Preliminary Stormwater Management Report

Westgate and Hall

Address: 3775 SW HALL BOULEVARD, BEAVERTON, OR 97005
Owner: Cedar Street
Prepared for: AMAA
Prepared by: Josh Lighthipe & Justin Collinson
Project Engineer: Josh Lighthipe

Revised April 2023 | KPFF Project #220347

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Designer’s Certification

Joshua A Lighthipe, PE

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Project Overview

Purpose of this report

This report describes the stormwater management design strategies for the proposed development. The basis for this report is the City of Beaverton 2019 Engineering Design Manual (EDM) and the requirements outlined therein. The purpose of the proposed stormwater management facilities is to protect existing public stormwater infrastructure and to improve the overall health of the watershed.

Project Description

The project consists of the redevelopment of a 1.86-acre property containing two existing commercial buildings and a large parking lot at the corner of SW Westgate Drive and SW Hall Boulevard in Beaverton, Oregon (see Figure 1 – Vicinity Map). The project site is bounded by SW Hall Boulevard to the north, SW Westgate Drive to the east, and private commercial retail lots to the west and Beaverton Creek to the south. The project includes the demolition of the two existing commercial buildings and associated asphalt-paved surface parking lots, and the construction of a 7-story residential apartment building with internal parking on the ground floor. Additionally, there is a shared access driveway on the west side of the building that also serves the neighboring properties to the west. This will mostly remain intact or be rebuilt in a similar manner to existing conditions.

Due to the right-of-way dedications along both street frontages the proposed site will be approximately 1.78 acres, or 0.08 acres smaller than the existing property.

See Appendix A for the proposed and existing area exhibits, for the amount of pervious and impervious areas. See Table 1 below for summary of the site areas.

TABLE 1: ON-SITE EXISTING AND POST-DEVELOPMENT AREAS

Basin	Impervious Area	Pervious Landscaped Area	Total Site Area	
	(sf)	(sf)	(sf)	(acres)
Existing	51,261	29,813	81,073	1.86
Post-Development	54,217	23,401	77,618	1.78

Figure 1: Vicinity Map

Existing Conditions

The topography of the project site drains to existing on-site parking lot catch basins. The high point is in the center of the parking lot planting divider with an elevation of approximately 176-feet. The low point at each existing catch basin is approximately between elevation 174- and 175-feet. Along the southern property line there is a steep embankment leading down to the Beaverton creek. The existing finished floor elevation of the two buildings are approximately 175.3- and 176.5-feet respectively. The parking lot has driveway access on the SW Hall Boulevard at the northeast corner of the property as well as driveway access along SW Westgate Drive.

All onsite stormwater drains to a central storm pipe that discharges into the public storm system at a catch basin in SW Westgate Drive at the southeastern side of the site. There are no existing water quality or flow control facilities on site.

Geotechnical investigations were documented in a report prepared by NV5 dated August 29th, 2022. Infiltration testing was not performed at the site, but based on the soils and groundwater conditions the report states that on-site infiltration is not feasible. They recommend detention facilities account for seasonally shallow ground water conditions. The report states that groundwater was encountered at a depth between approximately 7- and 15-feet BGS. Additional that shallower groundwater as well as perched water can typically be encountered during the wet winter to spring months and could be within 5-feet of the ground surface during wetter months. See Appendix B1 for a copy of this report.

Based on the NRCS soils survey information the site mostly consists of Cove Silty Clay Loam which is Hydrologic Soil Group D. See Appendix B2.

Proposed Conditions

In the proposed conditions, the two existing buildings will be removed, and all the existing drainage infrastructure and site features will be demolished. The new building will occupy the majority of the property excluding the southern edge of the property where a 50-foot setback will be provided from the edge of wetland to protect the Beaverton Creek area.

The ground floor has mixed-use retail with an internal parking garage. The parking garage has access from the southeast corner of the property along SW Westgate Drive. A portion of the residential building roof will be constructed with and eco-roof system.

Stormwater from roof and the ground surfaces will be conveyed through an internal pipe network to a sedimentation manhole and then into a detention gallery below the first-floor ground level parking area. It will provide temporarily storage for stormwater to slowly release through a flow control structure at predeveloped rates to the public storm system.

The sedimentation manhole is designed to provide some level of debris and sediment trapping to protect the detention gallery and outlet flow control structure. The detention gallery will be composed of R-Tank® modular units that provide 95% void space. See Appendix D1, Utility Plan C4.0 and Appendix E2, R-tank submittal of the preliminary design.

Per the city comments at the pre-app meeting there is an existing water quality filter cartridge manhole in SW Westgate that this site discharges to which is assumed to be sized for the runoff from this site. See Appendix D2 for as-builts of this facility. This facility should allow the project to opt for the in-lieu of fee option to avoid providing a water quality facility onsite.

The public frontage improvements to SW Westgate Drive and Hall Boulevard will include a new 10-foot curb-tight sidewalk with attached tree wells and a 5-foot bike lane.

This existing water quality manhole in Westgate will also provide treatment of the disturbed and new impervious areas along the SW Westgate drive frontage. The SW Hall frontage drains to a different public storm system and will therefore require a LIDA planter incorporated within the sidewalk furnishing zone to meet the water quality requirements for that new and disturbed impervious area.

See Appendix D1, Sheet C4.0 Utility Plan for more information.

See Table 2 below for summary of the post development site areas.

TABLE 2: POST DEVELOPMENT AREA TABLE

Catchment Area	Storm Facility	Source (Roof, road or other)		New Imp. Area (SF)	Landscape Area (SF)	Eco-roof Area (SF)	Total Area (SF)	
			CN =	98	73	61		
A1	DG	Upper Building		40,362			40,362	
A2	DG	Internal Courtyard		11,379			11,379	
A3	DG	Hardscape		2,476			2,476	
A4	DG	Eco-roof				5,767	5,767	
		Totals :		**54,217**	**-**	**5,767**	**59,984**	
							1.38	**Acres**
						Weighted CN	**94.4**	
					CN=	**48**		
B1	-	Landscape		-	15,770		15,770	
B2	-	Hardscape		1,319			1,319	
C1	-	Tree Wells			444		444	
C2	-	SW Westgate Frontage		3,561			3,561	
C3	-	SW Hall Frontage		3,163			3,163	
C4	LIDA Planter 1	SW Hall Frontage		1,272			1,272	
C5	LIDA Planter 1	Landscape Planter				209	209	
		Totals :		**9,315**	**16,214**	**-**	**25,738**	
							0.59	**Acres**
						Weighted CN	**81.5**	
	Total All Catchment areas (A+B+C)						**1.97**	**Acres**
Undisturbed Areas								
EX-1		Ex. Pavement-Undisturbed		1,784			1,784	
EX-2	LIDA Planter 1	Ex. Road - Undisturbed		3,296			3,296	
		Totals :		**5,080**	**-**	**209**	**5,080**	
							0.03	**Acres**

Stormwater Management Requirements

Water Quantity Control

Section 4.02.2 of the Standards identifies the criteria that require on-site detention for conveyance capacity, and identification of a downstream deficiency is the only criteria that could be applicable to the project. Due to the immediate proximity Beaverton Creek where the public storm drain serving this site discharges it is assumed there is no downstream deficiency.

Hydromodification

Per section 4.03 of the Standards, a hydromodification assessment is required for projects that modify 12,000-square-feet or greater of impervious surface. The project will result in more than 12,000-square-feet of modified impervious area, and therefore, a hydromodification assessment was conducted for the project.

The project site discharges to a public storm main that drains to Beaverton Creek. Per the Hydromodification Map, the Risk Level at this point is low. See Appendix C2.

The project size is less than 80,000-square-feet. Therefore, per Table 4-2 of the Standards, the project's hydromodification requirement is Category 2.

The project will use Peak-Flow Matching Detention to demonstrate compliance with the hydromodification requirement. Since only catchment A will drain to the detention gallery, the facility will over detain that area to make up for catchments B & C that do not have flow control. The combined total discharged from the post-development conditions will meet the hydromodification flow control target release rates.

Stormwater Quality Treatment

Per section 4.04 of the Standards, developments that create or modify more than 1,000-square-feet of impervious surface or increase the amount of stormwater runoff or pollution leaving the site are required to implement water quality approaches to reduce contaminants entering surface waters. This project will result in more than 1,000-square-feet of modified impervious surface, and therefore, must demonstrate compliance with the water quality requirements.

Along SW Westgate Dr before the bridge there is a regional storm water quality filter vault that the stormwater from this site will drain to, this project meets the fee in lieu exemption requirements per:

> CWS section 4.04.2.(a)(2).
>
> *There is a more efficient and effective regional approach within the subbasin that was designed to incorporate the development, or there is an approach in the subbasin which is demonstrated to have the capacity to treat the site.*

Therefore, this project is requesting to pay a fee-in-lieu for the modified impervious areas which drain to the existing water quality manhole. These impervious areas total **<u>59,097 sf</u>** (Areas A1, A2, A3, B2 & C2).

Per CWS 4.08.d.1 the area to be considered in the fee-in-lieu is the total impervious area and three times the modified impervious area (except not for ROW area C2), resulting in a required total of **170,169-square-feet** of water quality area.

Public improvements along SW Hall Blvd (Areas C3 & C4) drains to a different public storm main and will therefore require water quality treatment. Since it is not possible to provide a LIDA Planter to only capture this new and disturbed area of Areas C3 & C4, the project will instead provide a LIDA planter that manages and equal or greater area of undisturbed SW Hall roadway to offset the portion of new impervious area that cannot be practically managed.

Therefore, the LIDA planter will be located east of the low-point area along the frontage and will collect Area C4 and undisturbed road Area EX-2. Due to the constrained sidewalk furnishing zone the planter will only have an internal width of 3.5'. Therefore, the growing medium soil depth will be increased to 30" to utilize the 4.5% sizing ratio (typical 6% WQ sizing with a 25% reduction). See table 3 below for the planter sizing. The LIDA planter will be design per Beaverton standard detail No. 370.

TABLE 3: LIDA Planter Sizing Table

Actual Treatment Areas	Imp. Area (SF)	Min. LIDA Planter size (4.5% using 30" growing medium depth) (SF)	Actual LIDA Planter size (SF)	Sufficiently sized?
C4 + EX-2	4568	206	209	YES

Area Requiring Treatment	Imp. Area (SF)	Is Actual treatment area (C4 + EX-2) > required treatment area (C3 + C4)?
C3 + C4	4435	YES

Conveyance

The stormwater conveyance system is sized using the Santa Barbara Urban Hydrograph (SBUH) Method to convey the 25-year storm event. See Appendix C-X for Conveyance Calculations. (Included in final version of this report)

Hydrologic Analysis

Assumptions

The following assumptions were used in performing the stormwater calculations for the project:

- Water Quality Design Storm: 0.36-inches in 3-hours with an average return period of 96-hours

- 24-hour Design Storms with Type 1A Rainfall Distribution
    - 2-year: 2.50-inches
    - 5-year: 3.10-inches
    - 10-year: 3.45-inches
    - 25-year: 3.90-inches
    - 100-year: 4.50-inches
- Curve numbers were established for the project as:
    - Proposed Impervious Areas: CN=98
    - Proposed Eco-roof areas: CN=61
    - Predevelopment conditions: CN=79 (based on Hydrologic Soil Group D & Woods in fair condition)
    - Existing and proposed landscape areas: CN=73, based on enhanced plantings and mulch ground cover installed these areas
- Time of Concentration
    - Existing Condition: 5-minutes
    - Developed Condition: 5-minutes

Methods of Analysis

Runoff calculations were performed using Hydraflow Hydrographs Extension for AutoCAD® Civil 3D® 2020 with the SBUH method.

Peak-Flow Matching Detention

Section 4.08.6.c of the Standards gives the required flow targets for peak-flow matching detention as part of a hydromodification approach. The flow control targets, pre-developed and post-developed flows are summarized in the table below. The hydromodification accounts for the full property area (areas A1-A4 & B1-B2) and include the disturbed off-site areas (areas C1-C5), which totals 1.97 acres.

TABLE 4: Peak-Flow Matching Detention Summary

Storm Event	Post-developed Flow Control Target	Pre-developed Flow Rate (cfs)	Target Flow Rate (cfs)	Post-developed Flow Rate (cfs)
2-year, 24-hour	50% pre-developed	0.32	0.16	0.16
5-year, 24-hour	100% pre-developed	0.53	0.53	0.24
10-year, 24-hour	100% pre-developed	0.67	0.67	0.32
25-year, 24-hour	N/A	0.85	N/A	0.57
100-year, 24-hour	N/A	1.1	N/A	1.1

See Appendix C2 for the hydromodification calculations and Appendix A-1 for the Proposed Development Areas Map.

Operations and Maintenance

Complete O&M to be provided in the final version of this report.
R-tank O&M included in Appendix E3.

Engineering Conclusions

The stormwater system has been designed in accordance with the Clean Water Services Design & Construction Standards. The proposed detention gallery and flow control structure will meet hydromodification requirements. The water quality treatment requirements will be met through the off-site public water quality vault and therefore the project will be requesting a fee-in-lieu to satisfy the water quality requirement.

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Appendix A:

A-1 Proposed Development Areas

A-2 Existing Conditions Areas

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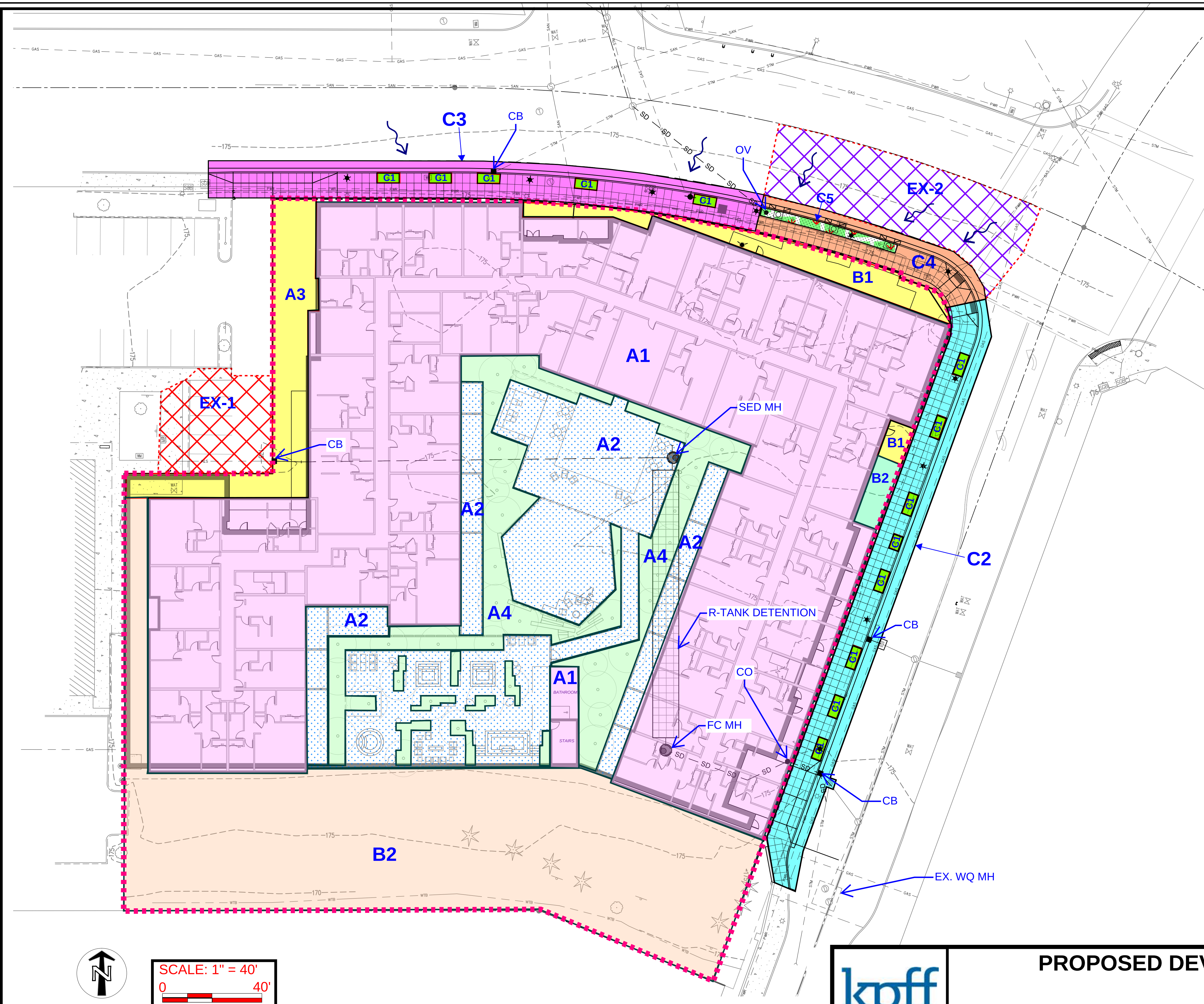

EXISTING AREA (UNDISTURBED)				
	EX-1	PRIVATE HARDSCAPE	1,784	sf
	EX-2	PUBLIC HARDSCAPE	3,296	sf

PUBLIC PROPOSED CONDITIONS				
	C1	PUBLIC LANDSCAPE	444	sf
	C2	PUBLIC HARDSCAPE	3,561	sf
	C3	PUBLIC HARDSCAPE	3,163	sf
	C4	PUBLIC HARDSCAPE	1,272	sf
	C5	LIDA PLANTER-1	209	sf

PRIVATE PROPOSED CONDITIONS				
		TOTAL PROPERTY AREA	77,618	sf
	A1	UPPER BUILDING	40,362	sf
	A2	INTERNAL COURTYARD	11,379	sf
	A3	HARDSCAPE	2,476	sf
	A4	ECOROOF	5,767	sf
	B1	HARDSCAPE	1,319	sf
	B2	LANDSCAPE	15,495	sf
	B2	LANDSCAPE	275	sf

Proposed Property Lines

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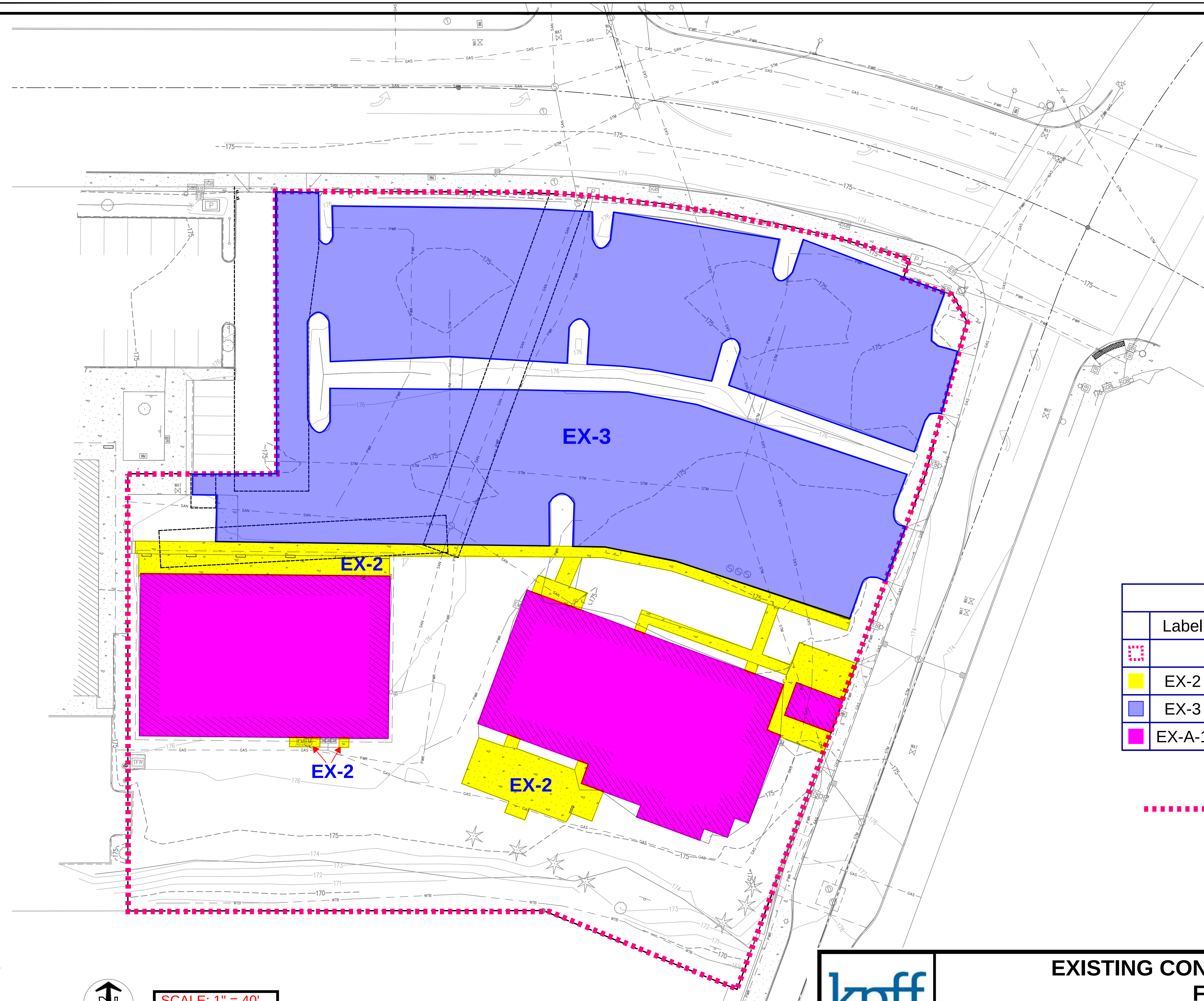
PROPOSED DEVELOPMENT AREAS FOR WESTGATE & HALL

EXHIBIT A-1

2/24/2023

File: N:\c\2022\2200347-Westgate-Hall-Mixed-Use\PROJ-INFO\Planning-Design\Storm\Basin-Maps\22347-Basin-Map.dwg TAB:PROP
Plotted: 4/17/23 at 12:03pm By: JCollinson



Existing Conditions				
	Label	Description	Quantity	Unit
		TOTAL PROPERTY AREA	81,073.1	sf
	EX-2	Concrete/Sidewalk	4,696.2	sf
	EX-3	Asphalt/Parking Lot	32,517.4	sf
	EX-A-1	Existing Building	14,046.9	sf

Existing Property Line

SCALE: 1" = 40'
0 40'

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EXISTING CONDITIONS AREAS FOR WESTGATE & HALL

EXHIBIT A-2

2/24/2023

Appendix B:

B1 Geotechnical Investigation

B2 NRCS Soil Survey Output

REPORT OF GEOTECHNICAL ENGINEERING SERVICES

Westgate + Hall Development
3775 SW Hall Boulevard
Beaverton, Oregon

For
Cedar St. Companies
August 29, 2022

Project: CedarSt-1-01

NV5

August 29, 2022

Cedar St. Companies
1020 West Lawrence Avenue, Suite 300
Chicago, IL 60640

Attention: Griffin Epping

Report of Geotechnical Engineering Services
Westgate + Hall Development
3775 SW Hall Boulevard
Beaverton, Oregon
Project: CedarSt-1-01

NV5 is pleased to submit this report of geotechnical engineering services for the Westgate + Hall development in Beaverton, Oregon. Our services for this project were conducted in accordance with our proposal dated July 8, 2022.

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

Sincerely,

NV5

Nick Paveglio, P.E.
Principal Engineer

NNP:kt
Attachments
One copy submitted
Document ID: CedarSt-1-01-082922-geor.docx

EXECUTIVE SUMMARY

This section provides a summary of the main geotechnical considerations associated with the Westgate + Hall development in Beaverton, Oregon. Our conclusions are based on the proposed site development information provided by the design team and the information presented in this report. This summary is an overview and the report should be referenced for a thorough discussion of the subsurface conditions and geotechnical recommendations for the project.

- Based on correspondence with the development team, the finished floor slab of the building may need to be raised to 183.6 feet (8 to 9 feet of new fill) to be above the flood elevation of Beaverton Creek. Fills of this magnitude will induce settlement of the subsurface soil at the site. Where new fills exceed 3 feet, fill-induced settlement should be complete prior to construction of structural elements (footings, floor slabs, pavement) associated with development. Confirmation of fill-induced settlement completion prior to construction of structural elements should be determined using settlement plates as described in the "Settlement Monitoring" section.

- The site is blanketed by 6 to 9.5 feet of undocumented fill. Due to the variable consistency and strength properties of the fill, it is not suitable to support the proposed building. The undocumented fill will have to be excavated to native soil beneath the footings and replaced with compacted crushed rock or foundations will need to bear on soil improvements that extend into native soil. Where foundation loads exceed 400 kips, foundations cannot be supported directly on native soil and should be underlain by soil improvements.

- Based on the thickness of fill and depth of groundwater at the site, over-excavation and backfill with crushed rock is likely cost prohibitive if the finished floor slab of the building is at the current elevation of the site (175 to 176 feet) and especially if the finished floor slab is raised to 183.6 feet. Therefore, we anticipate spread footings on soil improvements are the most feasible foundation alternative.

- There is a small risk for poor performance of slabs-on-grade and pavement established directly over undocumented fill. Slabs-on-grade and pavement can be constructed on undocumented fill, provided a small risk of distress is accepted (minor floor slab cracking or bird baths) and subgrades are evaluated as described in the "Site Preparation" section. To reduce the potential for cracking, slabs-on-grade can incorporate additional reinforcement to span areas where differential settlement occurs. If risk cannot be accepted, the fill will need to be removed and recompacted as structural fill or the pavement and slabs-on-grade will need to structurally supported.

- Existing soil at the site is suitable for use as structural fill, provided it is properly moisture conditioned. The moisture content of the near-surface on-site soil is expected to be significantly above the optimum moisture content required for construction and moderate to significant moisture conditioning is expected. Accordingly, we anticipate on-site soil can only be used as structural fill in the dry season and imported granular fill will be required for structural fill during much of the year.

- Liquefaction, lateral spreading, and fault rupture are not design considerations for the project.

- Groundwater was encountered at a depths between approximately 7 and 15 feet BGS during explorations at the site. Shallower groundwater as well as perched water is typically encountered during the wet winter to spring months and could be within 5 feet of the ground surface during certain times of the year. These shallow depths to groundwater could impact site cuts for foundations, utilities, and storm ponds. Dewatering should be assumed for excavations that extend more than a few feet below the existing ground surface.

- If the building floor slab is not raised above the flood elevation of Beaverton Creek, the building walls will need to be structurally designed to resist hydrostatic pressure and uplift pressures of the flood event.

