Table 530.1 in the Beaverton Engineering Design Manual (EDM). Per the table, the following is a list of the Order, including our response to each:

- 1st Order: Enhancement and/or Expansion of an Existing public stormwater management (SWM) facility.
    - Meeting this order is not feasible as there is no existing public facility between the project's frontage and the stormwater outfall location, beyond the Trimet ROW and tracks.
- 2nd Order: New Public Vegetated SWM facility, CWS Design & Construction standards, Section 4.06.2-4.

  - Not feasible due to the County's request not to include LIDA in the ROW improvements, including proximity of the ROW to the existing Trimet property and rail crossing.
- 3rd Order: Private Vegetated SWM facility:
  - This is not feasible as the development site has limited space, due to numerous proposed trees, parking, pedestrian improvements, and necessary utilities, all required to meet other code requirements.
- 4th Order: Street-side LIDA Swale/Planter in the public ROW:
  - Not feasible due to the County's request not to include LIDA in the ROW improvements, including proximity of the ROW to the existing Trimet property and rail crossing.
- 4th Order: Private Proprietary Treatment Facility:
  - This is not feasible as the development site has limited space, due to numerous proposed trees, parking, pedestrian improvements, and necessary utilities, all required to meet other code requirements.
- 5th Order: Public Proprietary Treatment Facility:
  - Not feasible due to the County's request not to include mechanical treatment within the ROW improvement.
- 6th Order: Fee-in-Lieu:
  - To determine the fee-in-lieu total, the design team provided an area take-off of the new/redeveloped impervious areas, within the project's half of each adjacent ROW. These offsite areas are then applied to the following three criteria, which are then utilized to prepare an estimated fee-in-lieu total.
    - Stormwater Quantity
    - Stormwater Hydromodification
    - Stormwater Quality
  - As the project does not propose to follow any of the above 3 noted design considerations for runoff in either frontage, the new/redeveloped area calculated for each frontage is consistently the same for all three criteria.
    - SW Baseline new/redeveloped area total is: 12,310 square feet.
    - SW 170th Avenue new/redeveloped area total is: 24,680 square feet.
    - Total area: 36,990 square feet.
  - This information was shared with Beaverton Engineering staff (see attached form), who shared the following fee-in-lieu totals:
    - Combined stormwater quality and hydromodification, at $1.50/square foot * 36,990 square feet= $55,485.00.
    - Stormwater quantity, calculated as (36,990 square feet / 2640) * 291 = $4,077.31.
    - **Total fee-in-lieu: $59,562.31.**

City of Beaverton
12725 SW Millikan Way, 4th Floor, PO Box 4755, Beaverton, OR 97076
(503) 350-4021

Stormwater Management Worksheet

Site Development Division
sitedevelopmentplansubmit@beavertonoregon.gov

Fee-in-Lieu

Date Submitted: 4-10-2023
Designed per EDM version: January 1, 2019
Designed per CWS version: R&O 19-22

This form replaces the Certified Impervious Surface Area Inventory and Water Quality Facility Information Sheet.
(This does not replace the development stormwater report)

Project Name: Elmonica Mixed Use - PUBLIC right-of-way improvements

Project Disturbed Area per Site Development Application and EPSC plans: ______________________

Tax Lot(s): Tract 1 (17160 SW Baseline Road) and Tract 2 (1065 SW 170th Avenue

Land Use Case file # (s): DR2022-0139/TP2022-0015/LD2022-0018/LLD2023-0003

City of Beaverton Site Development Permit Application # if known (e.g. SD2020-1234): ____________

(N/A) **Stormwater Conveyance Related Questions**

Project area that is not in roadway right-of-way (AKA Onsite)

Predevelopment / Pre-Redevelopment impervious area: ______________ Sq-Ft

A) Post development / Post-Redevelopment impervious area: ____________ Sq-Ft

Net Difference: ______________ Sq-Ft

Notes (optional): ____

Note: The Clean Water Services Rates and Charges Resolution and Order shows how to measure/determine impervious area.

Stormwater Quantity Questions

Project area that is not in roadway right-of-way (AKA Onsite)

Post development / Post-Redevelopment impervious area that does not receive quantity mitigation (unmanaged impervious area) ______________________ Sq-Ft

Project roadway right-of-way frontage improvement area (AKA Offsite)

Impervious area as measured from roadway crown to edge of right-of-way that does not receive quantity mitigation (unmanaged impervious area) 12,310 (Baseline) / 24,680 (170th) Sq-Ft

Notes (optional): ____

Stormwater Hydromodification Questions

Project area that is not in roadway right-of-way (AKA Onsite)

Post development / Post-Redevelopment impervious area that does not receive hydromodification mitigation (unmanaged impervious area) ________________________ Sq-Ft

Project roadway right-of-way frontage improvement area (AKA Offsite)

Impervious area as measured from roadway crown to edge of right-of-way that does not receive hydromodification mitigation (unmanaged impervious area) 12,310 (Baseline) / 24,680 (170th) Sq-Ft

Notes (optional): ____

Stormwater Quality Questions

Project area that is not in roadway right-of-way (AKA Onsite)

Post development / Post-Redevelopment impervious area that does not receive surface water treatment (unmanaged impervious area) ________________________ Sq-Ft

Project roadway right-of-way frontage improvement area (AKA Offsite)

Impervious area as measured from roadway crown to edge of right-of-way that does not receive surface water treatment (unmanaged impervious area) 12,310 (Baseline) / 24,680 (170th) Sq-Ft

Notes (optional): ____

Stormwater Utility Billing Setup Questions – Not to be used for Single Family Residential

For sites that have other than single family lots, please identify the area in Sq-Ft that will be assigned to the site, per **A** above.

A) Post development / Post-Redevelopment impervious area: _______________ Sq-Ft

City / CWS annual report to Oregon DEQ as required via the NPDES-Watershed based permit and the associated stormwater management plan (some questions are repetitive from above).

Post development / Post-Redevelopment impervious area added with this project with stormwater treatment: ________________________ Sq-Ft

Post development / Post-Redevelopment impervious area added with this project without stormwater treatment: ________________________ Sq-Ft

Post development / Post-Redevelopment impervious area added with this project with vegetated LIDA stormwater treatment facilities: ________________________ Sq-Ft

Post development / Post-Redevelopment impervious area added with this project structural stormwater treatment facilities (such as stormwater filters): ________________________ Sq-Ft

Total new impervious surface area (in Sq-Ft) related to this development / redevelopment project: ________________________ Sq-Ft