- Some concrete rubble, debris, wood, and other obstructions could be encountered in the fill at the site. These materials can lead to difficult excavation.

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ACRONYMS AND ABBREVIATIONS

AASHTO	American Association of State Highway and Transportation Officials
AC	asphalt concrete
ACP	asphalt concrete pavement
ADT	average daily traffic
ASCE	American Society of Civil Engineers
ASTM	American Society for Testing and Materials
BGS	below ground surface
CFA	continuous flight auger
CMMP	Contaminated Media Management Plan
CPT	cone penetration test
CRBG	Columbia River Basalt Group
CSZ	Cascadia Subduction Zone
DSMC	deep soil mix column
g	gravitational acceleration (32.2 feet/second^2)
H:V	horizontal to vertical
km	kilometers
ksf	kips per square foot
MCE	maximum considered earthquake
mm/yr	millimeters per year
OSHA	Occupational Safety and Health Administration
OSSC	2021 Oregon Standard Specifications for Construction
pcf	pounds per cubic foot
pci	pounds per cubic inch
PG	performance grade
psf	pounds per square foot
psi	pounds per square inch
SOSSC	State of Oregon Structural Specialty Code
SPT	standard penetration test

1.0 INTRODUCTION

NV5 is pleased to provide this geotechnical report for the Westgate + Hall development in Beaverton, Oregon. The site is located at 3775 SW Hall Boulevard. The site is shown relative to surrounding features on Figure 1. Figure 2 shows the existing conditions at the site. Acronyms and abbreviations used herein are defined above, immediately following the Table of Contents.

2.0 BACKGROUND

Table 1 provides a history of development at the site based on a review of resources obtained during preparation of the Phase I Environmental Site Assessment completed by NV5 (NV5, 2022).

Table 1. Development History at the Site

Year	Development
1915 through 1970	By 1915, the site appears as undeveloped land. By 1936, the site was used for agricultural purposes with row crops. By 1947, a rural residential structure was constructed on the north-central portion of the site. The site remained relatively unchanged through 1970.
1975 through present	By 1975, the residence had been removed and the site appears to be no longer used for agricultural purposes. The western structure was constructed in 1977 and the eastern structure was constructed in 1978. By 1981, the site appears developed in its current configuration. The site has remained relatively unchanged, except for ownership and commercial tenants, from 1981 through to the present.

3.0 PROJECT UNDERSTANDING

Proposed site development will consist of a six-story, above-grade structure and associated infrastructure. Due to the location of the site within the Beaverton Creek floodplain, below-grade levels are not planned. The structure will be U-shaped with an elevated courtyard in the center and will primarily be a residential structure with the exception of retail space on the ground floor. Figure 3 shows the footprint of the proposed structure.

Loading was not available at the time of the report. Based on experience with similar structures, maximum column and wall loading could be up to 800 kips and 10 kips per foot, respectively. Slab loading is expected to be less than 200 psf.

Due to the location of the site within the floodplain, it may be necessary to raise the site by 8 to 9 feet to maintain a finished floor slab elevation of 183.6 feet (2 feet above the flood elevation of Beaverton Creek). At the time this report was prepared, if was unknown if the raising the site would be required by the local jurisdiction.

4.0 PURPOSE AND SCOPE

The purpose of our services was to provide geotechnical engineering recommendations for design and construction of the proposed project. The specific scope of our services is summarized as follows:

- Reviewed existing geotechnical studies completed near the site.
- Completed the following explorations at the site.
  - Drilled four borings to depths between 11.5 and 51.5 feet BGS
  - Advanced two CPT probes to refusal between depths of approximately 58.5 and 60 feet BGS
- Conducted a laboratory testing program that consisted of the following:
  - Fifteen moisture content determinations in general accordance with ASTM D2216
  - Nine particle-size analyses in general accordance with ASTM D1140
  - Three Atterberg limits tests in general accordance with ASTM D4318
  - One consolidation test in accordance with ASTM D2435
  - One dry density determination in general accordance with ASTM D2937
- Provided this geotechnical report for the project that includes the following:
  - Assessment of liquefaction and lateral spreading
  - Foundation support options
  - Recommendations for site preparation and grading, including temporary and permanent slopes, fill placement criteria, suitability of on-site soil for fill, subgrade preparation, and wet weather construction
  - Recommendations for AC pavement
  - Seismic design recommendations
  - Recommendations for use in design of retaining structures, including backfill and drainage requirements and lateral earth pressures
  - Discussion of groundwater conditions at the site, including general recommendations for dewatering during construction and subsurface drainage as well as design groundwater elevation

5.0 SITE CONDITIONS

5.1 SURFACE CONDITIONS

The site is located at 3775 SW Hall Boulevard in Beaverton, Oregon. The approximately 1.85-acre site is bound by SW Hall Boulevard to the north, commercial development to the west, Beaverton Creek to the south, and SW Westgate Boulevard to the east. The site is currently occupied by two, single-story commercial buildings and associated AC parking and drive areas. Vegetation at the site consists of trees, grass, and landscape bushes in islands in the parking lots and near Beaverton Creek.

Topography at the site is generally flat with elevations generally ranging between 175 and 176 feet in the proposed development area. Topography slopes gently downward in the extreme southern portion of the site toward Beaverton Creek.

5.2 GEOLOGIC SETTING

The site is located west of the Tualatin Mountains (or Portland Hills). The Tualatin Mountains form the physiographic boundary between the Portland Basin to the east and the Tualatin Basin to the west. These basins are part of the larger Puget Sound-Willamette Valley physiographic province, a tectonically active lowland situated between the Coast Ranges to the west and the Cascade Mountains to the east (Orr and Orr, 1999).

The near-surface soils are mapped as silt, clay, and fine sand deposited by the glacial-outburst Missoula floods approximately 15,500 and 12,500 years ago. The floodwaters left behind thick accumulations of unconsolidated sediment where they flowed through the Portland Basin. The Missoula flood deposits typically range from 30 to 60 feet thick in the area.

The Miocene- to Pleistocene-aged Hillsboro Formation consisting predominately of siltstone, claystone, and sandstone underlies to the fine-grained flood deposits. Water well logs and geologic mapping indicate the Hillsboro Formation extends between 800 and 1,200 feet BGS in the area.

Basement rocks in the vicinity of the site are similar to those exposed in the adjacent Tualatin Mountains, which primarily consist of the Miocene CRBG (approximately 17 million to 6 million years old) (Beeson et al., 1991; Madin, 1990). The CRBG consists of thick flows of basalt that have been folded and faulted into mountains and valleys from the compressional tectonics of the region. Rivers flowing through the Portland Basin eroded channels through the uplifted basalts and deposited alluvium to fill adjacent valleys.

5.3 SUBSURFACE CONDITIONS

Subsurface conditions at the site were evaluated by drilling four borings (B-1 through B-4) to depths between 11.5 and 51.5 feet BGS and advancing two CPT probes (CPT-1 and CPT-2) to refusal on dense soil between depths of approximately 58.5 and 60 feet BGS. Locations of the explorations are shown on Figures 2 and 3. Logs and laboratory testing results from the borings are presented in Appendix A. The CPTs results are presented in Appendix B.

In addition to the explorations described above, GeoDesign, Inc. (now NV5) completed drilled borings in SW Westgate Drive in 2011 as part of replacement of the bridge over Beaverton Creek. Logs of applicable explorations are presented in Appendix C. The locations of the explorations are shown on Figures 2 and 3.

Subsurface conditions at the site consist of up to 9.5 feet of fill underlain by silt and clay with variable proportions of sand. A detailed description of the soil conditions at the site is presented below.

5.3.1 Fill

The near-surface soil directly below the ground surface consists of historical fill comprised predominately of silt and clay with of variable proportions of sand, construction debris, and trace organics and gravel. The fill extends to depths between 6 and 9.5 feet BGS.

5.3.2 Native Silt and Clay

Underlying the fill is soft to very stiff silt and clay with variable proportions of sand. Stiffness of the silt and clay generally increases with depth. The silt and clay are gray to brown and moist to wet. The clay generally has moderate to high plasticity and the silt generally has no to low plasticity. The silt and clay extend to the maximum depth explored in the explorations at the site.

Laboratory testing indicates the moisture content of the native silt and clay varied between 27 and 46 percent at the time of exploration.

5.3.3 Groundwater

Groundwater was observed in the borings between approximately 7 and 15 feet BGS. The shallowest groundwater level (B-4) was observed in fill and we anticipate this was perched groundwater. The remainder of the observed groundwater was in native soil between depths of approximately 10 and 15 feet BGS. Groundwater was measured in August when groundwater levels are near seasonal lows. We anticipate groundwater could raise to within 5 feet of the existing ground surface during certain times of the year. We anticipate that perched water could be present above these elevations and particularly in the fill where higher infiltrating soil is underlain by lower infiltrating soil.

6.0 SEISMIC HAZARDS

6.1 SEISMIC SETTING

6.1.1 Earthquake Source Zones

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

6.1.2 Regional Events

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

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

1. An interface event earthquake on the seismogenic part of the interface between the Juan de Fuca Plate and the North American Plate on the CSZ. This source is capable of generating earthquakes with a moment magnitude of 9.0.
2. A deep intraplate earthquake on the seismogenic part of the subducting Juan de Fuca Plate. These events typically occur at depths of between 30 and 60 km. This source is capable of generating an event with a moment magnitude of up to 8.0.

6.1.3 Local Events

An earthquake could occur on a local fault within the North American Plate. Such an event would cause ground shaking at the site that could be more intense than the CSZ events, although the duration would be shorter. The closest mapped distance and mapped length of nearby faults are provided in Table 2.

Table 2. Nearest Mapped Crustal Faults

Source	Closest Mapped Distance (km)	Mapped Length (km)
Beaverton Fault zone	2.0	15
Oatfield fault	5.8	24
Portland Hills fault	10.3	49
Helvetia fault	10.4	7

All faults in Table 2 have slip rates less than 0.2 mm/yr and the site does not meet the qualifications of being "near-fault" in ASCE-7-16 Section 11.4.1.

6.2 FAULT RUPTURE

Active faults are not mapped directly beneath the site. Therefore, it is our opinion that the risk of fault rupture at the site is low.

6.3 LIQUEFACTION/SEISMIC SETTLEMENT

Liquefaction is a phenomenon caused by a rapid increase in pore water pressure during a seismic event 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. Liquefaction can cause seismically induced densification of subsurface soil, which can result in settlement at the ground surface.

Low plasticity silty sand and silt may also be susceptible to seismic settlement during a seismic event under relatively higher levels of ground shaking; however, the magnitude of settlement at the ground surface is less than liquefaction settlement.

Based on laboratory testing, sandy soil is not present at the site and liquefaction is not a design consideration. Based on the stiffness and plasticity of silt and clay at the site we anticipate seismic settlement as a result of strain softening will be negligible.

6.4 LATERAL SPREADING

Lateral spreading is a liquefaction-related seismic hazard that occurs on gently sloping or flat sites underlain by liquefiable sediment adjacent to an open face, such as a riverbank. Liquefied soil adjacent to an open face can flow toward the open face, resulting in lateral ground displacement.

There is a small slope leading to Beaverton Creek in the southern portion of the site; however, because liquefaction is not expected, lateral spreading is not a design consideration for the project.

7.0 SITE DEVELOPMENT RECOMMENDATIONS

7.1 SITE PREPARATION

7.1.1 Demolition

Demolition should include removal of existing pavement, concrete curbs, abandoned utilities, and other buried elements. Material generated from demolition should be transported off site for disposal or recycled and used on site if it is acceptable for use as structural fill. The resulting voids should be backfilled with structural fill. All excavations should follow the CMMP prepared by the project environmental engineer and approved by the Oregon Department of Environmental Quality.

7.1.2 Grubbing and Stripping

Trees and shrubs should be removed from fill areas. In addition, root balls should be grubbed out to the depth of the roots, which could exceed 3 feet BGS. Depending on the methods used to remove root balls, considerable disturbance and loosening of the subgrade could occur during site grubbing. We recommend that soil disturbed during grubbing operations be removed to expose firm, undisturbed subgrade. The resulting excavations should be backfilled with structural fill.

The existing topsoil zone should be stripped and removed from all fill areas. Based on our experience, root zones will likely be between 2 and 6 inches, although greater stripping depths may be required to remove localized zones of loose or organic soil. Greater stripping depths (approaching 12 inches) may be anticipated in areas with thicker vegetation and shrubs, which should be expected in the tree and shrub area at the site. 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 in accordance with the project CMMP.

7.1.3 Undocumented Fill

7.1.3.1 General

Undocumented fill was encountered to depths of up to 9.5 feet BGS at the site. Documentation on the placement and compaction of the fill is not available. Due to the variable composition of the fill, and the unknown methods of placement and compaction, reliable strength properties for undocumented fill are extremely difficult to predict.

7.1.3.2 Foundation Areas

Undocumented fill should be removed from under new building foundations and footings supported on granular pads as discussed in the "Foundation Support Recommendations" section. Alternatively, the foundations can be underlain by soil improvements that extend to native soil.

7.1.3.3 Floor Slab Areas

There is a small risk for poor performance of floor slabs established directly over undocumented fill soil. Removal and replacement of undocumented fill would eliminate all risk. Provided a small risk of distress is accepted, there is an option to limit the subgrade stabilization to removal and replacement or scarifying and recompacting the upper 1 foot of the undocumented fill material within floor slab areas if the finished floor grade is not more than 1 foot above existing grades. Undocumented fill, if present, should be removed and replaced with structural fill if the finished floor grade is more than 1 foot above existing grades.

7.1.3.4 Pavement Areas

There is a small risk for poor performance of pavement established directly over undocumented fill soil. Removal and replacement of the undocumented fill would eliminate all risk. Provided a small risk of distress (or low areas resulting in localized "bird baths") is acceptable, there is an option to limit the subgrade stabilization to removal and replacement or scarifying and recompacting the upper 1 foot of the undocumented fill material within pavement areas if the finished pavement grade is less than 3 feet above existing grades. Undocumented fill, if present, should be removed and replaced with structural fill if the finished pavement grade is more than 3 feet above existing grades.

7.2 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 will likely not support construction traffic or pavement construction. Moreover, 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 and should, therefore, be the responsibility of the contractor. 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. The contractor should also be responsible for selecting the type of material for construction of haul roads and staging areas. A geotextile fabric can be placed as a barrier between the subgrade and imported granular material in areas of repeated construction traffic to help prevent silt migration into the base rock. 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 should be the contractor's responsibility. Cement amendment is discussed in the "Soil Amendment with Cement" section.

7.3 TEMPORARY SLOPES

Excavation side slopes for cuts of less than 15 feet high should be no steeper than 1.5H:1V, provided groundwater seepage does not occur. If slopes greater than 15 feet high are required, NV5 should be contacted to make additional recommendations. We recommend a minimum horizontal distance of 5 feet from the edge of the existing improvements to the top of the temporary slope. All cut slopes should be protected from erosion by covering them during wet weather. If sloughing or instability is observed, the slope should be flattened or supported by shoring. Excavations should not undermine adjacent utilities, foundations, walkways, streets, or other hardscapes, unless special shoring or underpinned support is provided.

7.4 EROSION CONTROL

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

7.5 SHORING

7.5.1 General

Shoring may be required to support temporary excavations at the site. We recommend that the contractor be responsible for final selection, design, and construction of an appropriate shoring system because they are in the best position to choose a system that fits the overall plan of operation. The shoring design should be stamped by a qualified professional engineer registered in the state of Oregon. The shoring designer should apply an appropriate safety factor against overturning and toe kick-out.

The shoring depth/design should take into consideration the excavations needed for foundations (including planned over-excavation). Groundwater should be maintained a minimum of 2 feet below the base of the shoring.

Construction debris could be present within the fill. The contractor should be prepared to advance through or modify the procedures if debris is encountered. Caving in the fill should be anticipated during construction.

7.5.2 Cantilevered Shoring

Support for shallower excavations that can tolerate some shoring deflection and corresponding ground surface settlement can be provided by installing cantilevered shoring. Cantilevered shoring should be designed using the lateral earth pressures shown on Figure 4. These pressures will allow moderate relaxation of the shoring toward the excavation causing ground surface settlement. Based on our experience, settlement on the order of 1 inch can be expected adjacent to the shoring. We anticipate that settlement will become negligible at a distance of approximately 20 feet from the wall. Figure 5 presents additional lateral earth pressures that result from common surcharge loading scenarios and that should be accounted for in the shoring design. We recommend a vertical live load of 250 psf be applied at the surface of the retained soil where the wall shoring retains roadways.

The design equivalent fluid pressure should also be increased for walls that retain sloping soil. We recommend the lateral earth pressures be increased using the following factors (Table 3) when designing walls that retain sloping soil.

Table 3. Lateral Earth Pressure Increase Factors for Sloping Soil Backfill

Slope of Retained Soil (degrees)	Lateral Earth Pressure Increase Factor
0	1.00
5	1.06
10	1.12
20	1.33
25	1.52
30	2.27

We recommend a minimum soldier pile embedment of 10 feet below the base of the excavation.

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

7.5.3 Anchored Shoring

Support for taller excavations or excavations that support adjacent structures can be provided by installing shoring with tieback anchors. Anchored shoring should be designed using the lateral earth pressures on Figure 4. The lateral earth pressures provided on Figure 4 assume the retained soil is level; we should be contacted for revised lateral pressures if the anchored shoring retains sloping soil. Figure 5 presents additional lateral earth pressures that result from common surcharge loading scenarios and that should be accounted for in the shoring design. The lateral earth pressures should be increased using the factors in Table 3 when designing walls that retain sloping soil.

In addition to the lateral earth pressures described above, soldier piles will be subject to compressive forces as a result of the downward component of the tieback anchor loads. We recommend a minimum soldier pile embedment of 10 feet below the base of the excavation. We recommend an allowable end bearing pressure of 2.5 ksf for piles embedded at least 10 feet below the excavation base and in fill or native fine-grained soil.

We estimate that pile skin friction will likely be low due to the presence soft to medium stiff silt and clay. We recommend using an allowable skin friction value of 0.25 ksf between the soldier pile grout and the native very soft to soft clay and silt.

The bonded zone for the tieback anchors should be maintained outside of the "unbonded zone" shown on Figure 4. It is difficult to estimate the bond strength between the tieback anchors and the surrounding soil, since the soil will consist of fill; wood chips; and very soft, medium plasticity silt. The bond strength will vary depending on the soil type and the method of construction. A variety of methods are available for construction of tieback anchors; therefore, we recommend that the contractor be responsible for selecting the appropriate bonded length, bond strength, and installation methods to achieve the required anchor capacity. Tieback anchors should be locked off at 100 percent of the design load.

Prior to installing production anchors, we recommend that performance testing be conducted on a minimum of three anchors. The purpose of this testing is to verify the installation procedure selected by the contractor before a large number of anchors are installed. Performance testing should be performed to 150 percent of the design load and in accordance with the guidelines provided in *Recommendations for Prestressed Rock and Soil Anchors* (Post Tensioning Institute, 2014). Additional performance testing should be performed on at least 2 percent of the remaining production anchors and at locations where ground conditions vary.

We recommend that proof testing be conducted on all remaining production anchors in accordance with the guidelines presented in *Recommendations for Prestressed Rock and Soils Anchors* (Post-Tensioning Institute, 2014). The anchors should be proof tested to at least 133 percent of the design load.

7.5.4 Other Considerations

If adjacent buildings or structures are located near a proposed excavation, unsupported excavations can be maintained outside of a 1H:1V downward projection that is offset a horizontal distance of 5 feet from the base of the existing footings. Excavations that must be inside this zone should be supported by temporary or permanent shoring designed for moment resistance for the full height of the excavation, including kick-out for the full buried depth of the retaining system.

If excavations are located in close proximity to existing structures, we recommend that utilities and adjacent structures be surveyed and existing cracks measured prior to, during, and after construction of the proposed building. We also recommend that the condition of adjacent structures be documented with photographs or videotaped before and after construction. If settlement or damage is observed in adjacent structures, NV5 should be contacted to provide additional recommendations.

Tieback anchors that extend into the gravel are likely to have higher than normal grout takes due to the open framework of the soil matrix. Based on experience in similar soil, grout takes of 150 to 200 percent or more of the neat volumes are possible. Additives or grout socks can be used to reduce grout volumes.

7.6 CONSTRUCTION DEWATERING

Construction dewatering will be required to maintain dry working conditions in excavations that extend below groundwater. Groundwater is anticipated to be present between approximately 5 and 15 feet BGS during a typical year. We anticipate that perched water could be present above static ground, particularly in fill soil and between the interfaces of dissimilar soil. We anticipate that dewatering will be required to construct the basement during certain times of the year.

Selection, design, and construction of dewatering systems should be the responsibility of the contractor who is in the best position to modify or adapt the system to changing groundwater conditions and construction sequencing and requirements. The construction dewatering system should be adaptable to varying flow conditions and capable of lowering the level of groundwater to a minimum of 2 feet below the base of the excavation. Dewatering deeper than this may be required to provide increased passive resistance for design of the shoring system. Water generated during dewatering should be pumped to a suitable disposal point.

7.7 MATERIALS

7.7.1 Structural Fill

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 3 inches in diameter. Recommendations for suitable fill material are provided in the following sections.

7.7.1.1 On-Site Soil

The on-site soil consists of fill comprised predominately of silt and clay underlain by alluvial silt and clay. The on-site soil may be used as structural fill, provided it can be moisture conditioned and meets the requirements of the CMMP. Based on our experience, the on-site silt and clay are sensitive to small changes in moisture content and may be difficult, if not impossible, to compact adequately during wet weather or when their moisture content is more than a few percentage points above optimum. Laboratory testing indicates that the moisture content of the on-site soil is greater than the anticipated optimum moisture content required for satisfactory compaction. Therefore, this soil may require extensive drying if it is used as structural fill. We recommend using imported granular material for structural fill if the moisture content of the on-site soil cannot be reduced.

7.7.1.2 Imported Granular Material

Imported granular material should be pit- or quarry-run rock, crushed rock, or crushed gravel and sand that is fairly well graded between coarse and fine and has less than 5 percent by dry weight passing the U.S. Standard No. 200 sieve. All granular material must be durable such that there is no degradation of the material during and after installation as structural fill. The percentage of fines can be increased to 12 percent if the fill is placed during dry weather and provided the fill material is moisture conditioned, as necessary, for proper compaction. The material should be placed in lifts with a maximum uncompacted thickness of 12 inches and compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D1557. During the wet season or when wet subgrade conditions exist, the initial lift should have a minimum thickness of 15 inches and should be compacted without the use of vibratory action.

7.7.1.3 Trench Backfill

Trench backfill placed beneath, adjacent to, and for at least 12 inches above utility lines (i.e., the pipe zone) should consist of well-graded granular material with a maximum particle size of 1½ inches and less than 10 percent by dry weight passing the U.S. Standard No. 200 sieve. The pipe zone backfill should be compacted to at least 90 percent of the maximum dry density, as determined by ASTM D1557, or as required by the pipe manufacturer or local building department. Within pavement areas and building pads, the upper portion of the trench backfill should consist of well-graded granular material, with a maximum particle size of 2½ inches.

Trench backfill should be compacted to at least 90 percent of the maximum dry density, as determined by ASTM D1557, or as required by the pipe manufacturer or local building department. The upper 3 feet of the trench backfill should be compacted to at least 95 percent of the maximum dry density, as determined by ASTM D1557.

7.7.1.4 Stabilization Material

Material used to stabilize staging areas, haul roads, and utility trench subgrade should consist of pit- or quarry-run rock, crushed rock, or crushed gravel and sand with a maximum particle size of 4 inches; should have less than 5 percent by dry weight passing the U.S. Standard No. 4 sieve; and should have at least two mechanically fractured faces. The material should be free of organic material and other deleterious material. The stabilization material should be placed in one lift and compacted to a well-keyed, firm condition.

7.7.1.5 Drain Rock

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

7.7.1.6 Floor Slab and Pavement Base Rock

Imported granular material placed beneath building slabs-on-grade should be clean, crushed rock or crushed gravel and sand that is fairly well graded between coarse and fine. The granular material should have a maximum particle size of 1½ inches, 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. 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.

7.7.1.7 Retaining Wall Select Backfill

Backfill material placed behind retaining walls and extending a horizontal distance of ½H, where H is the height of the retaining wall, should consist of imported granular material. The backfill should be placed and compacted as recommended for structural fill, with the exception of backfill placed immediately adjacent to walls. Backfill adjacent to walls should be compacted to a lesser standard to reduce the potential for generation of excessive pressure on the walls. Backfill located within a horizontal distance of 3 feet from retaining walls should 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 (slabs, sidewalk, or pavement) will be placed adjacent to the wall, we recommend the upper 2 feet of fill be compacted to 95 percent of the maximum dry density, as determined by ASTM D1557.

7.7.1.8 Recycled Concrete

Recycled concrete can be used for structural fill, provided the concrete is broken to a maximum particle size of 3 inches. This material can be used as trench backfill if it meets the requirements for imported granular material, which would require a smaller maximum particle size. The material should be placed in lifts with a maximum uncompacted thickness of 12 inches and compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D1557.

7.7.2 Geotextile Fabric

7.7.2.1 Subgrade Geotextile

Subgrade geotextile should conform to OSSC Table 02320-4 and OSSC 00350 (Geosynthetic Installation). A minimum initial aggregate base lift of 6 inches is required over geotextiles. All drainage aggregate and stabilization material should be underlain by a subgrade geotextile.

7.7.2.2 Drainage Geotextile

Drainage geotextile should conform to Type 2 material of OSSC Table 02320-1 and OSSC 00350 (Geosynthetic Installation). A minimum initial aggregate base lift of 6 inches is required over geotextiles.

7.7.3 Soil Amendment with Cement

7.7.3.1 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. The amount of cement used during amendment should be based on an assumed soil dry unit weight of 100 pcf.

In addition, the new Oregon Department of Environmental Quality requirements under 1200C permits include additional requirements for routing and testing runoff from sites where cement amendment is used.

7.7.3.2 Subbase Stabilization

Specific recommendations based on exposed site conditions for soil amendment 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. The amount of cement used to achieve this target generally varies with moisture content and soil type. It is difficult to predict field performance of soil to cement amendment due to variability in soil response, and we recommend laboratory testing to confirm expectations. In general, 6 percent cement by weight of dry soil can be used when the soil moisture content does not exceed approximately 20 percent. If the soil moisture content is in the range of 25 to 35 percent, 7 to 8 percent by weight of dry soil is recommended. 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.

We recommend assuming a minimum cement ratio of 6 percent by dry weight, with higher rates as discussed above. Because of the higher clay, organic, and moisture, we recommend using a higher cement ratio when stabilizing topsoil zone material, likely a minimum of 7 to 8 percent. Multiple tilling passes will likely be required for the clayey soil.

We recommend cement amendment equipment be equipped with balloon tires to reduce rutting and disturbance of the fine-grained soil. A 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 without the use of vibratory action. 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 subgrade beneath buildings and 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 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.

7.7.3.3 Cement-Amended Structural Fill

On-site soil that would not otherwise be suitable for structural fill may be amended and placed as fill over a subgrade prepared in conformance with the "Site Preparation" section. 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.

7.7.3.4 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. In general, cement amendment is not recommended during cold weather (temperatures less than 40 degrees Fahrenheit) or during steady rainfall.

7.8 EXCAVATION

7.8.1 General
Subsurface conditions consist of fill, silt, and clay. Groundwater could be present between depths of 5 and 15 feet BGS. Excavations in fill could ravel at all depths. Sloped excavations in native soil may be used to depths of 15 feet BGS and should have side slopes no steeper than 1.5H:1V, provided groundwater seepage does not occur. Slopes will need to flattened or shored if groundwater is present.

We recommend a minimum horizontal distance of 5 feet from the edge of the existing improvements to the top of any temporary slope. All cut slopes should be protected from erosion by covering them during wet weather. If seepage, sloughing, or instability is observed, the slope should be flattened or shored. Shoring will be required where slopes are not possible. The contractor should be responsible for selecting the appropriate shoring system.

It is the contractor's responsibility to select the excavation and dewatering methods, monitor the trench excavations for safety, and provide any shoring required to protect personnel and adjacent improvements. All trench excavations should be in accordance with applicable OSHA and state regulations.

7.8.2 Construction Debris
Construction debris could be present within the existing fill. Construction debris considerations include the following:

- Excavations can become difficult, if not impossible, with conventional equipment.
- Excavation volumes for utility trenches may be greater than anticipated due to sloughing and the need to remove oversized material.

8.0 SETTLEMENT CONSIDERATIONS

8.1 GENERAL
Due to the location of the site within the Beaverton Creek floodplain, the finished floor slab of the building may need to be raised to an elevation of 183.6 feet (2 feet above the design flood

elevation of 181.6). This will require approximately 8 to 9 feet of fill to be placed at the site. New fills of this magnitude will result in settlement of the fill and native soil at the site. Where new fills exceed 3 feet and to limit building settlements to typically tolerable limits, settlement-sensitive structural elements (footings, floor slab, pavement, and etc.) should not commence until the fill-induced settlement is complete.

Completion of settlement should be determined by surveying as described in the "Settlement Monitoring" section. Completion of settlement will vary based on the thickness of the undocumented fill; however, settlement will likely be complete within 10 to 16 weeks of fill placement. Paving is typically completed more than four months after placement of fill, and settlement monitoring likely will not be required in roadways or parking areas.

8.2 SURCHARGE

Surcharging can be completed to accelerate the settlement time as a result of filling. Surcharging involves stacking soil above the proposed finished grade to pre-compress the underlying soil. A surcharge height of 5 feet will typically reduce the settlement period by one month. The surcharge should extend laterally at least 5 feet beyond the areas of filling. The surcharge embankment side slopes should be inclined no steeper than 1H:1V.

Surcharging can be accomplished in stages (rolling surcharge) if there is not enough material available to cover all of the building footprints or if the building is constructed in multiple phases. If multiple surcharge stages occur over a single building footprint, we recommend that successive surcharge areas overlap by at least 20 feet. Recommendations for surcharge heights and duration will not change for a rolling surcharge, provided the building areas and loads do not change.

All fill placed below finished soil subgrade elevation should be placed and compacted as structural fill. Surcharge material placed above finished subgrade does not need to be compacted as structural fill, provided the total unit weight of the material is at least 100 pcf.

8.3 SETTLEMENT MONITORING

Settlement monitoring plates can be used to monitor fill-induced settlement associated with the project. The settlement monitoring plates should be installed prior to filling and surveyed immediately. A typical settlement plate detail is shown on Figure 6. For ease in handling, the casing and rod portions of the settlement plate are usually installed in 5-foot sections. As filling progresses, couplings are used to install additional sections. Continuity in the monitoring data is maintained by reading and recording the top of the measurement rod immediately prior to and following the addition of new sections. Care must be taken during fill construction not to bend or break the rods.

We recommend that the location of the settlement monitoring points be determined by NV5 and the contractor. For preliminary planning purposes, we recommend settlement monitoring points be installed at a rate of one per 15,000 square feet of area fill is placed.

9.0 FOUNDATION SUPPORT RECOMMENDATIONS

9.1 GENERAL

The site is blanketed by 6 to 9.5 feet of undocumented fill. Due to the variable consistency and strength properties of the fill, it is not suitable to support the proposed building. The undocumented fill will have to be excavated to native soil beneath the footings and replaced with compacted crushed rock or foundations will need to bear on soil improvements that extend into native soil. Where foundation loads exceed 400 kips, foundations cannot be supported directly on native soil and should be underlain by soil improvements.

Based on the thickness of fill and depth of groundwater at the site, over-excavation and backfill with crushed rock is likely cost prohibitive if the finished floor slab of the building is at the current elevation of the site (175 to 176 feet) and especially if the finished floor slab is raised to 183.6 feet. Therefore, we anticipate supporting the building foundations on soil improvements is the most feasible foundation solution. We recommend this be confirmed by the general contractor for the site. It should be noted that the allowable bearing capacity of spread footings on soil improvements is higher than spread footings on native soil which will reduce excavation, concrete, and rebar costs.

If development plans change, and depending on column loads, it may be possible to support the building on conventional spread footings on native soil without soil improvements if a basement is planned. We should be contacted to review our foundation recommendations if a basement is considered for the project.

9.2 CONVENTIONAL SPREAD FOOTINGS ON NATIVE SOIL

9.2.1 General

Provided the subgrade is prepared as described in the “Settlement Considerations” and “Site Preparation” sections, conventional spread footings on granular pads can be used to support column loading up to 400 kips. If column loads exceed 400 kips, foundation should be underlain by soil improvements.

Spread footings should not be constructed on undocumented fill that is present beneath the site. Where encountered below footings, undocumented fill should be completely removed to native soil. Upon verification of native soil by a member of our field staff, the over-excavation should be backfilled with compacted crushed rock to the planned footing base. Over-excavation should extend 6 inches beyond the margins of the footings for every foot excavated below the footing’s base grade. Over-excavation backfill should consist of crushed rock meeting the requirements for 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 prior to constructing the granular pad.

Granular pad thicknesses beneath footings should be as described in Table 4.

Table 4. Recommended Granular Pad Thicknesses for Spread Footings

	Maximum Load: Columns (kips)			Maximum Load: Continuous Footings (kips per foot)		
	0 to 200	200 to 300	300 to 400	0 to 6	6 to 8	8 to 10
Recommend Granular Pad Thickness (inches)	0	12	18	0	12	18

The granular pads should extend 6 inches beyond the margins of the foundations for every foot excavated below the foundations' base grade and should consist of imported granular material as described 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 before placing granular pads as well.

9.2.2 Dimensions and Capacities

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

Continuous wall and isolated spread footings should be at least 18 and 24 inches wide, respectively. The bottom of exterior footings should be at least 18 inches below the lowest adjacent exterior grade. The bottom of interior footings should be established at least 12 inches below the base of the slab.

9.2.3 Settlement

Settlement is the most significant foundation design consideration and it will primarily be controlled by the grading plan and floor slab loading. Provided subgrades are prepared as described in the "Settlement Considerations" and "Site Preparation" sections, we anticipate maximum total static settlement will be less than 1 inch. Differential settlement will likely approach 0.5 inch over 50-foot span. Our estimates of post-construction settlement assume that settlement will occur between the time of filling/slab construction and establishing connections between the building roof and foundation elements. It should be noted that differential settlement could exceed the settlement predicted over larger spans and particularly where significant cut and fill differences occur over the building footprint.

9.2.4 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. Our analysis indicates that the available passive earth pressure for footings confined by native soil and structural fill is 300 pcf, modeled as an equivalent fluid pressure. Typically, the movement required to develop the available passive resistance may be relatively large. Therefore, we recommend using a reduced passive equivalent fluid pressure of 250 pcf. The pressure should be reduced to 125 pcf where footings are below design groundwater. 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 passive resistance, a minimum of 10 feet of horizontal clearance must exist between the face of the footings and any adjacent down slopes.

For footings in contact with native soil, a coefficient of friction equal to 0.35 may be used when calculating resistance to sliding. For footings in contact with the granular footing pads, a coefficient of friction equal to 0.40 may be used when calculating resistance to sliding.

9.2.5 Subgrade Observation

All footing and floor subgrades should be evaluated by a representative of NV5 to evaluate the bearing conditions. Observations should also confirm that all loose or soft material, organic material, unsuitable fill, prior topsoil zones, and softened subgrades (if present) have been removed. Localized deepening of footing excavations may be required to penetrate any deleterious material.

9.3 CONVENTIONAL SPREAD FOOTINGS ON SOIL IMPROVEMENTS

9.3.1 General

Where column loads exceed 400 kips or when over-excavation to native soil is cost prohibitive, conventional spread footings should be underlain by soil improvements. Soil improvements can consist of rigid inclusions, DSMCs, or rammed aggregate piers. If the finished floor slab at the site is raised to an elevation of 183.6 feet, soil improvements should not be installed until fill-induced settlement is complete. Recommendations for foundation alternatives are presented in the following sections.

9.3.2 Rigid Inclusions

Rigid inclusions are a ground improvement system that relies on support from both the subgrade soil and the rigid inclusions. A rigid inclusion typically consists of an 18- to- 30-inch-diameter, unreinforced concrete column that extends through compressible soil and bears on relatively uncompressible soil. Rigid inclusions are typically constructed by advancing a CFA through the compressible soil and into a stiffer bearing soil. Once the auger advances to the design depth, the auger is raised and concrete is pumped into the resulting void.

After rigid inclusions are installed, conventional spread footings are constructed on the rigid inclusions. The allowable bearing pressure of spread footings underlain by rigid inclusions is typically up to approximately 4,000 to 6,000 psf. These values can typically be increased by one-third when considering transient loads, such as wind and seismic forces. We recommend that rigid inclusions have a minimum diameter of 20 inches.

For preliminary planning purposes, the concrete rigid inclusions should have a minimum 28-day compressive strength of 1,000 psi. We recommend rigid inclusions extend at least 5 to 10 feet into the native soil at the site. We anticipate that a 24-inch-thick layer of compacted, angular crushed rock should be placed between the top of the rigid inclusions and the bottom of the footings. We anticipate rigid inclusions embedded in the Hillsboro Formation can achieve allowable capacities of 100 to 160 kips, depending on diameter. Load testing should be performed in accordance with ASTM D1143 to verify the allowable capacity before construction. Contingencies should be in place if testing indicates deeper rigid inclusions are required than anticipated. We recommend additional load testing be completed during construction to verify capacities are met.

We generally recommend that each footing be underlain by at least three rigid inclusions unless individual spread footings are lightly loaded. The rigid inclusions should be arranged symmetrically and evenly beneath each foundation. Beneath continuous footings, we anticipate that rigid inclusions will require a center-to-center spacing of up to 8 feet, provided the structural engineer verifies that this span distance is acceptable. Rigid inclusions should not be allowed to communicate with each other during installation, which may require a one- to two-day cure time before attempting to install an adjacent rigid inclusion.

Lateral loads for spread footings on rigid inclusions can be designed as described in the "Conventional Spread Footings on Native Soil" section.

Rigid inclusions should be designed by a specialty design-build contractor. If rigid inclusions are used for this project, we recommend that NV5 be allowed to review the final design and proposed installation methods. All footing rigid inclusion installation should be monitored by NV5 personnel. In addition, NV5 personnel should also evaluate the compaction of crushed rock between the top of the rigid inclusions and bottom of the spread footings.

Buried debris could be encountered in the fill during installation of rammed aggregate piers. It might be necessary to remove buried obstructions or adjust the shoring design to address buried debris found during construction. We recommend a contingency be in place in the event buried debris is encountered during installation.

9.3.3 DSMCs

DSMCs consist of mixing a soil and cement slurry with the on-site soil to form columns of improved ground. DSMCs are created using a specialty drill rig that injects cement slurry into the ground during the drilling process. Paddles along the shaft mix the soil and cement slurry together until a relatively uniform column of soil and cement is formed. DSMCs typically vary in diameter between 36 and 72 inches. Spoils generated during installation can be used on site as structural fill or hauled off site. Typically, the amount of spoils generated is approximately 20 to 30 percent of the volume of the cement slurry.

After DSMCs are installed, conventional spread footings are constructed on the DSMCs. The allowable bearing pressure of spread footings underlain by rigid inclusions is typically up to approximately 4,000 to 6,000 psf. These values can typically be increased by one-third when considering transient loads, such as wind and seismic forces. DSMC center-to-center spacing is determined by foundation loading. We recommend DSMCs extend a least 5 to 10 feet into the native soil at the site. Load testing should be performed in accordance with ASTM D1143 to verify the allowable capacity before construction. Contingencies should be in place if testing indicates deeper DSMCs are required than anticipated. We recommend additional load testing be completed during construction to verify capacities are met.

We anticipate that a 24-inch-thick layer of compacted, angular crushed rock will be placed between the top of the DSMCs and the bottom of the footings.

Lateral loads for spread footings on DSMCs can be designed as described in the "Conventional Spread Footings on Native Soil" section.

DSMC design should be completed by design-build contractors. If DSMCs are designed by a design-build contractor, NV5 should be allowed to review the final ground improvement design and proposed installation method. Due to the moderate to high plasticity of portions of fine-grained soil, a minimum of two passes (up and down) should be assumed for DSMCs.

A representative of our firm should be present during installation of DSMCs to confirm that soil conditions are as anticipated and to observe data collected during installation. We should also observe any test installation and load testing that may be performed. Due to the moderate plasticity of portions of the subgrade soil, multiple mixing passes should be assumed.

Buried debris could be encountered in the fill during installation of rammed aggregate piers. It might be necessary to remove buried obstructions or adjust the shoring design to address buried debris found during construction. We recommend a contingency be in place in the event buried debris is encountered during installation.

9.3.4 Rammed Aggregate Piers

Spread footings supported on ground improved by rammed aggregate piers is suitable to support the proposed building. Rammed aggregate piers consist of compacted aggregate piers that reinforce and improve the soil. These systems are proprietary and designed and constructed by a specialty contractor. Rammed aggregate piers can be installed using an open-hole method or a displacement method. The maximum depth of the open-hole method is typically 20 to 22 feet BGS while the displacement method can advance to depths of up to 45 feet BGS. We recommend all rammed aggregate piers, regardless of the system, extend to native soil.

Allowable bearing pressures for foundations on rammed aggregate piers typically vary between 4,000 and 6,000 psf. Static settlement for foundations on rammed aggregate piers typical ranges between 0.5 and 1.0 inch. Static and seismic settlement should be combined for the total design settlement.

A representative of our firm should be present during installation of the rammed aggregate piers to confirm that soil conditions are as anticipated. We should also observe any test installation and load testing that may be performed.

Installing the aggregate piers may require drilling through groundwater. If groundwater is encountered, casing could be required to advance open-hole aggregate piers.

Buried debris could be encountered in the fill during installation of rammed aggregate piers. It might be necessary to remove buried obstructions or adjust the shoring design to address buried debris found during construction. We recommend a contingency be in place in the event buried debris is encountered during installation.

9.4 SLABS-ON-GRADE

If the building is constructed near current grades, undocumented fill will be present beneath the floor slab. There is a small risk for poor performance of slabs-on-grade established directly over undocumented fill. If undocumented fill is present at the proposed finished floor slab elevations, removal and replacement with imported structural fill would eliminate settlement risk from the undocumented fill. Slabs-on-grade can be constructed on undocumented fill, provided a small risk of distress is accepted (minor floor slab cracking) and subgrades are evaluated as described in the "Site Preparation" section. To reduce the potential for cracking, slabs-on-grade can incorporate additional reinforcement to span areas where differential settlement occurs.

If the finished floor slab of the building is raised to 183.6 feet, construction of floor slabs should not commence until fill-induced settlement is complete as described in the "Settlement Considerations" section. If the site is raised, an 8- to 9-foot-thick cap of structural fill will be present above the existing undocumented fill. This structural cap will reduce the potential for poor floor slab performance; however, the possibility of some floor slab distress is possible.

Satisfactory subgrade support for building slabs-on-grade of native soil or fill up to 200 psf can be obtained, provided the subgrade is prepared in accordance with the "Settlement Consideration" and "Site Preparation" sections. A modulus of reaction of 100 pci can be used for slabs-on-grade constructed on native soil, undocumented fill, or new structural fill over undocumented fill. A minimum 6-inch-thick layer of imported granular material should be placed and compacted over the prepared subgrade to assist as a capillary break. The floor slab 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 contaminated with excessive fines (greater than 5 percent by dry weight passing the U.S. Standard No. 200 sieve) should be replaced.

Vapor barriers beneath slabs-on-grade are typically required by flooring manufactures to maintain the warranty on their products. In our experience, adequate performance of floor adhesives can be achieved by using a clean base rock (less than 5 percent fines) beneath the floor slab with no vapor barrier. In fact, vapor barriers can frequently cause moisture problems by trapping water beneath the floor slab that is introduced during construction. If a vapor barrier is used, water should not be applied to the base rock prior to pouring the slab and the work should be completed during extended dry weather so that rainfall is not trapped on top of the vapor barrier.

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.

10.0 RETAINING STRUCTURES

10.1 BURIED WALLS

Buried walls above the design groundwater level should be designed using the lateral earth pressures shown on Figure 4. Buried walls that extend below the design groundwater level should be designed in accordance with Figure 7. These values do not include surcharged-induced lateral earth pressures. The values on Figure 5 can be used to compute surcharge-induced lateral earth pressures. We recommend a vertical live load of 250 psf be applied at the surface of the retained soil where the wall shoring retains roadways. The design equivalent fluid pressure should also be increased for walls that retain sloping soil in accordance with Table 3.

Permanent walls should also be designed to resist full hydrostatic pressure associated with the design groundwater elevation associated with the flood of 181.6 feet. Seismic lateral forces can be calculated using a dynamic force equal to $7.5H^2$ pounds per linear foot of wall, where H is the wall height. The seismic force should be applied as a distributed load with the centroid located at 0.6H from the wall base.

10.2 CONVENTIONAL RETAINING WALLS

Retaining walls can be designed using conventional Rankine theory. Walls not restrained from rotation can be designed using an equivalent fluid pressure of 35 pcf. The pressure should be increased to 55 pcf for walls restrained from rotation. The design pressures assume drainage is provided behind the walls to prevent hydrostatic pressures from developing. If the walls are not drained, unrestrained and restrained walls should be designed assuming an equivalent fluid pressures of 70 and 90 pcf, respectively.

Lateral forces can be resisted by frictional resistance on wall foundations and by passive earth pressures between the wall and the soil in front of the wall. A frictional coefficient of 0.3 can be used between the foundation and subgrade soil. An equivalent fluid pressure equal to 300 pcf can be used to compute the available passive resistance for footings above groundwater. If footings are below groundwater, the passive pressure should be reduced to 125 pcf. Adjacent slabs-on-grade, pavement, or the upper 12 inches of adjacent areas should not be considered when calculating passive resistance. This value assumes groundwater is below the base of the wall footing. Allowable bearing pressures for wall subgrades prepared as described in the "Site Preparation" section can be designed for an allowable bearing pressure of 2.0 ksf. This value does not include surcharged-induced lateral earth pressures. The values on Figure 5 can be used to compute surcharge-induced lateral earth pressures. We recommend a vertical live load of 250 psf be applied at the surface of the retained soil where the wall shoring retains roadways. The design equivalent fluid pressure should also be increased for walls that retain sloping soil in accordance with Table 3.

Seismic lateral forces can be calculated using a dynamic force equal to $7.5H^2$ pounds per linear foot of wall, where H is the wall height. The seismic force should be applied as a distributed load with the centroid located at 0.6H from the wall base.

10.3 RETAINING WALL BACKFILL

Backfill should be placed and compacted as recommended for structural fill, with the exception of backfill placed immediately adjacent to walls. Backfill adjacent to walls should be compacted to a lesser standard to reduce the potential for generation of excessive pressure on the walls. Backfill located within a horizontal distance of 3 feet from retaining walls should 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 (slabs, sidewalk, or pavement) will be placed adjacent to the wall, we recommend the upper 2 feet of fill be compacted to 95 percent of the maximum dry density, as determined by ASTM D1557. 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 construction, unless survey data indicates that settlement is complete prior to that time.

11.0 GROUNDWATER CONSIDERATIONS

11.1 GENERAL

Roof drains should 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. The ground surface adjacent to the building should be sloped to facilitate positive drainage away from the building.

11.2 DESIGN GROUNDWATER LEVEL

Based on groundwater measurements and our experience in the area, we recommend the design 100-year ordinary high groundwater elevation be taken as 171 feet (5 feet BGS). We understand that the flood elevation at the site is 181.6 feet. In a normal year the planned building will be below the ordinary high groundwater and special considerations will not be needed.

If the building needs to be designed for the flood elevation and the building is constructed at current grades, water will be above the planned base of the floor slab and the structure will need to be structurally designed to resist the lateral and uplift hydrostatic pressures associated with the water rising to an elevation of 181.6 feet.

11.3 INFILTRATION SYSTEMS

Infiltration testing was not completed as part of our investigation. Based on the soil and groundwater conditions at the site, on-site disposal of stormwater at the site will not be feasible.

Design of stormwater detention ponds should take into consideration shallow groundwater levels expected at the site.

12.0 SEISMIC DESIGN PARAMETERS

Seismic design criteria for the project will be based on the 2019 SOSSC and ASCE 7-16. Based result of explorations at the site the seismic site class is D.

ASCE 7-16 Section 11.4.8 requires a ground motion hazard study in accordance with Section 21.2 for structures on Site Class D sites with S_1 greater than or equal to 0.2 g (S_1 at the site is 0.407 g). Exception 2 of ASCE 7-16 Section 11.4.8 indicates a ground motion hazard study is not required for structures on Site Class D sites with S_1 greater to or equal 0.2 g, provided the value of the seismic response coefficient C_S is determined by Eq. (12.8-2) for values of $T \leq 1.5T_s$ and taken as equal to 1.5 times the value computed in accordance with either Eq. (12.8-3) for $T_L \geq T > 1.5T_s$ or Eq. (12.8-4) for $T > T_L$. We anticipate the building will meet these requirements, but if Exception 2 is not applicable, a ground motion hazard analysis will be required. We recommend the structural engineer evaluate these requirements and exceptions to determine if the parameters for Site Class D provided in Table 5 can be used for design or if a site-specific seismic hazard evaluation is required.

Table 5. Seismic Design Parameters*

Seismic Design Parameter	Short Period	1 second Period
MCE Spectral Acceleration	S_s = 0.886 g	S_1 = 0.407 g
Site Class	D	
Site Coefficient	F_a = 1.145	F_v = 1.893
Adjusted Spectral Acceleration	S_{MS} = 1.015 g	S_{M1} = 0.770 g
Design Spectral Response Acceleration Parameters	S_{DS} = 0.677 g	S_{D1} = 0.513 g

* The structural engineer should evaluate code requirements and exceptions to determine if these parameters can be used for design.

It should be noted that the 2019 SOSSC requires a site-specific seismic hazard evaluation for structures with more than six above-grade stories. If development plans change and seven or more above-grade floors are planned, we should be contacted to complete a site-specific seismic hazard evaluation.

13.0 PAVEMENT

13.1 GENERAL

AC pavement will be installed as part of the development. Pavement should be installed on native subgrade or new engineered fills prepared in conformance with the "Site Preparation" and "Structural Fill" sections. Our pavement recommendations are based on the following assumptions:

- The top 12 inches of soil subgrade is compacted to at least 92 percent of its maximum dry density, as determined by ASTM D1557, or until proof rolling with heavy equipment indicates that is it firm and unyielding.
- A resilient modulus of 4,500 psi for the subgrade soil is assumed for the base rock.
- The design manual provided for the project specifies pavement recommendations based on a design life of 20 years.
- Initial and terminal serviceability indices of 4.2 and 2.5, respectively.
- Reliability of 85 percent and standard deviation of 0.45.
- Fire access will consist of an imposed fire apparatus load of 75,000 pounds on an infrequent basis.
- No growth.

We have assumed that traffic will generally consist of passenger cars in parking areas and up to five garbage/large delivery/large moving trucks per day.

13.2 AC PAVEMENT

Our AC pavement design recommendations are presented in Table 6.

Table 6. 20-Year Standard AC Pavement Sections

Traffic Levels	Trucks per Day[1]	AC (inches)	Base Rock (inches)
Car Parking Stall	0	2.5	7
Drive Aisles	Up to 5	3	10

1. One-way truck ADT

If the subgrade is cement amended to the thicknesses indicated below and the amended soil achieves a seven-day unconfined compressive strength of at least 100 psi, the pavement can be constructed as recommended in Table 7.

Table 7. 20-Year AC Pavement Sections with Cement Amendment

Traffic Levels	Trucks per Day[1]	AC (inches)	Base Rock (inches)	Cement Amendment[2] (inches)
Car Parking Stall	0	2.5	4	12
Drive Aisles	Up to 5	3	4	12

1. One-way truck ADT
2. Assumes a minimum seven-day unconfined compressive strength of 100 psi.

The aggregate base should meet the requirements outlined in the "Structural Fill" section. The AC should be Level 2, ½-inch, dense ACP in the parking areas and Level 3, ½-inch, dense ACP in the truck areas according to OSSC 00744 (Asphalt Concrete Pavement). The AC should be compacted to 92 percent of the theoretical maximum density of the mix, as determined by AASHTO T 209. The minimum and maximum lift thicknesses are 2 and 3 inches, respectively, for ½-inch ACP. Asphalt binder should be performance graded and conform to PG 64-22. The binder grade should be adjusted depending on the aggregate gradation and amount of recycled asphalt pavement and/or recycled asphalt shingles in the contractor's mix design submittal.

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

Each AC mix design has a recommended compaction temperature range that is specific for the particular AC binder used. In colder temperatures, it is more difficult to maintain the temperature of the AC mix as it can lose heat while stored in the delivery truck, as it is placed, and in the time between placement and compaction. In Oregon, the AC surface temperature during paving should be at least 40 degrees Fahrenheit for lift thickness greater than 2.5 inches and at least 50 degrees Fahrenheit for lift thickness between 2 and 2.5 inches.

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

13.3 CONSTRUCTION CONSIDERATIONS

All thicknesses are intended to be the minimum acceptable. Design of the recommended pavement section assumes that construction will be completed during an extended period of dry weather. Wet weather construction could require an increased thickness of aggregate base. In addition, to prevent strength loss during curing, cement-amended soil should be allowed to cure for at least four days prior to construction traffic or placing the aggregate base. Lastly, the amended subgrade should be protected with a minimum of 4 inches of aggregate base prior to construction traffic access.

Construction traffic should be limited to non-building, unpaved 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. The aggregate base and cement-amended thicknesses (if installed) do not account for construction traffic, and haul roads and staging areas should be used as described in the "Site Preparation" section.

14.0 PERMANENT SLOPES

Permanent cut and fill slopes should not exceed 2H:1V. 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.

15.0 OBSERVATION OF CONSTRUCTION

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

16.0 LIMITATIONS

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

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

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

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

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

♦ ♦ ♦

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

Sincerely,

NV5

Nick Paveglio, P.E.
Principal Engineer

REGISTERED PROFESSIONAL ENGINEER
86690
OREGON
May 8, 2012
NICHOLAS N. PAVEGLIO
EXPIRES: 12/31/22

REFERENCES

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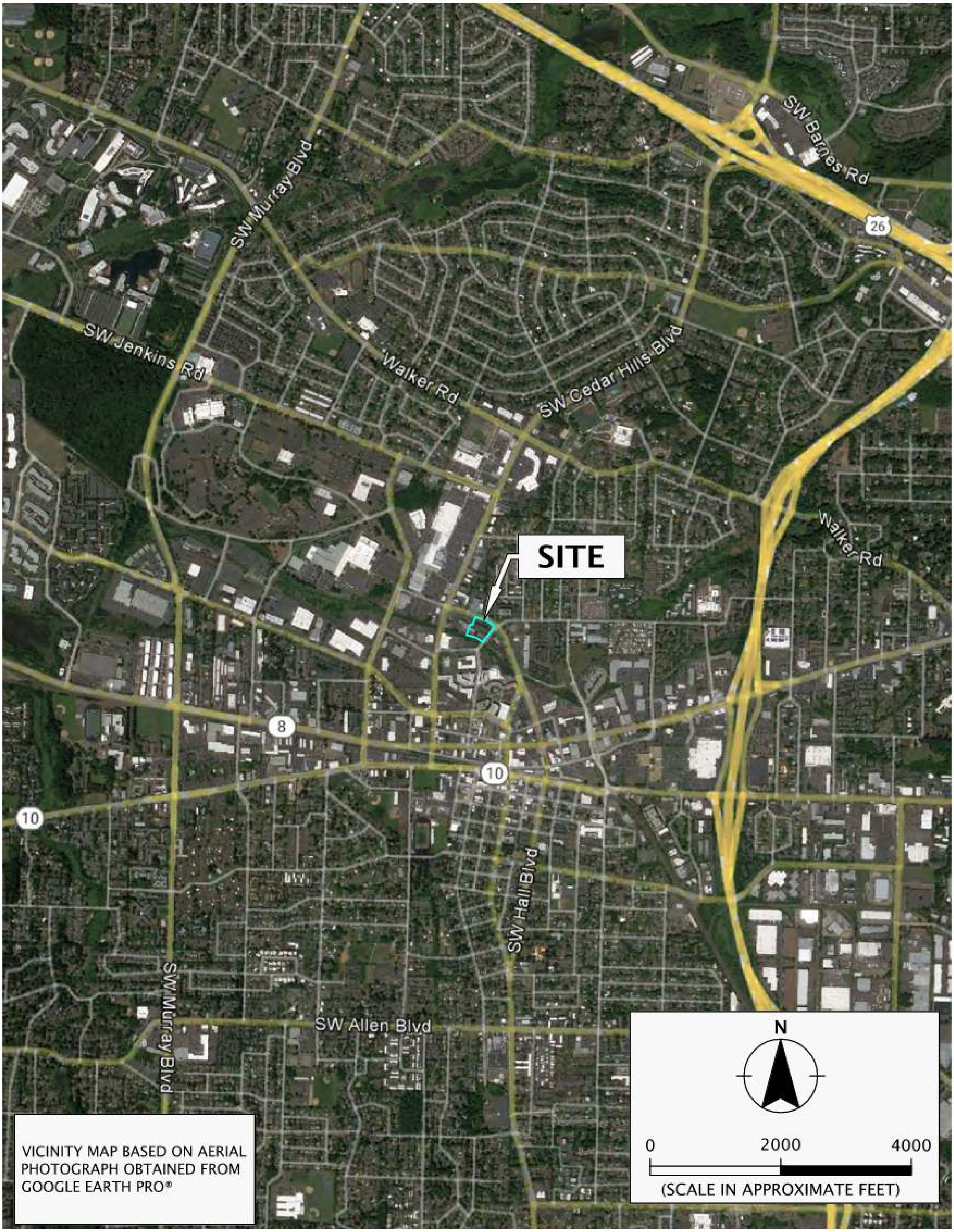
FIGURES



CEDARST-1-01	VICINITY MAP	
AUGUST 2022	WESTGATE + HALL DEVELOPMENT BEAVERTON, OR	FIGURE 1

LEGEND:

	SITE BOUNDARY
B-1	BORING
CPT-1	CPT
(7)	FILL THICKNESS (FEET BGS)
[15.2]	GROUNDWATER DEPTH (FEET BGS)
B-1	PRIOR BORING (GEODESIGN, 2011)

N

0 50 100

(SCALE IN FEET)

SITE PLAN BASED ON AERIAL PHOTOGRAPH DATED JUNE 17, 2021, OBTAINED FROM GOOGLE EARTH PRO.

NV5

CEDARST-1-01

AUGUST 2022

SITE PLAN - EXISTING CONDITIONS

WESTGATE + HALL DEVELOPMENT
BEAVERTON, OR

FIGURE 2

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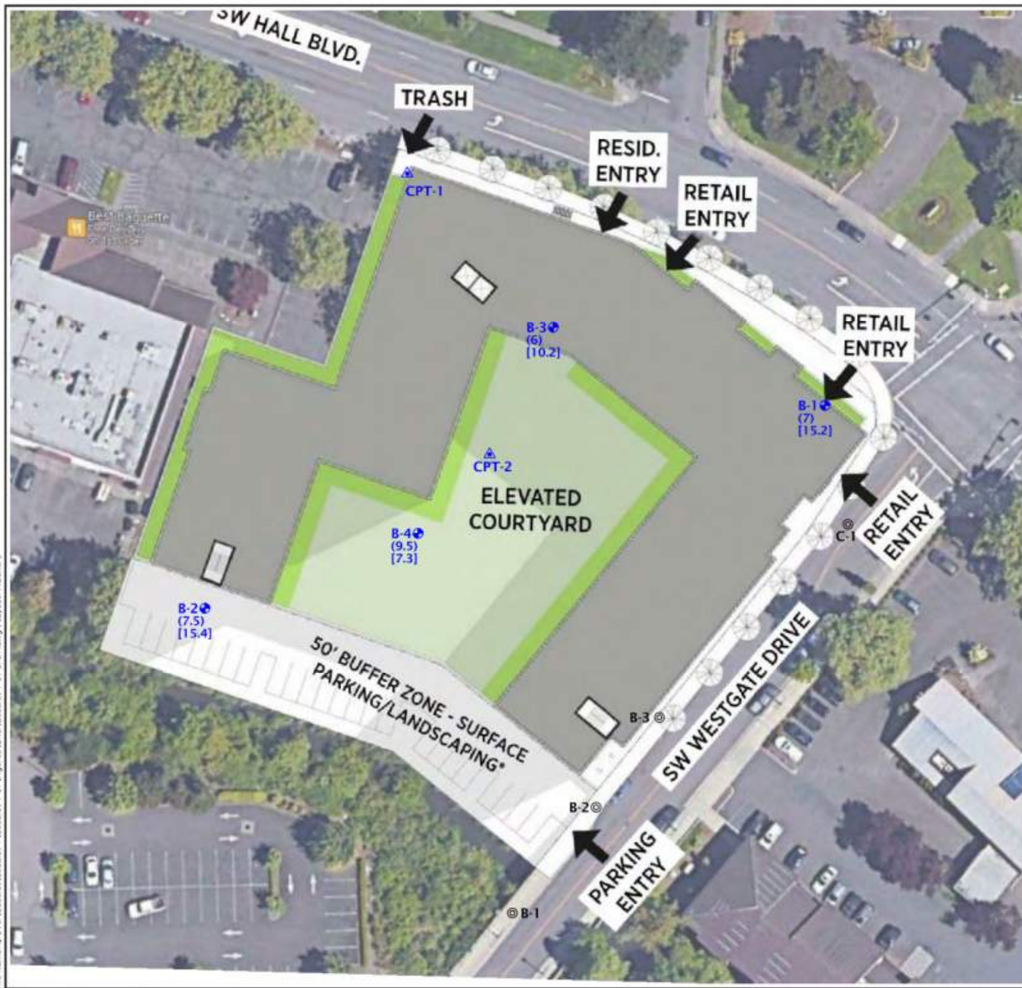
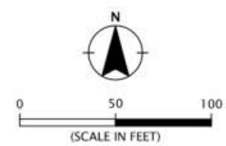
File Name: J:\A-D\CedarSt\CedarSt-1\CedarSt-1-01\Figures\CAD\CedarSt-1-01-SP01.dwg | Layout: FIGURE 2



LEGEND:

— - - —	SITE BOUNDARY
B-1	BORING
CPT-1	CPT
(7)	FILL THICKNESS (FEET BGS)
[15.2]	GROUNDWATER DEPTH (FEET BGS)
B-1	PRIOR BORING (GEODESIGN, 2011)



SITE PLAN BASED ON IMAGE OF SHEET 4 SITE PLAN DATED JULY 26, 2022, PREPARED BY ANKROM MOISAN ARCHITECTS, INC.

NV5

CEDARST-1-01

AUGUST 2022

SITE PLAN - PROPOSED CONDITIONS

WESTGATE + HALL DEVELOPMENT
BEAVERTON, OR

FIGURE 3

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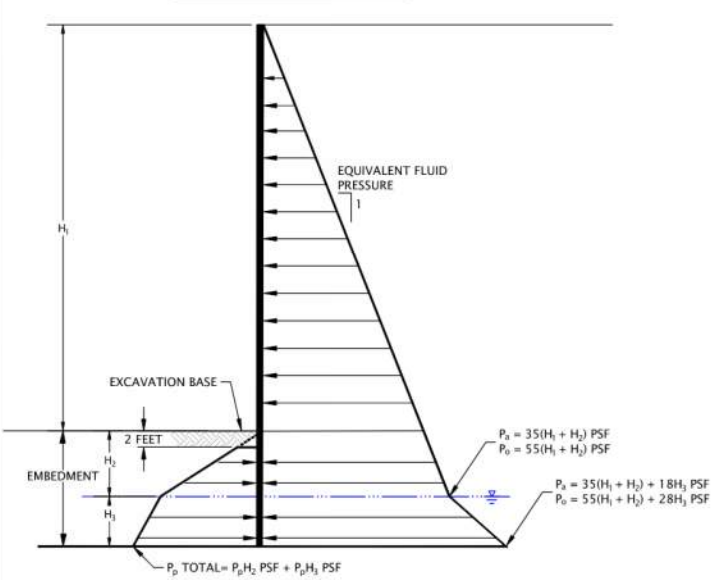
RECOMMENDED DESIGN PARAMETERS FOR TEMPORARY CANTILEVERED WALL



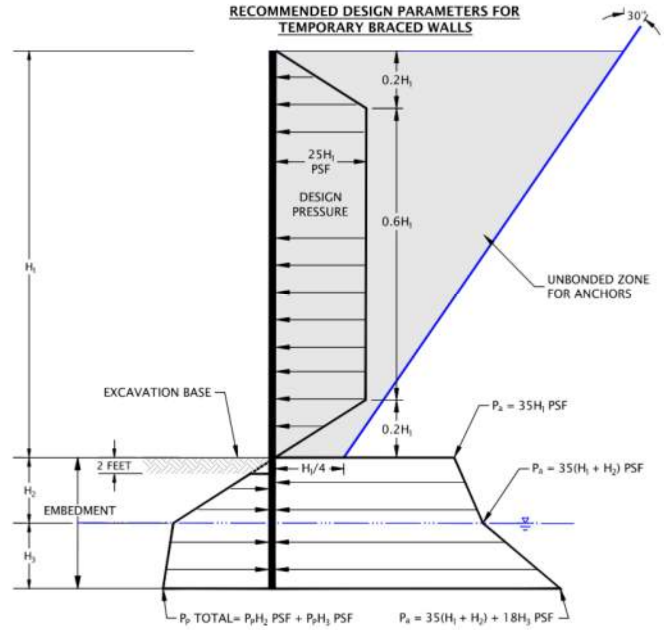
EXPLANATION:

P_a = ACTIVE EQUIVALENT FLUID PRESSURE (ASSUMES WALL IS FREE TO ROTATE)
P_o = AT-REST EQUIVALENT FLUID PRESSURE (ASSUMES WALL IS RESTRAINED FROM ROTATION)
P_p = 300 PCF IN FILL, SILT, AND CLAY ABOVE GROUNDWATER
125 PCF IN FILL, SILT, AND CLAY BELOW GROUNDWATER
H_1 = EXPOSED WALL HEIGHT (FEET)
H_2 = EMBEDMENT DEPTH ABOVE GROUNDWATER (FEET)
H_3 = EMBEDMENT DEPTH BELOW GROUNDWATER (FEET)
PASSIVE PRESSURE (P_p) ACTS OVER 2X THE PILE WIDTH IN FILL, SILT, AND CLAY.
ACTIVE PRESSURE (P_a) AND AT-REST PRESSURE (P_o) ACT OVER 1X THE PILE WIDTH BELOW THE EXCAVATION BASE.

NOTES:

1. DOES NOT INCLUDE SURCHARGE OR SEISMIC LOADS.
2. RETAINED SOIL IS ASSUMED TO BE LEVEL.
3. THE LATERAL EARTH PRESSURES ARE UNFACTORED.
4. PASSIVE PRESSURE RESISTANCE SHOULD BE NEGLECTED 2 FEET BELOW THE BOTTOM OF THE EXCAVATION.
5. FIGURE SHOULD BE USED IN CONJUNCTION WITH REPORT TEXT.
6. LATERAL EARTH PRESSURES ASSUME WATER WILL BE MAINTAINED BELOW THE BASE OF THE EXCAVATION.

RECOMMENDED DESIGN PARAMETERS FOR TEMPORARY BRACED WALLS



EXPLANATION:

P_a = ACTIVE EQUIVALENT FLUID PRESSURE
P_p = 300 PCF IN FILL, SILT, AND CLAY ABOVE GROUNDWATER
125 PCF IN FILL, SILT, AND CLAY BELOW GROUNDWATER
H_1 = EXPOSED WALL HEIGHT (FEET)
H_2 = EMBEDMENT DEPTH ABOVE GROUNDWATER (FEET)
H_3 = EMBEDMENT DEPTH BELOW GROUNDWATER (FEET)
PASSIVE PRESSURE (P_p) ACTS OVER 2X THE PILE WIDTH IN FILL, SILT, AND CLAY.
ACTIVE PRESSURE (P_a) ACTS OVER 1X THE PILE WIDTH BELOW THE EXCAVATION BASE.

NOTES:

1. DOES NOT INCLUDE SURCHARGE OR SEISMIC LOADS.
2. TIEBACKS SHOULD BE LOCKED OFF AT 100 PERCENT OF DESIGN LOAD.
3. THE LATERAL EARTH PRESSURES ARE UNFACTORED.
4. PASSIVE PRESSURE RESISTANCE SHOULD BE NEGLECTED 2 FEET BELOW THE BOTTOM OF THE EXCAVATION.
5. RETAINED SOIL IS ASSUMED TO BE LEVEL.
6. FIGURE SHOULD BE USED IN CONJUNCTION WITH REPORT TEXT.
7. LATERAL EARTH PRESSURES ASSUME WATER WILL BE MAINTAINED BELOW THE BASE OF THE EXCAVATION.

LATERAL EARTH PRESSURES
FOR TEMPORARY CANTILEVERED AND BRACED WALLS

WESTGATE + HALL DEVELOPMENT
BEAVERTON, OR

FIGURE 4

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NV5

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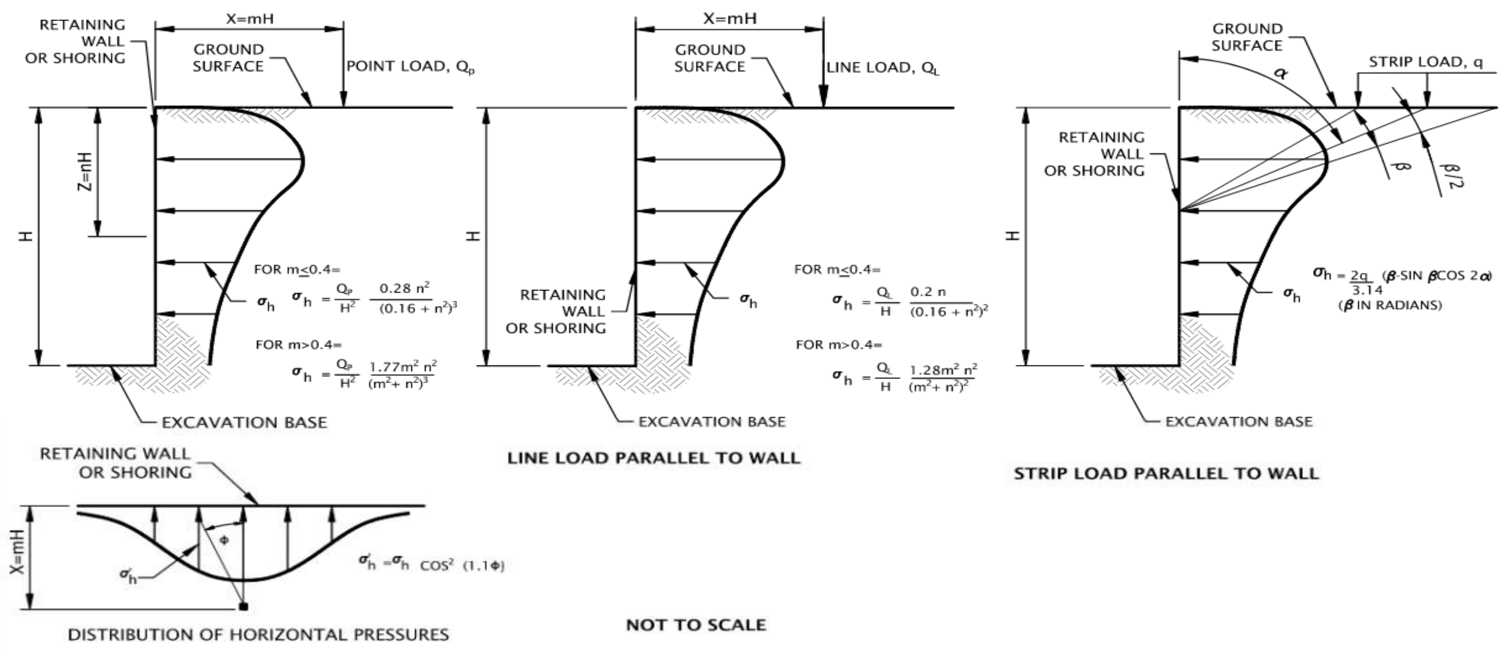
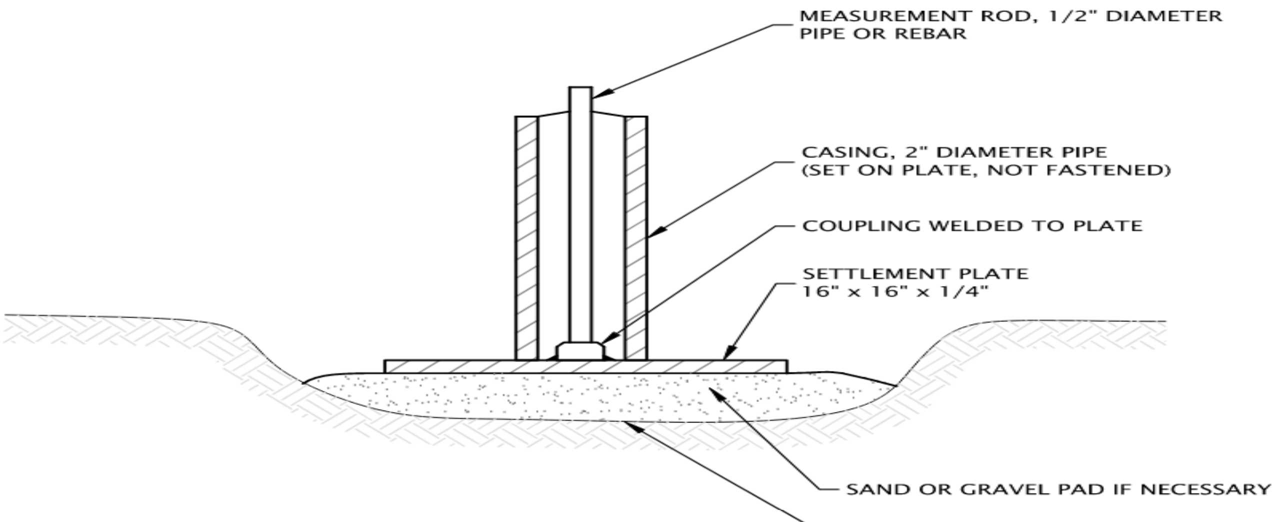


VERTICAL POINT LOAD

NOTES:

1. FIGURE SHOULD BE USED IN CONJUNCTION WITH REPORT TEXT.
2. THESE GUIDELINES APPLY TO RIGID WALLS WITH POISSON'S RATIO ASSUMED TO BE 0.5 FOR BACKFILL MATERIALS.
3. LATERAL PRESSURES FROM ANY COMBINATION OF ABOVE LOADS MAY BE DETERMINED BY THE PRINCIPLE OF SUPERPOSITION.

NV5	CEDARST-1-01	**SURCHARGE-INDUCED LATERAL EARTH PRESSURES**	
	AUGUST 2022	WESTGATE + HALL DEVELOPMENT BEAVERTON, OR	**FIGURE 5**



NOT TO SCALE

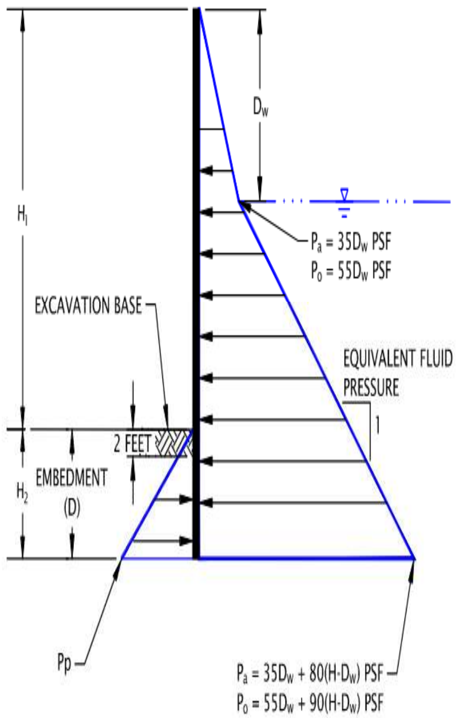
NOTES:

1. INSTALL MARKERS ON FIRM GROUND OR ON SAND OR GRAVEL PADS IF NEEDED FOR STABILITY. TAKE INITIAL READING ON TOP OF ROD AND AT ADJACENT GROUND LEVEL PRIOR TO PLACING ANY FILL.
2. FOR EASE IN HANDLING, ROD AND CASING ARE USUALLY INSTALLED IN 5-FOOT SECTIONS. AS FILL PROGRESSES, COUPLINGS ARE USED TO INSTALL ADDITIONAL LENGTHS. CONTINUITY IS MAINTAINED BY READING THE TOP OF THE MEASUREMENT ROD, THEN IMMEDIATELY ADDING THE NEW SECTION AND READING THE TOP OF THE ADDED ROD. BOTH READINGS ARE RECORDED.
3. RECORD THE ELEVATION OF THE TOP OF THE MEASUREMENT ROD IN EACH MARKER AT THE RECOMMENDED TIME INTERVALS. EACH TIME, NOTE THE ELEVATION OF THE ADJACENT FILL SURFACE.
4. READ THE MARKER TO THE NEAREST 0.01 FOOT, OR 0.005 FOOT IF POSSIBLE. NOTE THE FILL ELEVATION TO THE NEAREST 0.1 FOOT.
5. THE ELEVATIONS SHOULD BE REFERENCED TO A TEMPORARY BENCHMARK LOCATED ON STABLE GROUND AT LEAST 500 FEET FROM THE EMBANKMENT.

NV5	CEDARST-1-01	**SETTLEMENT PLATE DETAIL**	
	AUGUST 2022	WESTGATE + HALL DEVELOPMENT BEAVERTON, OR	**FIGURE 6**

Printed By: mmiller | Print Date: 8/29/2022 8:52:10 AM
File Name: J:\A-D\CedarSt\CedarSt-1\CedarSt-1-01\Figures\CAD\CedarSt-1-01-DET04.dwg | Layout: FIGURE 6

RECOMMENDED DESIGN PARAMETERS FOR PERMANENT CANTILEVERED WALLS WITH HYDROSTATIC PRESSURES



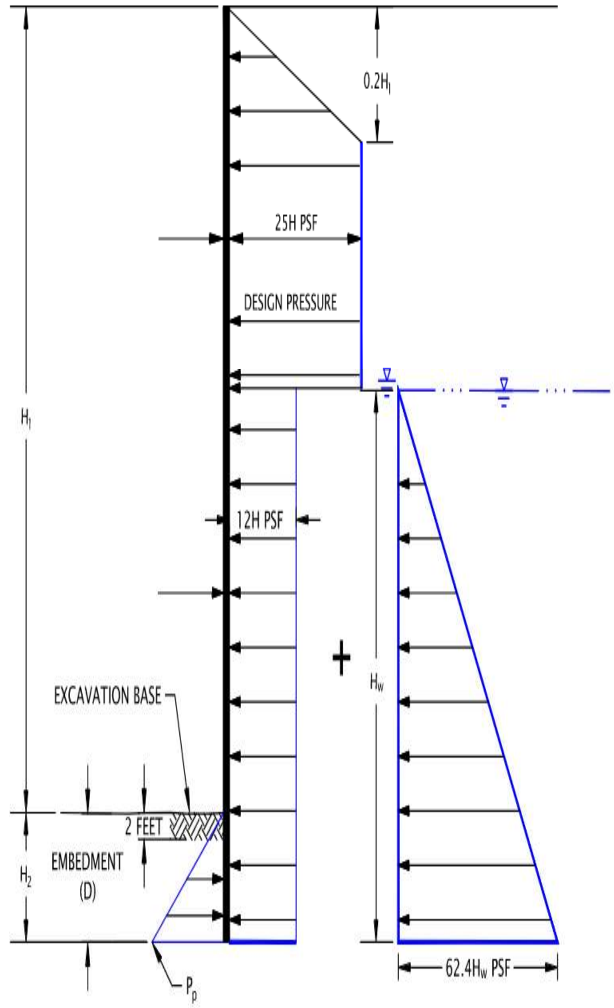
EXPLANATION:

P_a = ACTIVE EQUIVALENT FLUID PRESSURE (ASSUMES WALL IS FREE TO ROTATE)
P_a = 35 PCF (ABOVE GROUNDWATER) AND 80 PCF (BELOW GROUNDWATER)
P_0 = AT-REST EQUIVALENT FLUID PRESSURE (ASSUMES WALL IS RESTRAINED FROM ROTATION)
P_0 = 55 PCF (ABOVE GROUNDWATER) AND 90 PCF (BELOW GROUNDWATER)
P_p = 300 PCF IN FILL, SILT, AND CLAY ABOVE GROUNDWATER
125 PCF IN FILL, SILT, AND CLAY BELOW GROUNDWATER
H = EFFECTIVE WALL HEIGHT (FEET)
D = WALL EMBEDMENT DEPTH (FEET)
D_w = DEPTH TO GROUNDWATER (FEET)

NOTES:

1. DOES NOT INCLUDE SURCHARGE OR SEISMIC LOADS.
2. RETAINED SOIL IS ASSUMED TO BE LEVEL.
3. THE LATERAL EARTH PRESSURES ARE UNFACTORED.
4. PASSIVE PRESSURE RESISTANCE SHOULD BE NEGLECTED 2 FEET BELOW THE BOTTOM OF THE EXCAVATION.
5. FIGURE SHOULD BE USED IN CONJUNCTION WITH REPORT TEXT.
6. HYDROSTATIC PRESSURE CAUSED BY THE STATIC GROUNDWATER TABLE ON THE ACTIVE PRESSURE SIDE SHOULD BE INCLUDED IN FINAL DESIGN. CONSTRUCTION DEWATERING EXTERNAL TO THE EXCAVATION MAY BE REQUIRED TO REMOVE PERCHED GROUNDWATER BEFORE THE EXCAVATION IS ADVANCED.

RECOMMENDED DESIGN PARAMETERS FOR PERMANENT BRACED WALLS WITH HYDROSTATIC PRESSURES



EXPLANATION:

P_p = 300 PCF IN FILL, SILT, AND CLAY ABOVE GROUNDWATER
125 PCF IN FILL, SILT, AND CLAY BELOW GROUNDWATER
H_w = HEIGHT OF WATER (FEET)
H = EFFECTIVE WALL HEIGHT (FEET)
D = WALL EMBEDMENT DEPTH (FEET)

NOTES:

1. FIGURE SHOULD BE USED IN CONJUNCTION WITH REPORT TEXT.
2. ASSUMES WALL IS INTERNALLY BRACED AT MORE THAN ONE LEVEL.
3. HYDROSTATIC PRESSURE CAUSED BY THE STATIC GROUNDWATER TABLE ON THE ACTIVE PRESSURE SIDE SHOULD BE INCLUDED IN FINAL DESIGN. CONSTRUCTION DEWATERING EXTERNAL TO THE EXCAVATION MAY BE REQUIRED TO REMOVE PERCHED GROUNDWATER BEFORE THE EXCAVATION IS ADVANCED.
4. THE LATERAL EARTH PRESSURES ARE UNFACTORED.
5. VALUES DO NOT INCLUDE SEISMIC LOADS OR SURCHARGE LOADS.
6. RETAINED SOIL IS ASSUMED TO BE LEVEL.
7. PASSIVE PRESSURE RESISTANCE SHOULD BE NEGLECTED 2 FEET BELOW THE BOTTOM OF THE EXCAVATION.

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CEDARST-1-01
AUGUST 2022

LATERAL EARTH PRESSURES
FOR PERMANENT CANTILEVERED AND BRACED WALLS
WESTGATE + HALL DEVELOPMENT
BEAVERTON, OR

FIGURE 7

APPENDIX A

APPENDIX A

FIELD EXPLORATIONS

GENERAL

Subsurface conditions at the site were explored by drilling four borings (B-1 through B-4) to depths between 11.5 and 51.5 feet BGS and advancing two CPT probes (CPT-1 and CPT-2) to depths between approximately 58.5 and 60 feet BGS. Description of the CPTs are included in Appendix B.

The borings were drilled by Dan J. Fischer Excavating, Inc. using a trailer-mounted drill rig and solid-stem auger techniques. The exploration logs are presented in this appendix. Locations of the explorations shown on Figures 2 and 3.

The explorations were located in the field by pacing from existing site features. This information should be considered accurate only to the degree implied by the methods used. A member of our geotechnical staff observed the explorations. Representative samples of the various soils encountered were collected in the exploration for geotechnical laboratory testing.

SOIL SAMPLING

We collected representative samples of the various soil encountered in the explorations for geotechnical laboratory testing. Samples were collected from the borings using 1½-inch-inner diameter split-spoon (SPT) samplers in general accordance with ASTM D1586. The samplers were driven into the soil with a 140-pound automatic trip hammer free falling 30 inches. The sampler was driven a total distance of 18 inches. The number of blows required to drive the sampler the final 12 inches is recorded on the exploration logs, unless otherwise noted. Higher quality, relatively undisturbed samples were collected using a standard Shelby tube in general accordance with ASTM D1587. Sampling methods and intervals are shown on the exploration logs.

The hammer used to conduct the SPTs was lifted using a rope and cathead system. The hammer was raised using two wraps of the rope around the cathead to conduct the SPTs.

SOIL CLASSIFICATION

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

LABORATORY TESTING

CLASSIFICATION

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

ATTERBERG LIMITS

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

CONSOLIDATION TESTING

One-dimensional consolidation testing was completed on a select relatively undisturbed soil sample in general accordance with ASTM D2435. The test results are presented in this appendix.

DRY DENSITY

We tested the in-situ dry density of a select soil sample in general accordance with ASTM D2937. The test results are presented in this appendix.

MOISTURE CONTENT

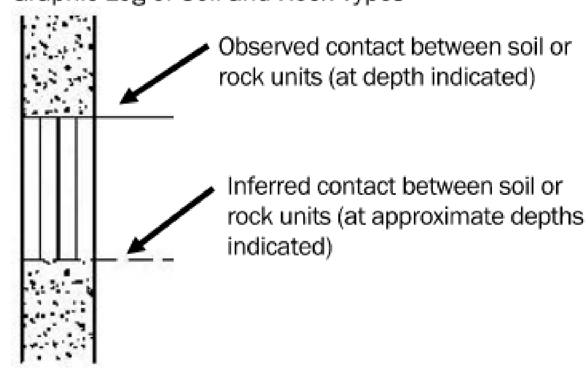
We determined the natural moisture content of select soil samples in general accordance with ASTM D2216. The test results are presented in this appendix.

PARTICLE-SIZE ANALYSIS

Particle-size analysis was performed on select soil samples in general accordance with ASTM D1140. 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 %

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	BLOW COUNT	MOISTURE CONTENT %	INSTALLATION AND COMMENTS
0.0	ASPHALT CONCRETE (3.5 inches).	0.3				
	AGGREGATE BASE (4.5 inches).	0.7				
	Stiff, brown-gray SILT with debris (wood) (ML), minor sand and gravel; moist - **FILL**.					
2.5				11		
5.0	soft to medium stiff, trace sand and gravel at 5.0 feet wood at 6.0 feet			4		
7.5	Medium stiff, dark gray CLAY (CL), trace sand and organics (rootlets); moist, organics are less than 1/4 inch in diameter.	7.0		7		
10.0	stiff at 10.0 feet			10		
12.5			CON		●	P
15.0	soft to medium stiff; moist to wet at 15.0 feet		ATT	4	●	LL = 45% PL = 20% 15.2 feet, during drilling
17.5						
	Stiff, gray SILT (ML), trace to minor sand; wet.	18.0				
20.0			P200	9	●	P200 = 83%
22.5						
25.0	medium stiff, some clay, trace sand at 25.0 feet		ATT	7	●	LL = 33% PL = 25%
27.5						
30.0						

Legend: ▲ BLOW COUNT; ● MOISTURE CONTENT %; RQD%; CORE REC% (scale 0 – 50 – 100)

DRILLED BY: Dan J. Fischer Excavating, Inc.

LOGGED BY: Z. Rogers

COMPLETED: 08/05/22

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

BORING BIT DIAMETER: 4 inches

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SEPTEMBER 2022

BORING B-1

WESTGATE + HALL DEVELOPMENT
BEAVERTON, OR

FIGURE A-1

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	BLOW COUNT	INSTALLATION AND COMMENTS
30.0	very stiff; laminated layers of sand at 30.0 feet		P200	20	P200 = 84%
32.5					
35.0	without layering at 35.0 feet		P200	20	P200 = 89%
37.5					
	Soft, gray CLAY (CL), some silt, trace sand; wet, clay has low to medium plasticity.	38.0			
40.0				3	
42.5				P	
	Very stiff to hard, brown-gray, sandy CLAY (CL); wet.	44.0	P200	30	P200 = 56%
45.0					
47.5					
50.0	very stiff, light gray, with sand at 50.0 feet			28	
	Exploration completed at a depth of 51.5 feet. SPT completed using two wraps with a cathead.	51.5			Surface elevation was not measured at the time of exploration.
52.5					
55.0					
57.5					
60.0					

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

DRILLED BY: Dan J. Fischer Excavating, Inc. LOGGED BY: Z. Rogers COMPLETED: 08/05/22

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

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BORING B-1
(continued)

WESTGATE + HALL DEVELOPMENT
BEAVERTON, OR

FIGURE A-1

BORING LOG - NV5 - 1 PER PAGE CEDARST-1-01-B1_4.GPJ GDI_NV5.GDT PRINT DATE: 8/29/22:KT:SN:KT

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	BLOW COUNT	MOISTURE CONTENT %	INSTALLATION AND COMMENTS
0.0	Stiff, brown-red-gray SILT with sand (ML), trace gravel; moist (2-inch-thick root zone) - **FILL**.					
2.5				9		
5.0	brown-gray, minor gravel at 5.0 feet			10		
7.5	Stiff, dark gray-dark brown with orange mottled CLAY (CH); moist, clay has high plasticity.	7.5	ATT	12		LL = 75% PL = 30%
10.0	medium stiff to stiff, dark gray at 10.0 feet			8		
15.0	medium stiff, gray, trace sand at 15.0 feet		P200	7		P200 = 78%
18.5	Stiff, gray SILT (ML), trace sand; wet.	18.5		14		
23.0	Very stiff, gray CLAY (CL), some silt, trace sand and gravel; wet, clay has low plasticity.	23.0				
25.0			P200	22		P200 = 92%
30.0						

▽ 15.4 feet, during drilling

DRILLED BY: Dan J. Fischer Excavating, Inc.   LOGGED BY: Z. Rogers   COMPLETED: 08/05/22

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

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	▲ BLOW COUNT ● MOISTURE CONTENT % RQD% CORE REC% (0 – 50 – 100)	INSTALLATION AND COMMENTS
30.0		trace to minor sand, without gravel at 30.0 feet				17	
32.5							
35.0		trace sand and organics (roots); organics are less than 1/8 inch in diameter at 35.0 feet		P200		27	P200 = 98%
37.5							
40.0		medium stiff, without organics at 40.0 feet				7	
42.5							
45.0		very stiff, with sand; moist to wet at 45.0 feet moist at 46.0 feet		P200		19	P200 = 80%
47.5							
50.0		hard, brown with gray mottles, minor sand at 50.0 feet				36	
52.5		Exploration completed at a depth of 51.5 feet. SPT completed using two wraps with a cathead.	51.5				Surface elevation was not measured at the time of exploration.
55.0							
57.5							
60.0							

DRILLED BY: Dan J. Fischer Excavating, Inc. LOGGED BY: Z. Rogers COMPLETED: 08/05/22

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

NV5 | CEDARST-1-01 | SEPTEMBER 2022

BORING B-2
(continued)

WESTGATE + HALL DEVELOPMENT
BEAVERTON, OR

FIGURE A-2

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	▲ BLOW COUNT ● MOISTURE CONTENT % RQD% CORE REC% (0 50 100)	INSTALLATION AND COMMENTS
0.0	ASPHALT CONCRETE (3.0 inches).	0.3			
	AGGREGATE BASE (3.0 inches).	0.5			
	Hard, brown SILT with sand and debris (asphalt concrete) (ML), minor gravel; moist - **FILL**.				
2.5				6-50/5"	
5.0	stiff, without debris at 5.0 feet			14	
	Medium stiff, gray-brown CLAY (CL); moist.	6.0			
7.5			P200	7	P200 = 88%
10.0	stiff at 10.0 feet			9	10.2 feet, during drilling
	Exploration completed at a depth of 11.5 feet.	11.5			Surface elevation was not measured at the time of exploration.
12.5	SPT completed using two wraps with a cathead.				
15.0					
17.5					
20.0					
22.5					
25.0					
27.5					
30.0					

DRILLED BY: Dan J. Fischer Excavating, Inc. LOGGED BY: Z. Rogers COMPLETED: 08/05/22

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

NV5

CEDARST-1-01

SEPTEMBER 2022

BORING B-3

WESTGATE + HALL DEVELOPMENT
BEAVERTON, OR

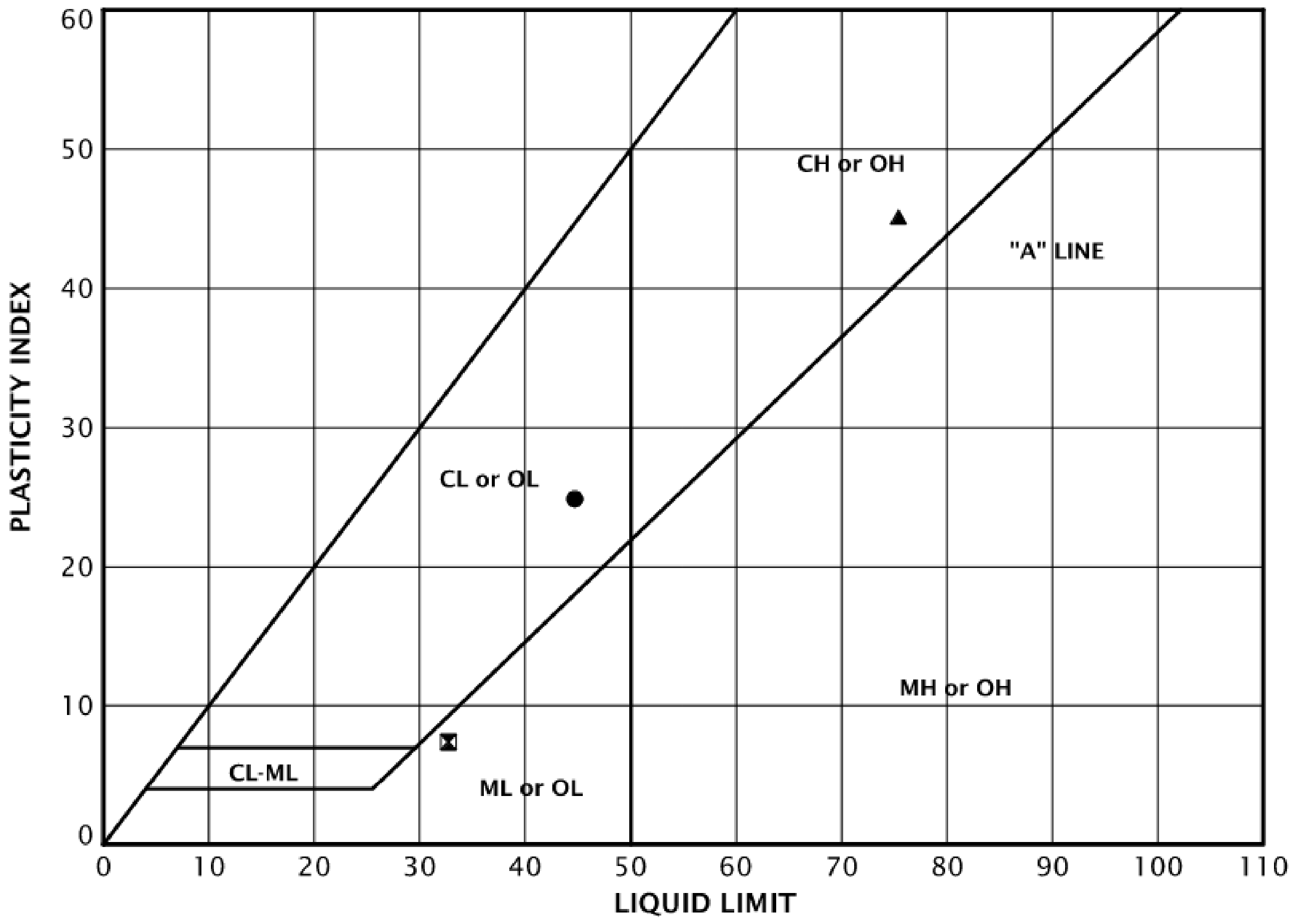
FIGURE A-3

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	▲ BLOW COUNT ● MOISTURE CONTENT % RQD% CORE REC% (0 – 50 – 100)	INSTALLATION AND COMMENTS
0.0		Medium stiff, brown-light gray SILT (ML), minor sand; moist (2-inch-thick root zone) - **FILL**.					
2.5						5	
5.0		brown with orange and gray mottles, trace sand at 5.0 feet				8	7.3 feet, during drilling
7.5		stiff, gray, with debris (asphalt concrete, concrete), minor sand at 7.5 feet				9	
10.0		Stiff, dark gray CLAY (CL); moist, clay has medium to high plasticity.	9.5			9	
12.5		Exploration completed at a depth of 11.5 feet. SPT completed using two wraps with a cathead.	11.5				Surface elevation was not measured at the time of exploration.
15.0							
17.5							
20.0							
22.5							
25.0							
27.5							
30.0							

DRILLED BY: Dan J. Fischer Excavating, Inc. LOGGED BY: Z. Rogers COMPLETED: 08/05/22

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

NV5	CEDARST-1-01	BORING B-4	
	SEPTEMBER 2022	WESTGATE + HALL DEVELOPMENT BEAVERTON, OR	FIGURE A-4



KEY	EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	MOISTURE CONTENT (PERCENT)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
●	B-1	15.0	33	45	20	25
⧗	B-1	25.0	33	33	25	8
▲	B-2	7.5	46	75	30	45

NV5

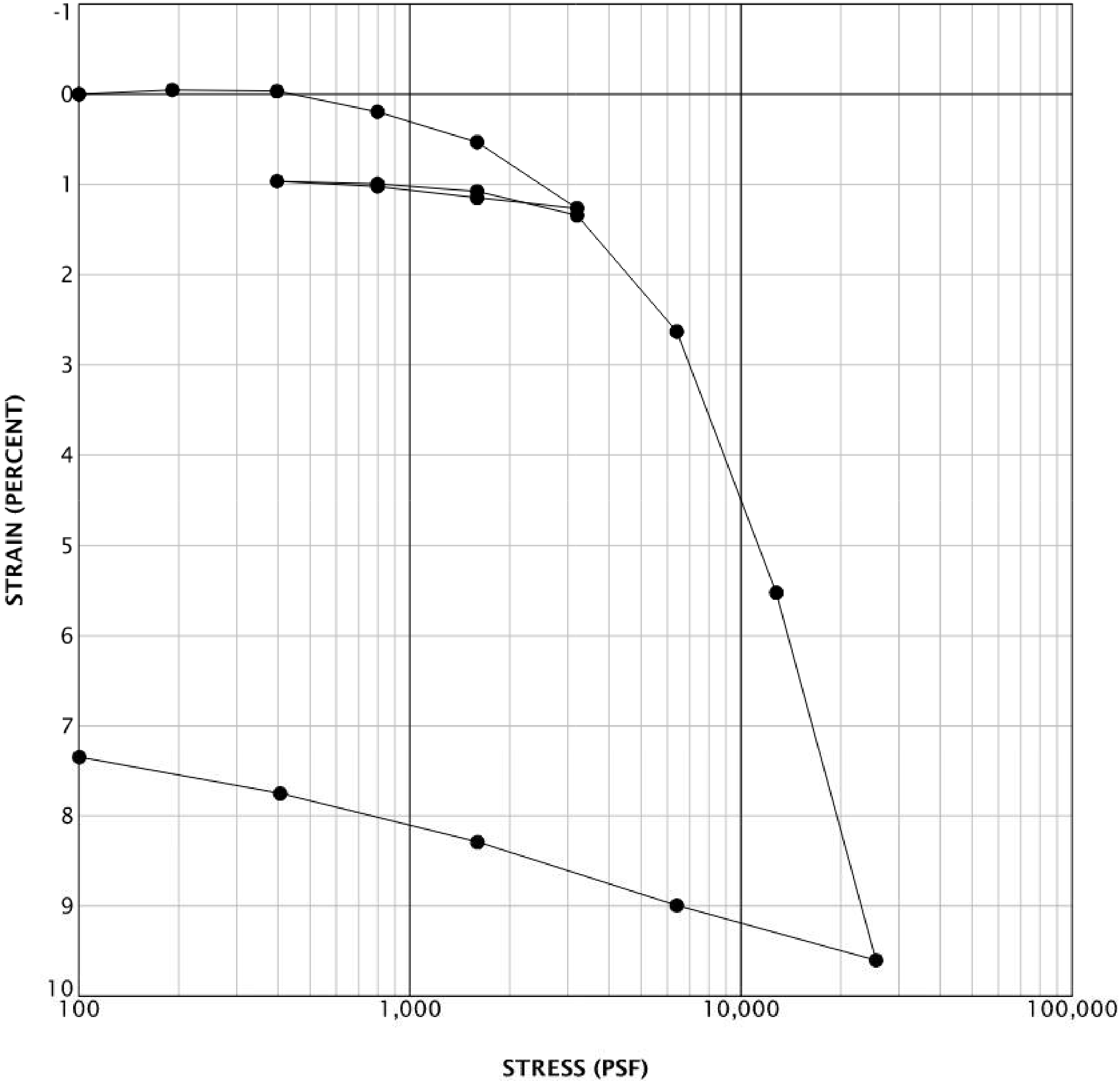
CEDARST-1-01

SEPTEMBER 2022

ATTERBERG LIMITS TEST RESULTS

WESTGATE + HALL DEVELOPMENT
BEAVERTON, OR

FIGURE A-5



KEY	EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	MOISTURE CONTENT (PERCENT)	DRY DENSITY (PCF)
●	B-1	13.0	30	92

NV5

CEDARST-1-01

SEPTEMBER 2022

CONSOLIDATION TEST RESULTS

WESTGATE + HALL DEVELOPMENT
BEAVERTON, OR

FIGURE A-6

SAMPLE INFORMATION			MOISTURE CONTENT (PERCENT)	DRY DENSITY (PCF)	SIEVE			ATTERBERG LIMITS		
EXPLORATION NUMBER	SAMPLE DEPTH (FEET)	ELEVATION (FEET)			GRAVEL (PERCENT)	SAND (PERCENT)	P200 (PERCENT)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
B-1	13.0		30	92						
B-1	15.0		33					45	20	25
B-1	20.0		35				83			
B-1	25.0		33					33	25	8
B-1	30.0		29				84			
B-1	35.0		34				89			
B-1	44.0		27				56			
B-2	7.5		46					75	30	45
B-2	15.0		32				78			
B-2	25.0		34				92			
B-2	35.0		29				98			
B-2	45.0		27				80			
B-3	2.5		11							
B-3	7.5		38				88			
B-4	2.5		20							

NV5	CEDARST-1-01	SUMMARY OF LABORATORY DATA	
	SEPTEMBER 2022	WESTGATE + HALL DEVELOPMENT BEAVERTON, OR	FIGURE A-7

APPENDIX B

APPENDIX B

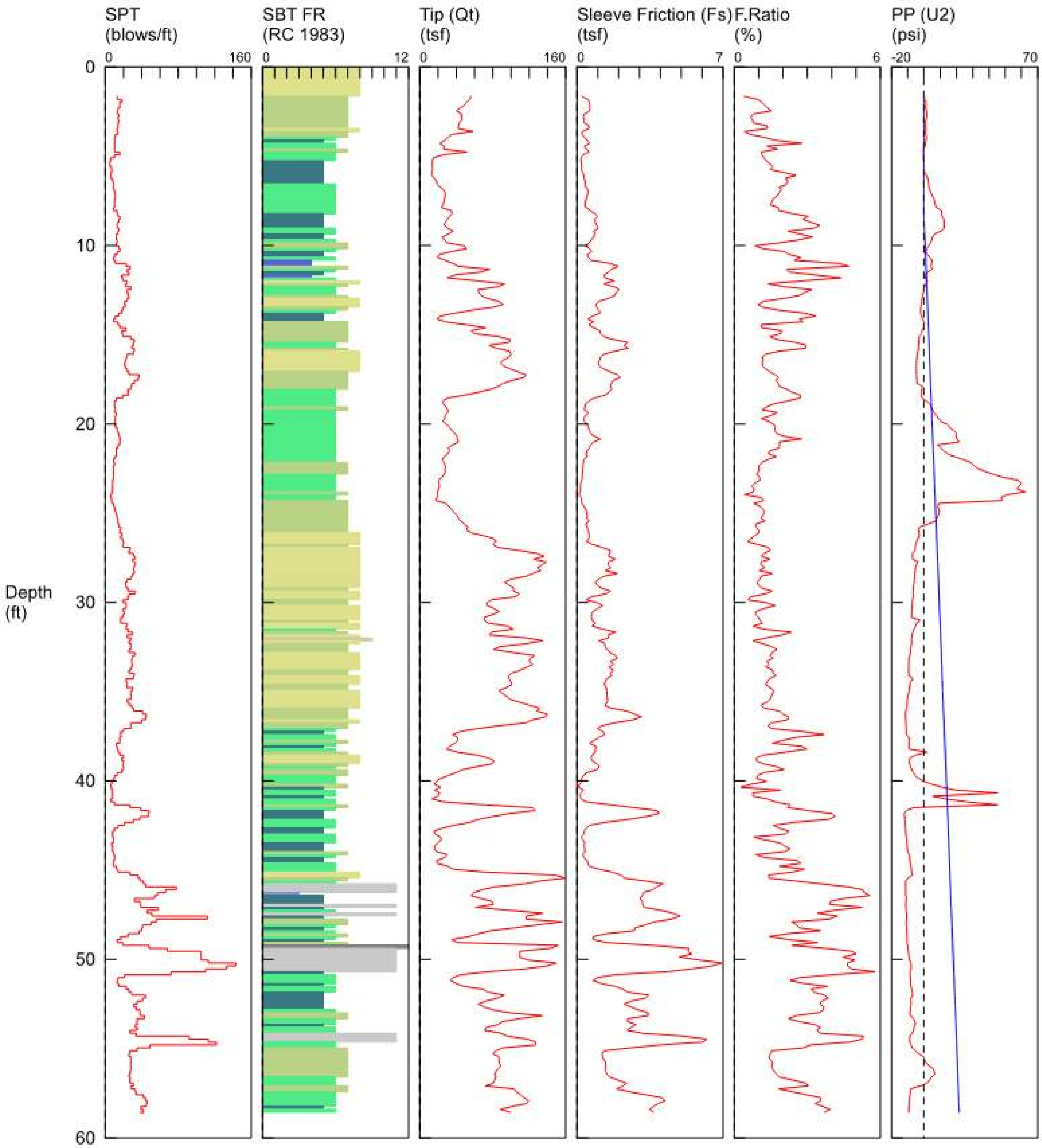
CPT PROBE EXPLORATIONS

Our subsurface exploration program included two CPTs (CPT-1 and CPT-2) that extended to depths between approximately 58.5 and 60 feet BGS. The CPTs were completed in general accordance with ASTM D5778 by Oregon Geotechnical Explorations, Inc. of Keizer, Oregon on August 5, 2022. Locations of the CPTs are shown on Figures 2 and 3. The results of the CPTs are presented in this appendix.

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 are typically recorded at 0.1-meter intervals. At select depths, the CPT advancement can be suspended and pore water dissipation rates can be measured.

NV5 / CPT-1 / 3775 SW Hall Blvd Beaverton

OPERATOR: OGE DMM
CONE ID: DDG1586
HOLE NUMBER: CPT-1
TEST DATE: 8/5/2022 8:15:36 AM
TOTAL DEPTH: 58.563 ft

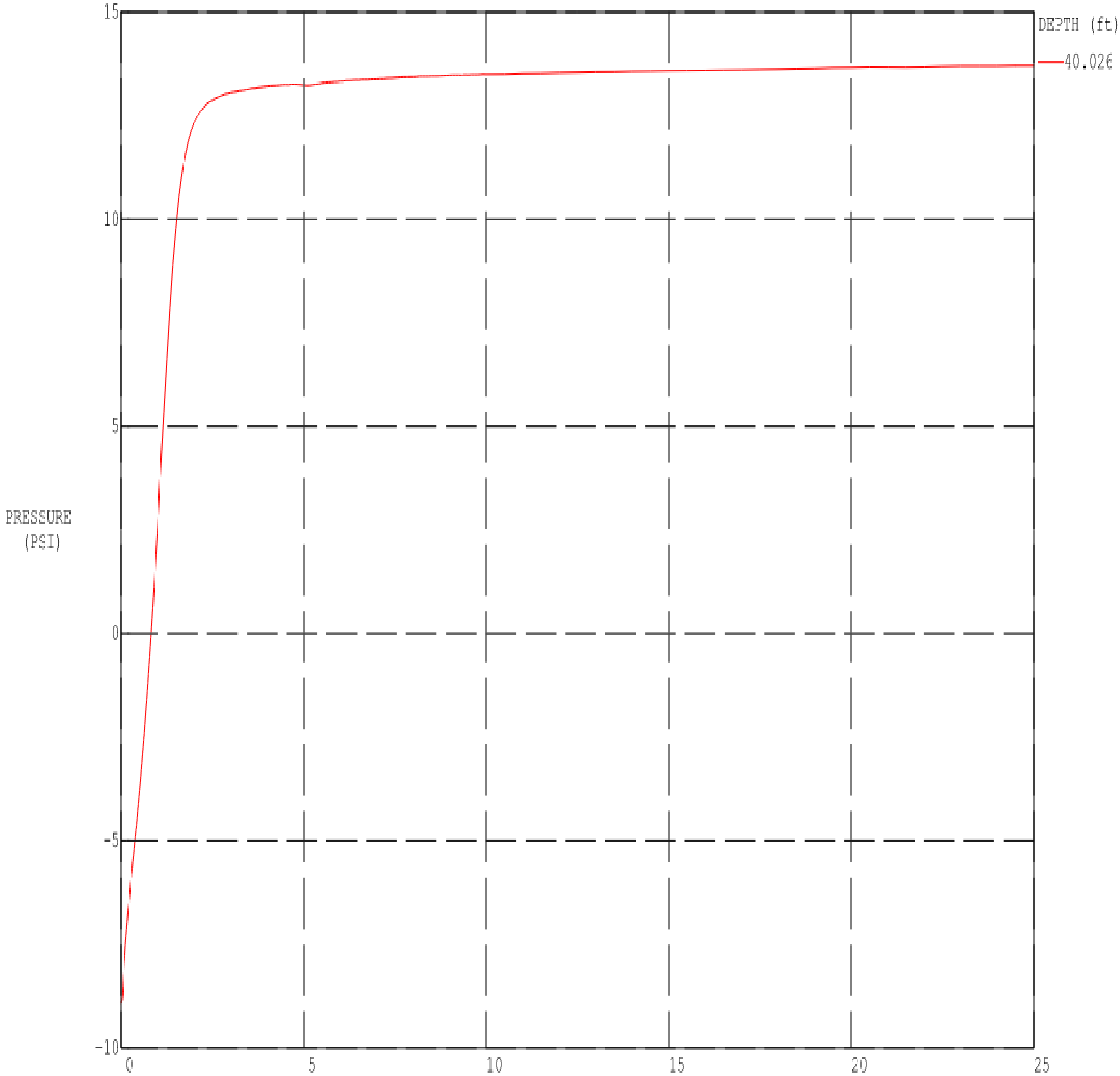


1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

*SBT/SPT CORRELATION: UBC-1983

COMMENT: NV5 / CPT-1 / 3775 SW Hall Blvd Beaverton

TEST DATE: 8/5/2022 8:15:36 AM



MAXIMUM PRESSURE = 13.71 (PSI)

HYDROSTATIC PRESSURE = 13.726 (PSI), WATER TABLE: 8.36 ft

NV5 / CPT-1 / 3775 SW Hall Blvd Beaverton

OPERATOR: OGE DMM
CONE ID: DDG1586
HOLE NUMBER: CPT-1
TEST DATE: 8/5/2022 8:15:36 AM
TOTAL DEPTH: 58.563 ft

Depth ft	Tip (Qt) (tsf)	Sleeve Friction (Fs) (tsf)	F.Ratio (%)	PP (U2) (psi)	SPT (blows/ft)	Zone	Soil Behavior Type UBC-1983
1.640	55.60	0.2363	0.425	0.179	13	8	sand to silty sand
1.804	55.04	0.5128	0.932	0.329	18	7	silty sand to sandy silt
1.969	51.61	0.5829	1.129	1.249	16	7	silty sand to sandy silt
2.133	48.80	0.6075	1.245	1.041	16	7	silty sand to sandy silt
2.297	44.22	0.6021	1.361	0.946	14	7	silty sand to sandy silt
2.461	40.14	0.6124	1.526	1.524	13	7	silty sand to sandy silt
2.625	45.57	0.3636	0.798	1.612	15	7	silty sand to sandy silt
2.789	45.36	0.3027	0.667	1.653	14	7	silty sand to sandy silt
2.953	46.14	0.3488	0.756	1.070	15	7	silty sand to sandy silt
3.117	43.56	0.3322	0.763	0.975	14	7	silty sand to sandy silt
3.281	42.45	0.5650	1.331	0.889	14	7	silty sand to sandy silt
3.445	42.00	0.5714	1.360	1.127	13	7	silty sand to sandy silt
3.609	58.11	0.2522	0.434	0.884	14	8	sand to silty sand
3.773	35.71	0.2271	0.636	0.934	11	7	silty sand to sandy silt
3.937	31.00	0.3721	1.200	1.501	10	7	silty sand to sandy silt
4.101	27.38	0.4696	1.715	1.111	10	6	sandy silt to clayey silt
4.265	21.78	0.6032	2.770	0.773	10	5	clayey silt to silty clay
4.429	26.85	0.3591	1.337	0.644	10	6	sandy silt to clayey silt
4.593	26.03	0.4635	1.780	-0.297	10	6	sandy silt to clayey silt
4.757	51.61	0.5700	1.104	-0.175	16	7	silty sand to sandy silt
4.921	25.13	0.2835	1.128	-0.345	10	6	sandy silt to clayey silt
5.085	14.94	0.1837	1.229	-0.281	6	6	sandy silt to clayey silt
5.249	13.99	0.1832	1.309	-0.143	5	6	sandy silt to clayey silt
5.577	13.78	0.2183	1.585	0.052	7	5	clayey silt to silty clay
5.741	13.89	0.2066	1.487	0.136	7	5	clayey silt to silty clay
5.906	13.54	0.2760	2.038	0.265	6	5	clayey silt to silty clay
6.070	13.06	0.3307	2.532	1.510	6	5	clayey silt to silty clay
6.234	16.11	0.3365	2.088	2.265	8	5	clayey silt to silty clay
6.398	18.61	0.4103	2.204	2.535	9	5	clayey silt to silty clay
6.562	18.75	0.4034	2.152	2.680	9	5	clayey silt to silty clay
6.726	22.90	0.4225	1.845	3.036	9	6	sandy silt to clayey silt
6.890	26.28	0.5094	1.938	3.578	10	6	sandy silt to clayey silt
7.054	28.02	0.5145	1.836	4.311	11	6	sandy silt to clayey silt
7.218	27.71	0.4910	1.772	5.148	11	6	sandy silt to clayey silt
7.382	27.13	0.4349	1.603	6.177	10	6	sandy silt to clayey silt
7.710	25.68	0.3817	1.486	6.862	10	6	sandy silt to clayey silt
7.874	23.71	0.4294	1.811	7.801	9	6	sandy silt to clayey silt
8.038	34.55	0.6299	1.823	10.796	13	6	sandy silt to clayey silt
8.202	35.47	0.8457	2.385	10.860	14	6	sandy silt to clayey silt
8.366	32.77	0.9960	3.040	11.034	16	5	clayey silt to silty clay
8.530	30.38	0.8540	2.811	12.307	15	5	clayey silt to silty clay
8.694	27.90	0.8812	3.158	12.372	13	5	clayey silt to silty clay
8.858	26.57	0.9324	3.510	12.375	13	5	clayey silt to silty clay
9.022	30.95	1.0153	3.281	12.522	15	5	clayey silt to silty clay

Depth ft	Tip (Qt) (tsf)	Sleeve Friction (Fs) (tsf)	F.Ratio (%)	PP (U2) (psi)	SPT (blows/ft)	Zone	Soil Behavior Type UBC-1983
9.186	36.52	0.8014	2.195	10.345	14	6	sandy silt to clayey silt
9.350	31.72	0.8452	2.664	6.669	12	6	sandy silt to clayey silt
9.514	24.82	0.7976	3.213	5.234	12	5	clayey silt to silty clay
9.678	26.89	0.7098	2.640	4.875	13	5	clayey silt to silty clay
9.843	30.47	0.5604	1.839	3.977	12	6	sandy silt to clayey silt
10.007	48.11	0.4090	0.850	2.862	15	7	silty sand to sandy silt
10.171	51.46	0.6035	1.173	1.608	16	7	silty sand to sandy silt
10.335	34.56	0.7146	2.068	1.086	13	6	sandy silt to clayey silt
10.499	21.71	0.4867	2.242	1.785	10	5	clayey silt to silty clay
10.663	19.36	0.5290	2.733	3.574	9	5	clayey silt to silty clay
10.827	27.90	0.6855	2.457	4.982	11	6	sandy silt to clayey silt
10.991	36.54	1.5756	4.312	4.660	23	4	silty clay to clay
11.155	41.99	1.9730	4.699	4.451	27	4	silty clay to clay
11.319	76.31	1.6396	2.148	5.012	24	7	silty sand to sandy silt
11.483	66.94	1.5086	2.254	2.111	26	6	sandy silt to clayey silt
11.647	38.35	1.2838	3.348	0.719	18	5	clayey silt to silty clay
11.811	30.41	1.3372	4.396	0.805	19	4	silty clay to clay
11.975	62.21	1.7762	2.855	1.980	24	6	sandy silt to clayey silt
12.139	92.66	1.2890	1.391	1.238	22	8	sand to silty sand
12.303	86.60	1.7778	2.053	0.467	28	7	silty sand to sandy silt
12.467	63.85	2.0269	3.174	-0.082	24	6	sandy silt to clayey silt
12.631	64.48	1.8650	2.892	-0.370	25	6	sandy silt to clayey silt
12.795	67.29	1.8019	2.678	-0.746	26	6	sandy silt to clayey silt
12.959	73.84	1.5197	2.058	-1.195	24	7	silty sand to sandy silt
13.123	86.25	1.0422	1.208	-1.506	21	8	sand to silty sand
13.287	91.65	0.8833	0.964	-1.714	22	8	sand to silty sand
13.451	80.42	0.9310	1.158	-2.075	19	8	sand to silty sand
13.615	57.16	1.0429	1.824	-2.272	18	7	silty sand to sandy silt
13.780	38.43	1.1099	2.888	-2.098	15	6	sandy silt to clayey silt
13.944	23.08	0.7737	3.352	-1.637	11	5	clayey silt to silty clay
14.108	18.63	0.4987	2.677	-0.825	9	5	clayey silt to silty clay
14.272	28.37	0.8004	2.821	0.100	14	5	clayey silt to silty clay
14.436	51.47	0.5584	1.085	0.172	16	7	silty sand to sandy silt
14.600	72.12	0.8148	1.130	-0.272	23	7	silty sand to sandy silt
14.764	56.14	1.0956	1.952	-1.383	18	7	silty sand to sandy silt
14.928	59.53	1.1382	1.912	-1.562	19	7	silty sand to sandy silt
15.092	86.21	1.2712	1.474	-2.955	28	7	silty sand to sandy silt
15.256	98.78	1.7258	1.747	-3.213	32	7	silty sand to sandy silt
15.420	98.46	2.4441	2.482	-3.372	31	7	silty sand to sandy silt
15.584	75.95	2.2500	2.962	-3.415	29	6	sandy silt to clayey silt
15.748	84.22	2.4630	2.925	-3.510	32	6	sandy silt to clayey silt
15.912	98.15	1.8265	1.861	-3.798	31	7	silty sand to sandy silt
16.076	100.12	1.2919	1.290	-4.052	24	8	sand to silty sand
16.240	97.96	1.1241	1.148	-4.349	23	8	sand to silty sand
16.404	91.37	1.0832	1.186	-4.510	22	8	sand to silty sand
16.568	88.75	1.0194	1.149	-4.651	21	8	sand to silty sand
16.732	91.82	1.0691	1.164	-4.708	22	8	sand to silty sand
16.896	98.87	1.2495	1.264	-4.694	24	8	sand to silty sand
17.060	106.86	1.5548	1.455	-4.701	26	8	sand to silty sand
17.224	116.37	1.9373	1.665	-4.637	37	7	silty sand to sandy silt
17.388	114.21	2.0803	1.821	-4.551	36	7	silty sand to sandy silt
17.552	101.57	1.7336	1.707	-4.404	32	7	silty sand to sandy silt
17.717	91.46	1.6507	1.805	-4.263	29	7	silty sand to sandy silt

Depth ft	Tip (Qt) (tsf)	Sleeve Friction (Fs) (tsf)	F.Ratio (%)	PP (U2) (psi)	SPT (blows/ft)	Zone	Soil Behavior Type UBC-1983
17.881	86.42	1.6049	1.857	-0.948	28	7	silty sand to sandy silt
18.045	77.73	1.6747	2.155	-1.381	25	7	silty sand to sandy silt
18.209	64.18	1.5232	2.373	-1.810	25	6	sandy silt to clayey silt
18.373	49.05	1.3311	2.714	-1.869	19	6	sandy silt to clayey silt
18.537	30.68	0.8325	2.714	-1.247	12	6	sandy silt to clayey silt
18.701	25.50	0.5536	2.171	1.340	10	6	sandy silt to clayey silt
18.865	25.61	0.4152	1.621	3.470	10	6	sandy silt to clayey silt
19.029	28.68	0.3804	1.327	4.445	11	6	sandy silt to clayey silt
19.193	30.73	0.3527	1.148	5.186	10	7	silty sand to sandy silt
19.357	31.00	0.5493	1.772	6.080	12	6	sandy silt to clayey silt
19.521	27.60	0.4128	1.495	7.662	11	6	sandy silt to clayey silt
19.685	27.49	0.3060	1.113	8.894	11	6	sandy silt to clayey silt
19.849	25.17	0.3574	1.420	10.304	10	6	sandy silt to clayey silt
20.013	29.71	0.4843	1.630	14.121	11	6	sandy silt to clayey silt
20.177	32.76	0.5481	1.673	16.044	13	6	sandy silt to clayey silt
20.341	35.29	0.5825	1.651	18.116	14	6	sandy silt to clayey silt
20.505	37.93	0.6509	1.716	19.661	15	6	sandy silt to clayey silt
20.669	40.78	0.8199	2.010	19.701	16	6	sandy silt to clayey silt
20.833	41.03	1.1312	2.757	20.243	16	6	sandy silt to clayey silt
20.997	42.45	0.7862	1.852	21.731	16	6	sandy silt to clayey silt
21.161	36.16	0.6737	1.863	8.361	14	6	sandy silt to clayey silt
21.325	31.20	0.5793	1.856	10.225	12	6	sandy silt to clayey silt
21.490	32.59	0.4741	1.455	12.329	12	6	sandy silt to clayey silt
21.654	31.10	0.4337	1.395	14.381	12	6	sandy silt to clayey silt
21.818	26.79	0.4628	1.728	18.597	10	6	sandy silt to clayey silt
21.982	24.70	0.3412	1.382	23.817	9	6	sandy silt to clayey silt
22.146	26.64	0.2998	1.125	28.808	10	6	sandy silt to clayey silt
22.310	27.89	0.2845	1.020	30.482	9	7	silty sand to sandy silt
22.474	27.94	0.2787	0.998	32.944	9	7	silty sand to sandy silt
22.638	27.82	0.2753	0.990	37.482	9	7	silty sand to sandy silt
22.802	27.02	0.2481	0.918	42.680	9	7	silty sand to sandy silt
22.966	24.31	0.2491	1.024	48.101	9	6	sandy silt to clayey silt
23.130	20.65	0.2194	1.063	57.605	8	6	sandy silt to clayey silt
23.294	20.82	0.1753	0.842	60.088	8	6	sandy silt to clayey silt
23.458	21.65	0.1557	0.719	58.710	8	6	sandy silt to clayey silt
23.622	20.43	0.1674	0.819	57.247	8	6	sandy silt to clayey silt
23.786	19.32	0.1493	0.773	62.621	7	6	sandy silt to clayey silt
23.950	19.62	0.0836	0.426	51.403	6	7	silty sand to sandy silt
24.114	18.62	0.2075	1.114	47.426	7	6	sandy silt to clayey silt
24.278	17.25	0.2080	1.206	48.188	7	6	sandy silt to clayey silt
24.442	28.36	0.2268	0.800	8.960	9	7	silty sand to sandy silt
24.606	31.77	0.3377	1.063	8.447	10	7	silty sand to sandy silt
24.770	34.96	0.4137	1.183	9.023	11	7	silty sand to sandy silt
24.934	37.46	0.4583	1.223	9.363	12	7	silty sand to sandy silt
25.098	40.67	0.4827	1.187	8.758	13	7	silty sand to sandy silt
25.262	42.72	0.4665	1.092	7.701	14	7	silty sand to sandy silt
25.427	44.86	0.5689	1.268	7.234	14	7	silty sand to sandy silt
25.591	48.43	0.4231	0.874	2.574	15	7	silty sand to sandy silt
25.755	52.88	0.4707	0.890	-2.862	17	7	silty sand to sandy silt
25.919	51.12	0.5037	0.985	-3.404	16	7	silty sand to sandy silt
26.083	56.49	0.5705	1.010	-3.129	18	7	silty sand to sandy silt
26.247	69.54	0.5745	0.826	-3.052	17	8	sand to silty sand
26.411	81.25	0.5561	0.684	-3.853	19	8	sand to silty sand

Depth ft	Tip (Qt) (tsf)	Sleeve Friction (Fs) (tsf)	F.Ratio (%)	PP (U2) (psi)	SPT (blows/ft)	Zone	Soil Behavior Type UBC-1983
26.575	83.66	0.4495	0.537	-4.522	20	8	sand to silty sand
26.739	80.19	0.7466	0.931	-5.148	19	8	sand to silty sand
26.903	84.85	1.1800	1.391	-5.436	27	7	silty sand to sandy silt
27.067	108.28	1.6063	1.483	-5.519	26	8	sand to silty sand
27.231	128.18	1.5570	1.215	-7.075	31	8	sand to silty sand
27.395	136.43	1.5052	1.103	-7.100	33	8	sand to silty sand
27.559	124.94	1.8318	1.466	-7.082	30	8	sand to silty sand
27.723	138.88	1.5115	1.088	-3.562	33	8	sand to silty sand
27.887	134.10	1.4688	1.095	-4.674	32	8	sand to silty sand
28.051	129.10	1.6224	1.257	-5.229	31	8	sand to silty sand
28.215	132.65	1.3458	1.015	-5.456	32	8	sand to silty sand
28.379	123.73	1.9341	1.563	-5.646	30	8	sand to silty sand
28.543	116.44	1.2708	1.091	-5.778	28	8	sand to silty sand
28.707	100.09	1.0392	1.038	-5.912	24	8	sand to silty sand
28.871	92.84	0.8671	0.934	-6.102	22	8	sand to silty sand
29.035	97.67	0.9646	0.988	-6.370	23	8	sand to silty sand
29.199	102.83	1.1360	1.105	-6.624	25	8	sand to silty sand
29.364	102.59	1.5955	1.555	-6.671	33	7	silty sand to sandy silt
29.528	108.79	1.3865	1.274	-6.681	26	8	sand to silty sand
29.692	104.09	0.6959	0.669	-6.606	25	8	sand to silty sand
29.856	77.97	0.4367	0.560	-7.071	19	8	sand to silty sand
30.020	73.60	0.9739	1.323	-7.182	23	7	silty sand to sandy silt
30.184	73.40	1.2326	1.679	-7.186	23	7	silty sand to sandy silt
30.348	81.28	0.9813	1.207	-7.388	19	8	sand to silty sand
30.512	87.23	0.7608	0.872	-7.594	21	8	sand to silty sand
30.676	80.40	0.7027	0.874	-7.683	19	8	sand to silty sand
30.840	70.78	0.6935	0.980	-7.730	17	8	sand to silty sand
31.004	70.59	0.7463	1.057	-2.497	17	8	sand to silty sand
31.168	74.96	0.8667	1.156	-4.515	24	7	silty sand to sandy silt
31.332	96.09	0.7364	0.766	-5.193	23	8	sand to silty sand
31.496	102.55	1.2222	1.192	-6.279	25	8	sand to silty sand
31.660	78.93	1.8560	2.351	-6.787	30	6	sandy silt to clayey silt
31.824	76.13	1.4979	1.968	-7.334	24	7	silty sand to sandy silt
31.988	116.31	1.5145	1.302	-7.690	28	8	sand to silty sand
32.152	135.01	1.0443	0.774	-7.755	26	9	sand
32.316	117.33	1.3287	1.132	-7.576	28	8	sand to silty sand
32.480	87.61	1.3455	1.536	-8.130	28	7	silty sand to sandy silt
32.644	81.06	1.4161	1.747	-8.168	26	7	silty sand to sandy silt
32.808	102.43	1.5884	1.551	-8.304	33	7	silty sand to sandy silt
32.972	125.54	1.8474	1.472	-8.683	30	8	sand to silty sand
33.136	121.22	1.9256	1.588	-9.370	29	8	sand to silty sand
33.301	119.94	1.6056	1.339	-9.402	29	8	sand to silty sand
33.465	123.59	1.7445	1.412	-9.479	30	8	sand to silty sand
33.629	117.80	1.5257	1.295	-9.497	28	8	sand to silty sand
33.793	110.67	1.7378	1.570	-9.504	26	8	sand to silty sand
33.957	100.85	1.6106	1.597	-9.415	32	7	silty sand to sandy silt
34.121	96.93	1.6571	1.710	-9.431	31	7	silty sand to sandy silt
34.285	97.74	1.4000	1.432	-6.751	23	8	sand to silty sand
34.449	99.99	1.3528	1.353	-8.182	24	8	sand to silty sand
34.613	100.53	1.3027	1.296	-8.685	24	8	sand to silty sand
34.777	94.85	1.3910	1.467	-8.903	30	7	silty sand to sandy silt
34.941	86.66	1.4106	1.628	-9.084	28	7	silty sand to sandy silt
35.433	98.97	1.4567	1.472	-10.318	24	8	sand to silty sand

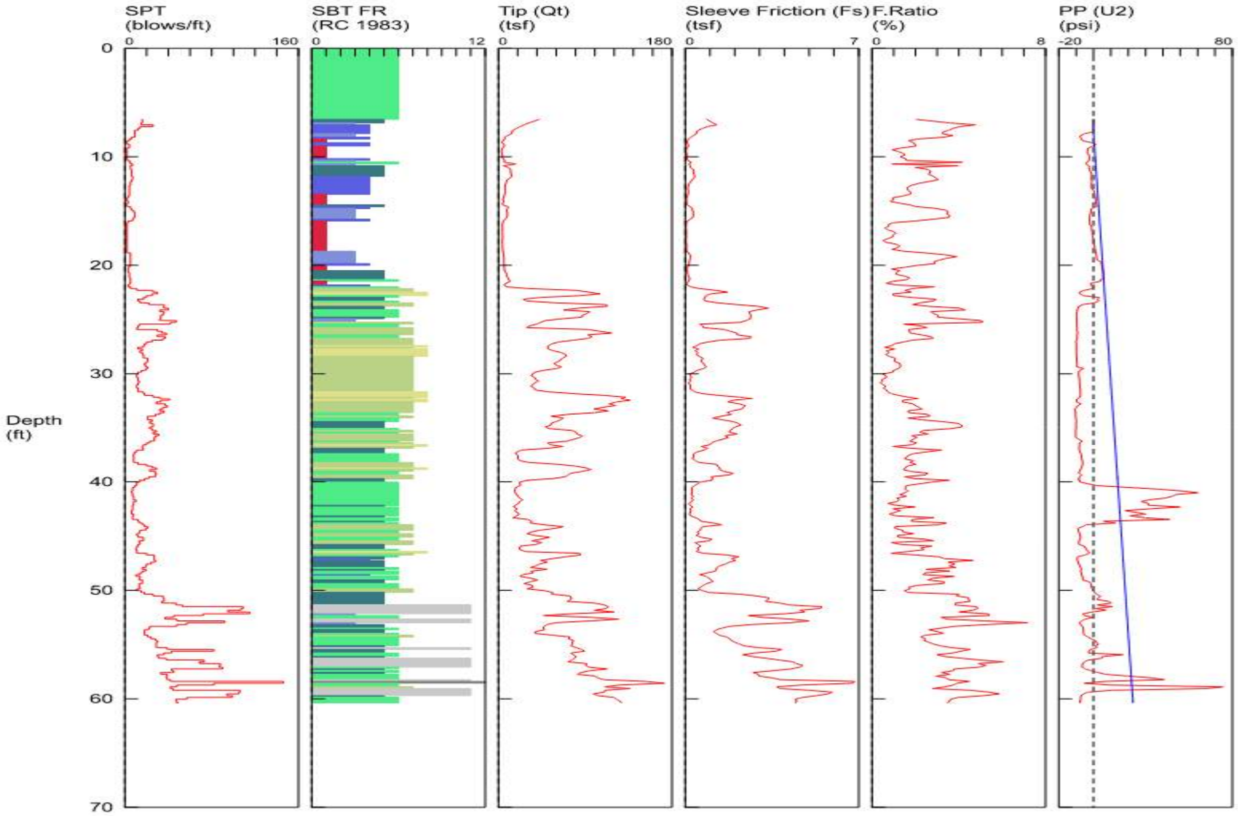
Depth ft	Tip (Qt) (tsf)	Sleeve Friction (Fs) (tsf)	F.Ratio (%)	PP (U2) (psi)	SPT (blows/ft)	Zone	Soil Behavior Type UBC-1983
35.597	109.39	1.4568	1.332	-10.370	26	8	sand to silty sand
35.761	119.86	1.3130	1.095	-10.399	29	8	sand to silty sand
35.925	131.32	1.6020	1.220	-10.660	31	8	sand to silty sand
36.089	128.20	2.3889	1.863	-11.359	41	7	silty sand to sandy silt
36.253	139.60	2.7698	1.984	-11.297	45	7	silty sand to sandy silt
36.417	138.07	3.0806	2.231	-11.270	44	7	silty sand to sandy silt
36.581	123.75	2.7014	2.183	-11.250	40	7	silty sand to sandy silt
36.745	116.24	1.8038	1.552	-11.089	28	8	sand to silty sand
36.909	87.48	1.2006	1.372	-11.016	28	7	silty sand to sandy silt
37.073	63.66	0.9745	1.531	-10.960	20	7	silty sand to sandy silt
37.238	44.29	1.2454	2.812	-10.876	17	6	sandy silt to clayey silt
37.402	35.97	1.3256	3.685	-10.833	17	5	clayey silt to silty clay
37.566	41.38	1.0498	2.537	-9.318	16	6	sandy silt to clayey silt
37.730	41.59	0.8397	2.019	-9.316	16	6	sandy silt to clayey silt
37.894	38.46	0.6035	1.569	-9.327	12	7	silty sand to sandy silt
38.058	32.81	0.9060	2.761	-9.295	13	6	sandy silt to clayey silt
38.222	29.58	0.8835	2.987	-9.216	14	5	clayey silt to silty clay
38.386	40.55	0.8611	2.124	1.751	16	6	sandy silt to clayey silt
38.550	63.06	0.7404	1.174	-5.100	20	7	silty sand to sandy silt
38.714	74.31	0.6746	0.908	-8.526	18	8	sand to silty sand
38.878	81.65	0.7386	0.905	-9.123	20	8	sand to silty sand
39.042	79.26	0.9503	1.199	-8.837	19	8	sand to silty sand
39.206	62.96	1.0771	1.711	-8.340	20	7	silty sand to sandy silt
39.370	46.47	0.9518	2.048	-7.624	18	6	sandy silt to clayey silt
39.534	40.49	0.4944	1.221	-6.615	13	7	silty sand to sandy silt
39.698	31.27	0.3039	0.972	-5.476	10	7	silty sand to sandy silt
39.862	23.22	0.2115	0.911	-3.408	9	6	sandy silt to clayey silt
40.026	17.77	0.2349	1.322	-0.002	7	6	sandy silt to clayey silt
40.190	15.95	0.1076	0.674	4.841	6	6	sandy silt to clayey silt
40.354	22.66	0.0605	0.267	11.458	7	7	silty sand to sandy silt
40.518	16.21	0.3090	1.906	19.588	8	5	clayey silt to silty clay
40.682	22.09	0.1657	0.750	45.435	8	6	sandy silt to clayey silt
40.846	19.69	0.1515	0.770	5.340	8	6	sandy silt to clayey silt
41.011	13.42	0.1810	1.349	13.080	6	5	clayey silt to silty clay
41.175	25.58	0.4301	1.681	26.373	10	6	sandy silt to clayey silt
41.339	70.78	1.6300	2.303	45.285	27	6	sandy silt to clayey silt
41.503	123.63	2.7100	2.192	-6.436	39	7	silty sand to sandy silt
41.667	126.21	3.8413	3.044	-10.867	48	6	sandy silt to clayey silt
41.831	97.72	3.9293	4.021	-12.146	47	5	clayey silt to silty clay
41.995	79.91	3.3043	4.135	-11.907	38	5	clayey silt to silty clay
42.159	63.99	2.4749	3.868	-11.774	31	5	clayey silt to silty clay
42.323	54.63	1.3442	2.460	-11.597	21	6	sandy silt to clayey silt
42.487	37.41	0.5974	1.597	-11.436	14	6	sandy silt to clayey silt
42.651	25.09	0.4636	1.848	-11.386	10	6	sandy silt to clayey silt
42.815	15.71	0.3519	2.240	-11.245	8	5	clayey silt to silty clay
42.979	16.49	0.3450	2.092	-11.089	8	5	clayey silt to silty clay
43.143	22.95	0.1833	0.799	-10.916	9	6	sandy silt to clayey silt
43.307	23.68	0.2972	1.255	-10.710	9	6	sandy silt to clayey silt
43.471	19.24	0.3639	1.892	-10.520	7	6	sandy silt to clayey silt
43.635	16.66	0.3416	2.050	-10.404	8	5	clayey silt to silty clay
43.799	15.99	0.3664	2.292	-10.238	8	5	clayey silt to silty clay
43.963	18.45	0.3812	2.066	-10.055	9	5	clayey silt to silty clay
44.127	30.52	0.2709	0.887	-8.923	10	7	silty sand to sandy silt

Depth ft	Tip (Qt) (tsf)	Sleeve Friction (Fs) (tsf)	F.Ratio (%)	PP (U2) (psi)	SPT (blows/ft)	Zone	Soil Behavior Type UBC-1983
44.291	27.13	0.3337	1.230	-8.821	10	6	sandy silt to clayey silt
44.455	18.65	0.4824	2.586	-8.647	9	5	clayey silt to silty clay
44.619	18.91	0.5160	2.729	-8.390	9	5	clayey silt to silty clay
44.783	30.72	0.5732	1.865	-8.057	12	6	sandy silt to clayey silt
44.948	36.71	1.0549	2.874	-7.864	14	6	sandy silt to clayey silt
45.112	70.59	1.6820	2.383	-7.606	27	6	sandy silt to clayey silt
45.276	149.02	2.0946	1.406	-7.018	36	8	sand to silty sand
45.440	158.94	2.2713	1.429	-10.105	38	8	sand to silty sand
45.604	132.01	3.2947	2.496	-11.186	42	7	silty sand to sandy silt
45.768	115.58	4.1250	3.569	-11.307	44	6	sandy silt to clayey silt
45.932	81.30	3.8092	4.685	-11.266	78	11	very stiff fine grained (*)
46.096	68.75	3.6288	5.278	-11.207	66	11	very stiff fine grained (*)
46.260	59.65	3.1749	5.323	-11.220	57	11	very stiff fine grained (*)
46.424	56.01	3.1233	5.577	-11.082	54	3	clay
46.588	67.05	3.0138	4.495	-10.980	32	5	clayey silt to silty clay
46.752	79.18	3.1172	3.937	-11.043	38	5	clayey silt to silty clay
46.916	80.70	3.1455	3.898	-11.157	39	5	clayey silt to silty clay
47.080	61.04	3.2087	5.257	-11.193	58	11	very stiff fine grained (*)
47.244	95.88	3.9740	4.145	-11.089	46	5	clayey silt to silty clay
47.408	134.36	4.5966	3.421	-10.651	51	6	sandy silt to clayey silt
47.572	116.94	4.9424	4.226	-10.721	112	11	very stiff fine grained (*)
47.736	116.15	4.5428	3.911	-10.660	56	5	clayey silt to silty clay
47.900	156.04	3.6824	2.360	-10.905	50	7	silty sand to sandy silt
48.064	129.15	3.2913	2.548	-10.885	41	7	silty sand to sandy silt
48.228	104.26	2.7776	2.664	-10.783	40	6	sandy silt to clayey silt
48.392	77.64	2.7484	3.540	-10.601	37	5	clayey silt to silty clay
48.556	64.38	1.4006	2.176	-10.574	25	6	sandy silt to clayey silt
48.720	54.27	0.7862	1.449	-10.456	17	7	silty sand to sandy silt
48.885	35.08	0.7816	2.228	-10.418	13	6	sandy silt to clayey silt
49.049	40.65	1.3934	3.428	-10.474	19	5	clayey silt to silty clay
49.213	151.55	3.6004	2.376	-9.892	48	7	silty sand to sandy silt
49.377	142.72	5.3678	3.761	-9.996	68	12	sand to clayey sand (*)
49.541	109.76	5.3077	4.836	-9.717	105	11	very stiff fine grained (*)
49.705	109.57	5.4851	5.006	-9.515	105	11	very stiff fine grained (*)
49.869	109.41	4.8826	4.463	-9.184	105	11	very stiff fine grained (*)
50.033	118.48	5.8676	4.952	-8.921	113	11	very stiff fine grained (*)
50.197	149.20	6.9339	4.647	-7.989	143	11	very stiff fine grained (*)
50.361	139.09	6.4632	4.647	-7.993	133	11	very stiff fine grained (*)
50.525	114.57	5.5025	4.803	-7.937	110	11	very stiff fine grained (*)
50.689	74.96	4.3268	5.772	-8.449	72	11	very stiff fine grained (*)
50.853	44.61	1.7934	4.020	-8.599	21	5	clayey silt to silty clay
51.017	36.21	0.9643	2.663	-8.427	14	6	sandy silt to clayey silt
51.181	33.92	0.7810	2.302	-8.243	13	6	sandy silt to clayey silt
51.345	40.12	1.1555	2.880	-8.216	15	6	sandy silt to clayey silt
51.509	47.71	1.8267	3.829	-8.209	23	5	clayey silt to silty clay
51.673	69.14	2.1840	3.159	-7.649	26	6	sandy silt to clayey silt
51.837	75.33	2.4734	3.283	-7.105	29	6	sandy silt to clayey silt
52.001	92.62	3.4298	3.703	-6.984	44	5	clayey silt to silty clay
52.165	85.16	3.2879	3.861	-8.177	41	5	clayey silt to silty clay
52.329	72.11	2.5823	3.581	-8.093	35	5	clayey silt to silty clay
52.493	63.96	2.2500	3.518	-8.159	31	5	clayey silt to silty clay
52.657	70.03	2.5725	3.673	-8.216	34	5	clayey silt to silty clay
52.822	80.30	2.9384	3.659	-8.440	38	5	clayey silt to silty clay

Depth ft	Tip (Qt) (tsf)	Sleeve Friction (Fs) (tsf)	F.Ratio (%)	PP (U2) (psi)	SPT (blows/ft)	Zone	Soil Behavior Type UBC-1983
52.986	98.05	3.4022	3.470	-8.606	38	6	sandy silt to clayey silt
53.150	133.88	3.4029	2.542	-8.522	43	7	silty sand to sandy silt
53.314	109.53	2.4348	2.223	-6.388	35	7	silty sand to sandy silt
53.478	90.81	2.4119	2.656	-5.529	35	6	sandy silt to clayey silt
53.642	85.63	2.8317	3.307	-5.470	33	6	sandy silt to clayey silt
53.806	73.04	2.5826	3.536	-6.232	35	5	clayey silt to silty clay
53.970	70.94	2.4081	3.394	-6.417	27	6	sandy silt to clayey silt
54.134	89.02	3.1365	3.524	-7.345	34	6	sandy silt to clayey silt
54.298	95.75	5.0648	5.290	-8.622	92	11	very stiff fine grained (*)
54.462	118.30	6.2229	5.260	-8.914	113	11	very stiff fine grained (*)
54.626	126.99	6.0305	4.749	-8.851	122	11	very stiff fine grained (*)
54.790	127.13	3.9850	3.135	-8.154	49	6	sandy silt to clayey silt
54.954	90.45	2.5629	2.833	-7.236	35	6	sandy silt to clayey silt
55.118	83.74	1.2856	1.535	-5.572	27	7	silty sand to sandy silt
55.282	80.16	1.1976	1.494	-3.463	26	7	silty sand to sandy silt
55.446	86.47	1.2257	1.418	-0.138	28	7	silty sand to sandy silt
55.610	86.70	1.2823	1.479	0.315	28	7	silty sand to sandy silt
55.774	86.88	1.3699	1.577	0.907	28	7	silty sand to sandy silt
55.938	88.68	1.3628	1.537	2.898	28	7	silty sand to sandy silt
56.102	86.93	1.3545	1.558	5.581	28	7	silty sand to sandy silt
56.266	83.28	1.3654	1.639	6.211	27	7	silty sand to sandy silt
56.430	82.14	1.4334	1.745	6.610	26	7	silty sand to sandy silt
56.594	83.47	1.5469	1.853	4.739	27	7	silty sand to sandy silt
56.923	80.49	2.3743	2.950	1.787	31	6	sandy silt to clayey silt
57.087	72.16	2.1920	3.038	-2.136	28	6	sandy silt to clayey silt
57.251	103.29	1.8955	1.835	-8.336	33	7	silty sand to sandy silt
57.415	105.57	2.5475	2.413	-8.488	34	7	silty sand to sandy silt
57.579	110.86	3.2815	2.960	-8.696	42	6	sandy silt to clayey silt
57.743	114.27	4.0461	3.541	-8.880	44	6	sandy silt to clayey silt
57.907	119.14	4.2292	3.550	-9.050	46	6	sandy silt to clayey silt
58.071	116.88	3.8853	3.324	-9.166	45	6	sandy silt to clayey silt
58.235	100.77	3.6635	3.636	-9.293	39	6	sandy silt to clayey silt
58.399	88.71	3.4809	3.924	-9.338	42	5	clayey silt to silty clay
58.563	99.03	3.6609	3.697	-9.397	38	6	sandy silt to clayey silt

NV5 / CPT-2 / 3775 SW Hall Blvd Beaverton

OPERATOR: OGE DMM
CONE ID: DDG1586
HOLE NUMBER: CPT-2
TEST DATE: 8/5/2022 10:15:15 AM
TOTAL DEPTH: 60.367 ft

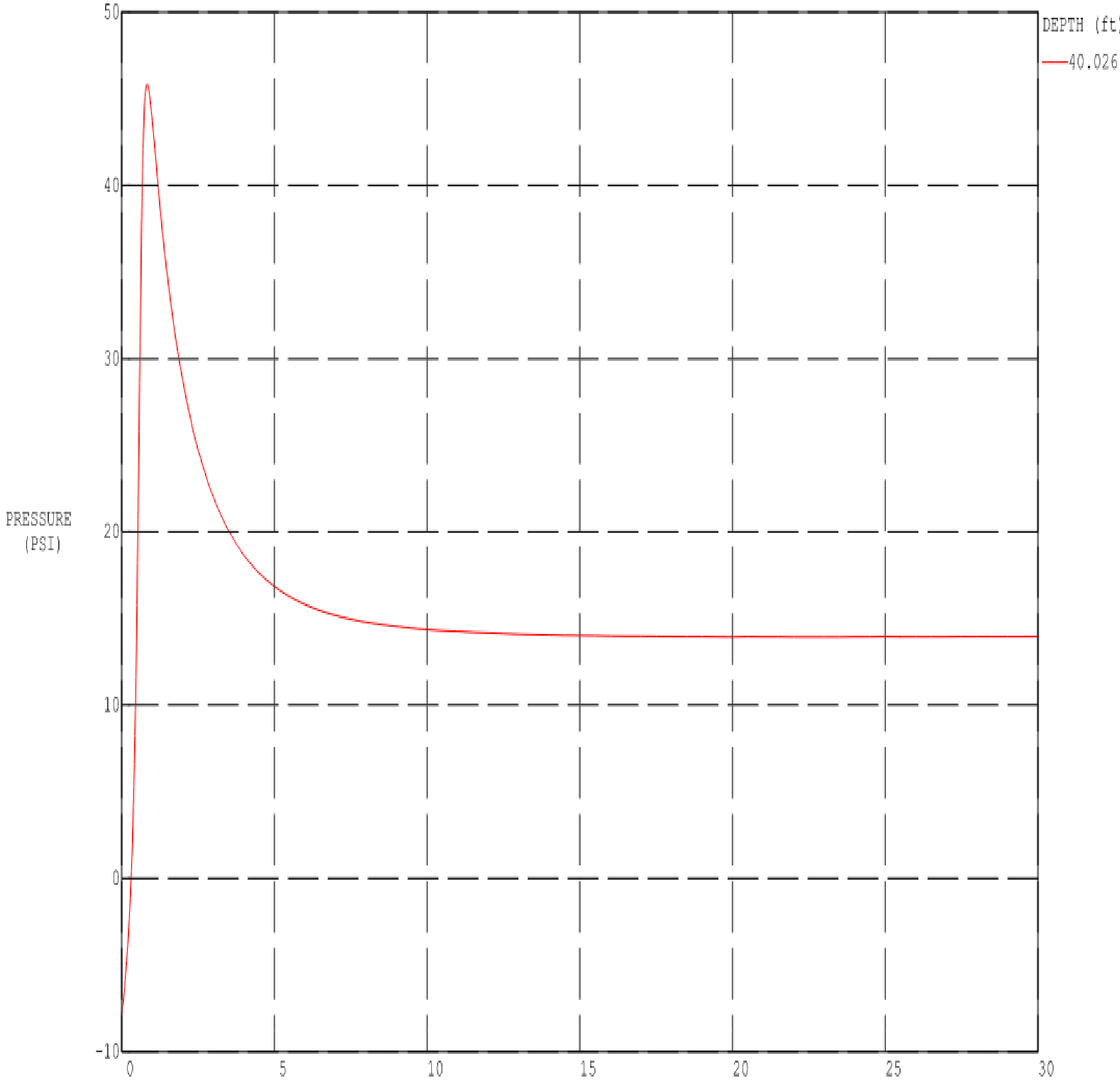


1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

*SBT/SPT CORRELATION: UBC-1983

COMMENT: NV5 / CPT-2 / 3775 SW Hall Blvd Beaverton

TEST DATE: 8/5/2022 10:15:15 AM



MAXIMUM PRESSURE = 45.864 (PSI)

HYDROSTATIC PRESSURE = 13.966 (PSI), WATER TABLE: 7.80 ft

NV5 / CPT-2 / 3775 SW Hall Blvd Beaverton

OPERATOR: OGE DMM
CONE ID: DDG1586
HOLE NUMBER: CPT-2
TEST DATE: 8/5/2022 10:15:15 AM
TOTAL DEPTH: 60.367 ft

Depth ft	Tip (Qt) (tsf)	Sleeve Friction (Fs) (tsf)	F.Ratio (%)	PP (U2) (psi)	SPT (blows/ft)	Zone	Soil Behavior Type UBC-1983
6.562	42.19	0.8634	2.047	0.002	16	6	sandy silt to clayey silt
6.890	31.20	1.1299	3.621	0.163	15	5	clayey silt to silty clay
7.054	26.70	1.2741	4.771	-0.302	26	3	clay
7.218	22.25	0.9028	4.059	-0.186	14	4	silty clay to clay
7.382	18.87	0.7042	3.732	-0.100	12	4	silty clay to clay
7.546	14.78	0.5254	3.554	-0.431	9	4	silty clay to clay
7.710	14.00	0.4607	3.291	-0.420	9	4	silty clay to clay
7.874	13.62	0.4408	3.235	-5.025	9	4	silty clay to clay
8.038	9.19	0.3423	3.725	-7.690	9	3	clay
8.202	6.69	0.2204	3.295	-7.569	6	3	clay
8.366	6.23	0.1276	2.046	-6.864	4	4	silty clay to clay
8.530	4.57	0.0770	1.685	-6.300	2	1	sensitive fine grained
8.694	3.61	0.0690	1.915	-1.324	2	1	sensitive fine grained
8.858	5.48	0.1084	1.977	1.449	4	4	silty clay to clay
9.022	6.17	0.1248	2.023	-0.345	4	4	silty clay to clay
9.186	4.26	0.0584	1.369	-3.011	2	1	sensitive fine grained
9.350	3.23	0.0300	0.928	-3.492	2	1	sensitive fine grained
9.514	3.35	0.0400	1.197	-3.433	2	1	sensitive fine grained
9.678	3.49	0.0500	1.431	-3.583	2	1	sensitive fine grained
9.843	4.00	0.0600	1.500	-2.640	2	1	sensitive fine grained
10.007	3.85	0.0500	1.298	-2.562	2	1	sensitive fine grained
10.171	4.49	0.0700	1.560	-2.463	2	1	sensitive fine grained
10.335	6.41	0.1010	1.576	-2.569	4	4	silty clay to clay
10.499	5.66	0.2354	4.156	-3.696	5	3	clay
10.663	17.88	0.1634	0.914	-4.610	7	6	sandy silt to clayey silt
10.827	4.87	0.1933	3.971	-2.608	5	3	clay
10.991	10.37	0.2068	1.995	-2.658	5	5	clayey silt to silty clay
11.155	12.63	0.3233	2.560	-2.474	6	5	clayey silt to silty clay
11.319	12.59	0.3391	2.692	-2.025	6	5	clayey silt to silty clay
11.483	12.89	0.3419	2.653	-1.238	6	5	clayey silt to silty clay
11.647	13.21	0.3461	2.621	-1.154	6	5	clayey silt to silty clay
11.811	13.72	0.3896	2.840	-1.134	7	5	clayey silt to silty clay
11.975	11.43	0.3384	2.961	-1.313	7	4	silty clay to clay
12.139	8.74	0.2657	3.040	-1.249	6	4	silty clay to clay
12.303	8.12	0.2073	2.552	-0.973	5	4	silty clay to clay
12.467	7.61	0.1613	2.119	-1.200	5	4	silty clay to clay
12.631	6.97	0.1274	1.829	-1.186	4	4	silty clay to clay
12.795	6.46	0.1063	1.644	-1.145	4	4	silty clay to clay
12.959	6.47	0.1043	1.611	-1.064	4	4	silty clay to clay
13.123	6.96	0.1172	1.684	0.116	4	4	silty clay to clay
13.287	6.97	0.1239	1.777	0.420	4	4	silty clay to clay
13.451	6.18	0.1095	1.772	0.653	4	4	silty clay to clay
13.615	5.63	0.0794	1.410	0.896	3	1	sensitive fine grained
13.780	5.22	0.0500	0.959	1.020	2	1	sensitive fine grained

Depth ft	Tip (Qt) (tsf)	Sleeve Friction (Fs) (tsf)	F.Ratio (%)	PP (U2) (psi)	SPT (blows/ft)	Zone	Soil Behavior Type UBC-1983
13.944	4.36	0.0436	1.001	1.052	2	1	sensitive fine grained
14.108	5.47	0.0457	0.836	1.215	3	1	sensitive fine grained
14.272	4.34	0.0546	1.258	1.354	2	1	sensitive fine grained
14.436	4.32	0.0753	1.741	1.596	2	1	sensitive fine grained
14.600	9.88	0.1984	2.008	1.880	5	5	clayey silt to silty clay
14.764	9.56	0.2817	2.945	-1.238	6	4	silty clay to clay
14.928	8.40	0.2872	3.419	-1.474	8	3	clay
15.092	9.03	0.3171	3.511	-1.640	9	3	clay
15.256	9.24	0.3176	3.435	-1.812	9	3	clay
15.420	8.91	0.3195	3.585	-2.181	9	3	clay
15.584	8.53	0.3022	3.543	-2.277	8	3	clay
15.748	7.08	0.2035	2.874	-2.386	7	3	clay
15.912	5.08	0.1031	2.027	-2.397	3	4	silty clay to clay
16.076	4.37	0.0429	0.983	-2.286	2	1	sensitive fine grained
16.240	4.02	0.0306	0.761	-2.172	2	1	sensitive fine grained
16.404	3.99	0.0309	0.774	-1.227	2	1	sensitive fine grained
16.568	4.00	0.0234	0.584	-1.116	2	1	sensitive fine grained
16.732	4.10	0.0304	0.741	-0.991	2	1	sensitive fine grained
16.896	4.12	0.0408	0.990	-0.832	2	1	sensitive fine grained
17.060	4.17	0.0455	1.091	-0.719	2	1	sensitive fine grained
17.224	4.21	0.0401	0.952	-0.605	2	1	sensitive fine grained
17.388	4.26	0.0404	0.949	-0.388	2	1	sensitive fine grained
17.552	4.29	0.0308	0.717	-0.156	2	1	sensitive fine grained
17.717	3.92	0.0198	0.505	0.014	2	1	sensitive fine grained
17.881	3.88	0.0245	0.632	0.188	2	1	sensitive fine grained
18.045	4.07	0.0441	1.084	0.365	2	1	sensitive fine grained
18.209	4.11	0.0522	1.272	0.501	2	1	sensitive fine grained
18.373	4.19	0.0499	1.190	0.658	2	1	sensitive fine grained
18.537	4.30	0.0394	0.916	0.839	2	1	sensitive fine grained
18.701	4.53	0.0658	1.453	1.073	2	1	sensitive fine grained
18.865	4.94	0.1222	2.474	1.361	5	3	clay
19.029	5.44	0.1809	3.322	1.533	5	3	clay
19.193	5.44	0.2125	3.910	1.703	5	3	clay
19.357	5.69	0.1962	3.450	1.864	5	3	clay
19.521	5.65	0.1747	3.093	2.002	5	3	clay
19.685	5.53	0.1580	2.855	4.259	5	3	clay
19.849	5.64	0.1636	2.899	4.519	5	3	clay
20.013	5.87	0.1213	2.065	4.812	4	4	silty clay to clay
20.177	5.71	0.0888	1.556	4.882	3	1	sensitive fine grained
20.341	6.11	0.0548	0.897	5.118	3	1	sensitive fine grained
20.505	6.21	0.0794	1.278	5.252	3	1	sensitive fine grained
20.669	7.74	0.0868	1.122	5.626	4	5	clayey silt to silty clay
20.833	8.55	0.0940	1.099	5.640	4	5	clayey silt to silty clay
20.997	8.13	0.1219	1.500	5.624	4	5	clayey silt to silty clay
21.161	9.79	0.1700	1.736	5.710	5	5	clayey silt to silty clay
21.325	10.50	0.1688	1.608	4.091	5	5	clayey silt to silty clay
21.490	12.55	0.1208	0.963	3.721	5	6	sandy silt to clayey silt
21.654	7.62	0.0496	0.651	-1.633	4	1	sensitive fine grained
21.818	5.91	0.0674	1.140	-1.417	3	1	sensitive fine grained
21.982	7.83	0.2244	2.867	-0.742	5	4	silty clay to clay
22.146	29.61	0.5532	1.868	-0.224	11	6	sandy silt to clayey silt
22.310	80.95	1.2329	1.523	-5.485	26	7	silty sand to sandy silt
22.474	93.90	1.7008	1.811	-8.202	30	7	silty sand to sandy silt

Depth ft	Tip (Qt) (tsf)	Sleeve Friction (Fs) (tsf)	F.Ratio (%)	PP (U2) (psi)	SPT (blows/ft)	Zone	Soil Behavior Type UBC-1983
22.638	105.47	1.0129	0.960	-7.948	25	8	sand to silty sand
22.802	84.47	0.8434	0.998	-5.939	20	8	sand to silty sand
22.966	43.37	0.7925	1.827	3.265	17	6	sandy silt to clayey silt
23.130	25.49	0.7485	2.936	2.318	12	5	clayey silt to silty clay
23.294	28.56	0.8123	2.845	3.039	14	5	clayey silt to silty clay
23.458	58.37	1.2885	2.208	0.590	22	6	sandy silt to clayey silt
23.622	111.69	2.0924	1.873	-6.129	36	7	silty sand to sandy silt
23.786	112.86	2.9650	2.627	-8.436	36	7	silty sand to sandy silt
23.950	82.98	3.3458	4.032	-9.055	40	5	clayey silt to silty clay
24.114	67.40	2.9043	4.309	-9.338	32	5	clayey silt to silty clay
24.278	94.58	2.5735	2.721	-9.574	36	6	sandy silt to clayey silt
24.442	91.93	2.5963	2.824	-9.871	35	6	sandy silt to clayey silt
24.606	87.14	2.6496	3.041	-9.812	33	6	sandy silt to clayey silt
24.770	86.95	2.6224	3.016	-9.799	33	6	sandy silt to clayey silt
24.934	66.26	2.6285	3.967	-9.744	32	5	clayey silt to silty clay
25.098	48.77	2.4595	5.043	-9.633	47	3	clay
25.262	43.47	2.2151	5.095	-9.755	42	3	clay
25.427	39.77	0.5848	1.470	-9.647	13	7	silty sand to sandy silt
25.591	31.41	0.7157	2.279	-9.622	12	6	sandy silt to clayey silt
25.755	29.36	0.7400	2.521	-9.495	11	6	sandy silt to clayey silt
25.919	93.02	1.5695	1.687	-9.610	30	7	silty sand to sandy silt
26.083	105.19	1.8170	1.727	-9.445	34	7	silty sand to sandy silt
26.247	117.55	2.0759	1.766	-9.164	38	7	silty sand to sandy silt
26.411	92.68	2.1063	2.273	-9.422	30	7	silty sand to sandy silt
26.575	97.22	2.6496	2.725	-9.411	37	6	sandy silt to clayey silt
26.739	92.68	2.6069	2.813	-9.445	35	6	sandy silt to clayey silt
26.903	78.12	1.4342	1.836	-9.907	25	7	silty sand to sandy silt
27.067	63.07	0.9376	1.487	-10.125	20	7	silty sand to sandy silt
27.231	55.24	0.5677	1.028	-10.050	18	7	silty sand to sandy silt
27.395	52.12	0.4770	0.915	-9.989	17	7	silty sand to sandy silt
27.559	51.00	0.2987	0.586	-9.978	12	8	sand to silty sand
27.723	51.60	0.5385	1.044	-9.928	16	7	silty sand to sandy silt
27.887	57.19	0.3807	0.666	-9.930	14	8	sand to silty sand
28.051	64.86	0.5113	0.788	-9.864	16	8	sand to silty sand
28.215	69.39	0.4470	0.644	-9.860	17	8	sand to silty sand
28.379	70.58	0.5322	0.754	-9.787	17	8	sand to silty sand
28.543	66.61	0.7014	1.053	-9.758	21	7	silty sand to sandy silt
28.707	64.27	0.6786	1.056	-9.742	21	7	silty sand to sandy silt
28.871	62.37	0.6354	1.019	-9.683	20	7	silty sand to sandy silt
29.035	57.73	0.7414	1.284	-9.592	18	7	silty sand to sandy silt
29.199	59.79	0.7771	1.300	-9.579	19	7	silty sand to sandy silt
29.364	65.25	0.8480	1.300	-9.590	21	7	silty sand to sandy silt
29.528	60.34	0.7383	1.224	-5.476	19	7	silty sand to sandy silt
29.692	45.87	0.4470	0.975	-8.177	15	7	silty sand to sandy silt
29.856	37.60	0.3626	0.964	-8.572	12	7	silty sand to sandy silt
30.020	35.42	0.1734	0.490	-8.554	11	7	silty sand to sandy silt
30.184	33.77	0.2154	0.638	-8.526	11	7	silty sand to sandy silt
30.348	38.05	0.2471	0.649	-8.377	12	7	silty sand to sandy silt
30.512	38.22	0.1761	0.461	-8.213	12	7	silty sand to sandy silt
30.676	41.69	0.2023	0.485	-8.123	13	7	silty sand to sandy silt
30.840	37.54	0.1417	0.377	-8.127	12	7	silty sand to sandy silt
31.004	35.31	0.1870	0.530	-8.077	11	7	silty sand to sandy silt
31.168	33.67	0.2106	0.626	-7.962	11	7	silty sand to sandy silt

Depth ft	Tip (Qt) (tsf)	Sleeve Friction (Fs) (tsf)	F.Ratio (%)	PP (U2) (psi)	SPT (blows/ft)	Zone	Soil Behavior Type UBC-1983
31.332	37.09	0.1967	0.530	-7.789	12	7	silty sand to sandy silt
31.496	38.22	0.2527	0.661	-7.687	12	7	silty sand to sandy silt
31.660	51.97	0.3721	0.716	-7.628	17	7	silty sand to sandy silt
31.824	76.12	0.6738	0.885	-7.594	18	8	sand to silty sand
31.988	109.52	1.3362	1.220	-7.590	26	8	sand to silty sand
32.152	131.92	2.2031	1.670	-7.828	32	8	sand to silty sand
32.316	126.94	2.7188	2.142	-9.191	41	7	silty sand to sandy silt
32.480	136.57	1.9630	1.437	-9.814	33	8	sand to silty sand
32.644	122.67	1.7138	1.397	-9.923	29	8	sand to silty sand
32.808	114.75	2.1717	1.893	-7.084	37	7	silty sand to sandy silt
32.972	121.38	2.4159	1.990	-9.109	39	7	silty sand to sandy silt
33.136	108.59	2.3504	2.164	-9.740	35	7	silty sand to sandy silt
33.301	98.84	2.2771	2.304	-10.762	32	7	silty sand to sandy silt
33.465	113.95	1.8970	1.665	-10.456	36	7	silty sand to sandy silt
33.629	99.22	1.6836	1.697	-10.370	32	7	silty sand to sandy silt
33.793	72.11	1.8597	2.579	-10.259	28	6	sandy silt to clayey silt
33.957	54.51	1.4523	2.664	-10.327	21	6	sandy silt to clayey silt
34.121	66.89	1.1043	1.651	-10.184	21	7	silty sand to sandy silt
34.285	64.82	1.4727	2.272	-9.994	25	6	sandy silt to clayey silt
34.449	59.93	1.9166	3.198	-9.980	23	6	sandy silt to clayey silt
34.613	54.29	2.1486	3.957	-10.143	26	5	clayey silt to silty clay
34.777	54.38	2.2471	4.132	-10.082	26	5	clayey silt to silty clay
34.941	48.44	2.0089	4.147	-10.014	23	5	clayey silt to silty clay
35.105	52.62	1.9132	3.636	-10.189	25	5	clayey silt to silty clay
35.269	76.95	1.8090	2.351	-10.248	29	6	sandy silt to clayey silt
35.433	80.60	1.7793	2.208	-10.218	26	7	silty sand to sandy silt
35.597	81.35	2.1036	2.586	-10.279	31	6	sandy silt to clayey silt
35.761	86.77	1.5881	1.830	-10.225	28	7	silty sand to sandy silt
35.925	84.41	1.7470	2.070	-10.297	27	7	silty sand to sandy silt
36.253	71.12	1.2278	1.726	-6.499	23	7	silty sand to sandy silt
36.417	51.02	1.0469	2.052	-8.082	20	6	sandy silt to clayey silt
36.581	53.74	0.9941	1.850	-8.188	17	7	silty sand to sandy silt
36.745	66.63	0.6461	0.970	-8.275	16	8	sand to silty sand
36.909	52.14	0.8942	1.715	-8.336	17	7	silty sand to sandy silt
37.073	23.61	0.7585	3.212	-8.179	11	5	clayey silt to silty clay
37.238	19.32	0.4887	2.530	-7.891	9	5	clayey silt to silty clay
37.402	17.63	0.3396	1.926	-7.309	8	5	clayey silt to silty clay
37.566	20.04	0.3372	1.683	-6.971	8	6	sandy silt to clayey silt
37.894	20.21	0.3022	1.495	-6.773	8	6	sandy silt to clayey silt
38.058	22.19	0.4232	1.907	-6.497	8	6	sandy silt to clayey silt
38.222	31.41	0.5460	1.738	-6.295	12	6	sandy silt to clayey silt
38.386	61.98	1.2525	2.021	-6.340	20	7	silty sand to sandy silt
38.550	79.76	1.2967	1.626	-6.508	25	7	silty sand to sandy silt
38.714	90.39	1.3878	1.535	-7.068	29	7	silty sand to sandy silt
38.878	96.02	1.3929	1.451	-7.476	23	8	sand to silty sand
39.042	86.95	1.5404	1.772	-8.200	28	7	silty sand to sandy silt
39.206	75.16	1.9266	2.563	-8.515	29	6	sandy silt to clayey silt
39.370	71.93	1.7837	2.480	-6.805	28	6	sandy silt to clayey silt
39.534	69.63	1.0464	1.503	-8.080	22	7	silty sand to sandy silt
39.698	56.58	1.0979	1.940	-8.388	18	7	silty sand to sandy silt
39.862	30.15	1.0647	3.532	-8.259	14	5	clayey silt to silty clay
40.026	21.99	0.6054	2.753	-7.805	11	5	clayey silt to silty clay
40.354	22.64	0.3605	1.592	1.891	9	6	sandy silt to clayey silt

Depth ft	Tip (Qt) (tsf)	Sleeve Friction (Fs) (tsf)	F.Ratio (%)	PP (U2) (psi)	SPT (blows/ft)	Zone	Soil Behavior Type UBC-1983
40.518	17.94	0.2762	1.539	20.096	7	6	sandy silt to clayey silt
40.682	17.26	0.2622	1.519	37.074	7	6	sandy silt to clayey silt
40.846	16.77	0.2763	1.648	51.555	6	6	sandy silt to clayey silt
41.011	17.32	0.2638	1.523	60.184	7	6	sandy silt to clayey silt
41.175	19.00	0.2427	1.277	47.764	7	6	sandy silt to clayey silt
41.339	20.09	0.3155	1.571	45.326	8	6	sandy silt to clayey silt
41.503	21.97	0.2523	1.148	41.203	8	6	sandy silt to clayey silt
41.667	25.13	0.3160	1.258	31.461	10	6	sandy silt to clayey silt
41.831	21.65	0.2446	1.130	27.473	8	6	sandy silt to clayey silt
41.995	20.96	0.1522	0.726	31.468	8	6	sandy silt to clayey silt
42.159	17.59	0.1904	1.082	31.548	7	6	sandy silt to clayey silt
42.323	15.42	0.3052	1.979	49.934	7	5	clayey silt to silty clay
42.487	19.50	0.1892	0.970	32.727	7	6	sandy silt to clayey silt
42.651	16.80	0.2202	1.311	18.225	6	6	sandy silt to clayey silt
42.979	21.64	0.1700	0.786	30.203	8	6	sandy silt to clayey silt
43.143	18.78	0.2080	1.108	20.955	7	6	sandy silt to clayey silt
43.307	15.03	0.4290	2.854	29.663	7	5	clayey silt to silty clay
43.471	30.31	0.5682	1.875	43.992	12	6	sandy silt to clayey silt
43.635	34.64	0.6255	1.805	5.837	13	6	sandy silt to clayey silt
43.799	32.66	1.1204	3.431	12.583	16	5	clayey silt to silty clay
43.963	47.46	1.4715	3.100	-4.324	18	6	sandy silt to clayey silt
44.127	67.03	0.7169	1.069	-6.204	21	7	silty sand to sandy silt
44.291	59.33	0.6271	1.057	-8.962	19	7	silty sand to sandy silt
44.455	43.62	0.6083	1.395	-8.814	14	7	silty sand to sandy silt
44.619	34.13	0.4848	1.420	-8.465	13	6	sandy silt to clayey silt
44.783	29.55	0.5631	1.905	-8.086	11	6	sandy silt to clayey silt
44.948	37.53	0.4365	1.163	-7.701	12	7	silty sand to sandy silt
45.112	52.90	0.6391	1.208	-7.508	17	7	silty sand to sandy silt
45.276	44.26	1.0647	2.405	-7.837	17	6	sandy silt to clayey silt
45.440	34.59	0.9831	2.842	-8.327	13	6	sandy silt to clayey silt
45.604	44.88	0.3981	0.887	-8.511	14	7	silty sand to sandy silt
45.768	38.78	0.5088	1.312	-8.585	12	7	silty sand to sandy silt
45.932	22.87	0.6290	2.751	-5.379	11	5	clayey silt to silty clay
46.096	21.81	0.4902	2.247	-4.631	10	5	clayey silt to silty clay
46.260	21.96	0.5295	2.412	-4.023	11	5	clayey silt to silty clay
46.424	28.11	0.4575	1.628	-3.077	11	6	sandy silt to clayey silt
46.588	84.10	0.7323	0.871	-2.456	20	8	sand to silty sand
46.752	85.00	1.6997	2.000	-4.971	27	7	silty sand to sandy silt
46.916	69.74	2.1614	3.099	-9.159	27	6	sandy silt to clayey silt
47.080	56.18	1.8760	3.339	-9.431	27	5	clayey silt to silty clay
47.244	43.58	2.0323	4.664	-9.250	28	4	silty clay to clay
47.408	50.65	1.7777	3.510	-8.903	24	5	clayey silt to silty clay
47.572	42.59	1.6273	3.821	-8.513	20	5	clayey silt to silty clay
47.736	38.98	1.3858	3.555	-7.887	19	5	clayey silt to silty clay
47.900	33.03	1.2782	3.870	-7.520	16	5	clayey silt to silty clay
48.064	50.60	1.2577	2.486	-6.823	19	6	sandy silt to clayey silt
48.228	33.68	1.2064	3.582	-7.476	16	5	clayey silt to silty clay
48.392	36.18	0.8923	2.466	-6.973	14	6	sandy silt to clayey silt
48.556	24.96	0.5865	2.350	-6.966	10	6	sandy silt to clayey silt
48.720	22.25	0.8427	3.787	-6.427	14	4	silty clay to clay
48.885	34.00	0.9534	2.804	-6.277	13	6	sandy silt to clayey silt
49.049	40.78	1.0479	2.570	-6.315	16	6	sandy silt to clayey silt
49.213	33.38	1.1045	3.309	-2.494	16	5	clayey silt to silty clay

Depth ft	Tip (Qt) (tsf)	Sleeve Friction (Fs) (tsf)	F.Ratio (%)	PP (U2) (psi)	SPT (blows/ft)	Zone	Soil Behavior Type UBC-1983
49.377	26.40	0.8723	3.304	-1.581	13	5	clayey silt to silty clay
49.541	34.98	0.7378	2.109	-1.193	13	6	sandy silt to clayey silt
49.705	35.82	0.6049	1.689	-1.349	14	6	sandy silt to clayey silt
49.869	29.52	0.5188	1.758	-0.635	11	6	sandy silt to clayey silt
50.033	47.54	0.7090	1.491	-0.252	15	7	silty sand to sandy silt
50.197	56.17	0.9281	1.652	0.515	18	7	silty sand to sandy silt
50.361	55.34	2.2150	4.002	1.540	26	5	clayey silt to silty clay
50.525	73.96	2.8802	3.894	3.966	35	5	clayey silt to silty clay
50.689	75.66	3.3680	4.452	2.835	36	5	clayey silt to silty clay
50.853	74.90	3.3371	4.455	2.104	36	5	clayey silt to silty clay
51.017	71.30	3.2306	4.531	8.681	34	5	clayey silt to silty clay
51.181	99.20	3.8147	3.845	10.202	47	5	clayey silt to silty clay
51.345	109.85	4.2133	3.835	2.236	53	5	clayey silt to silty clay
51.509	114.25	5.5101	4.823	10.923	109	11	very stiff fine grained (*)
51.673	111.99	5.4532	4.869	1.009	107	11	very stiff fine grained (*)
51.837	99.63	4.6397	4.657	7.327	95	11	very stiff fine grained (*)
52.001	120.09	4.8183	4.012	-0.950	115	11	very stiff fine grained (*)
52.165	74.81	4.0080	5.358	-6.145	72	11	very stiff fine grained (*)
52.329	46.85	2.5381	5.418	-5.241	45	3	clay
52.493	96.65	3.0785	3.185	-0.050	37	6	sandy silt to clayey silt
52.657	124.69	4.2950	3.445	-4.837	48	6	sandy silt to clayey silt
52.822	96.17	4.9868	5.185	-6.837	92	11	very stiff fine grained (*)
52.986	59.28	4.2550	7.178	-8.086	57	11	very stiff fine grained (*)
53.150	50.40	2.7011	5.359	-7.424	48	3	clay
53.314	47.44	1.7963	3.786	-6.472	23	5	clayey silt to silty clay
53.478	46.40	1.4917	3.215	-5.944	22	5	clayey silt to silty clay
53.642	47.97	1.2561	2.619	-5.875	18	6	sandy silt to clayey silt
53.806	38.09	1.1504	3.020	-5.778	18	5	clayey silt to silty clay
53.970	37.81	1.2489	3.303	-5.807	18	5	clayey silt to silty clay
54.134	55.40	1.3570	2.450	-5.295	21	6	sandy silt to clayey silt
54.298	74.00	1.6578	2.240	-3.143	24	7	silty sand to sandy silt
54.462	72.67	1.7932	2.468	-1.687	28	6	sandy silt to clayey silt
54.626	77.68	1.8547	2.388	0.923	30	6	sandy silt to clayey silt
54.790	76.62	2.0044	2.616	1.660	29	6	sandy silt to clayey silt
54.954	76.76	2.1781	2.838	2.726	29	6	sandy silt to clayey silt
55.118	74.97	2.3682	3.159	1.653	29	6	sandy silt to clayey silt
55.282	80.79	2.9658	3.671	1.857	39	5	clayey silt to silty clay
55.446	86.13	3.9076	4.537	-1.755	82	11	very stiff fine grained (*)
55.610	89.41	3.6679	4.102	-2.551	43	5	clayey silt to silty clay
55.774	75.16	2.7949	3.718	-2.998	36	5	clayey silt to silty clay
55.938	77.39	2.3179	2.995	16.944	30	6	sandy silt to clayey silt
56.102	80.58	2.7032	3.355	5.148	31	6	sandy silt to clayey silt
56.266	78.71	3.2760	4.162	4.544	38	5	clayey silt to silty clay
56.430	75.98	3.7944	4.994	-2.939	73	11	very stiff fine grained (*)
56.594	71.93	4.3532	6.052	-5.179	69	11	very stiff fine grained (*)
56.759	91.47	4.4838	4.902	-6.306	88	11	very stiff fine grained (*)
56.923	90.38	4.7480	5.253	-6.549	87	11	very stiff fine grained (*)
57.087	93.84	4.6208	4.924	-6.540	90	11	very stiff fine grained (*)
57.251	112.73	4.2641	3.783	-4.878	43	6	sandy silt to clayey silt
57.415	94.31	3.5286	3.742	-1.370	45	5	clayey silt to silty clay
57.579	85.61	2.7341	3.194	2.578	33	6	sandy silt to clayey silt
57.743	81.95	2.9467	3.596	7.143	39	5	clayey silt to silty clay
57.907	100.50	2.9682	2.953	19.695	38	6	sandy silt to clayey silt

Depth ft	Tip (Qt) (tsf)	Sleeve Friction (Fs) (tsf)	F.Ratio (%)	PP (U2) (psi)	SPT (blows/ft)	Zone	Soil Behavior Type UBC-1983
58.071	102.58	3.5063	3.418	33.276	39	6	sandy silt to clayey silt
58.235	108.99	3.8338	3.517	40.816	42	6	sandy silt to clayey silt
58.399	152.31	6.8306	4.485	6.544	146	11	very stiff fine grained (*)
58.563	172.22	6.7534	3.921	-5.365	82	12	sand to clayey sand (*)
58.727	124.98	4.6761	3.742	9.293	48	6	sandy silt to clayey silt
58.891	110.33	3.6925	3.347	74.601	42	6	sandy silt to clayey silt
59.055	136.73	3.8129	2.789	65.571	44	7	silty sand to sandy silt
59.219	110.40	4.9780	4.509	10.536	106	11	very stiff fine grained (*)
59.383	109.71	5.9343	5.409	-3.252	105	11	very stiff fine grained (*)
59.547	98.26	5.7689	5.871	-5.910	94	11	very stiff fine grained (*)
59.711	103.45	5.2775	5.102	-7.098	99	11	very stiff fine grained (*)
59.875	119.36	4.7268	3.960	-7.458	57	5	clayey silt to silty clay
60.039	122.44	4.4394	3.626	-7.529	47	6	sandy silt to clayey silt
60.203	125.74	4.4912	3.572	-7.692	48	6	sandy silt to clayey silt
60.367	127.55	4.4512	3.490	-7.819	49	6	sandy silt to clayey silt

APPENDIX C

APPENDIX C

PREVIOUS EXPLORATIONS

This appendix contains logs of applicable explorations completed by NV5 as part of the SW Westgate Avenue bridge replacement over Beaverton Creek in 2011. Locations of the explorations are shown on Figures 2 and 3.

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	BLOW COUNT	INSTALLATION AND COMMENTS
0	ASPHALT CONCRETE (3 inches).	174.0; 173.7 / 0.3			
	AGGREGATE BASE (9 inches).	173.0 / 1.0			Driller Comment: gravel to 1.0 foot, then softer, silty gravel
	Medium stiff, brown-gray SILT with gravel to silty GRAVEL with sand (ML/GM); moist - **FILL**.	171.0 / 3.0		5	Driller Comment: smooth at 3.0 feet
5	Medium stiff, brown-orange CLAY (CL), some silt, trace fine sand; moist (alluvium).	169.0 / 5.0	ATT	13	LL = 41% PL = 26%
	Stiff, brown-orange SILT (ML), minor fine sand; moist.				
	grades to with fine sand at 7.5 feet			9	
10	grades to very stiff, light brown; moist to wet, stratified beds of silty sand to silt at 10.0 feet		DD HYD SIEV	18	DD = 84 pcf Switch to 4 7/8-inch drag bit 11.5 feet, during drilling
15	grades to stiff, light brown-orange, with fine sand; moist at 15.0 feet		P200	9	P200 = 71% Driller Comment: firmer at 18.0 feet
20	Very stiff, light gray-blue SILT (ML), minor sand; moist to wet, stratified beds of silt with sand to silty sand (3 to 5 inches) (Missoula Flood Deposits).	154.0 / 20.0	P200	24	P200 = 88%
			DD CON		DD = 83 pcf
25	moist at 25.0 feet			28	
30	grades to with sand; moist to wet at 30.0 feet			20	Driller Comment: small zone of gravel approx. 12 inches thick observed in cuttings at 32.0 feet
35	grades to gray, trace fine sand, some clay; moist, homogenous at 35.0 feet			18	
40					

Legend: ▲ BLOW COUNT ● MOISTURE CONTENT % RQD% CORE REC% (scale 0 – 50 – 100)

DRILLED BY: Western States Soil Conservation, Inc. LOGGED BY: JGH COMPLETED: 01/04/11

BORING METHOD: mud rotary (see report text) BORING BIT DIAMETER: 4 7/8-inch

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BORING B-1

ROSE BIGGI AVE (HALL BLVD - CRESCENT ST)
BEAVERTON, OR

FIGURE A-1

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	BLOW COUNT	INSTALLATION AND COMMENTS
40	grades to stiff, minor fine sand; moist to wet at 40.0 feet			11	
45	Dense, light gray-brown, fine to medium, silty SAND (SM), minor clay; moist, laminated to stratified beds of medium sand, silty sand to sandy silt (Sandy River Mudstone).	129.0 / 45.0	DD DS	76	DD = 109 pcf
50	Dense, light gray-brown, clayey SAND (SC), some silt; moist.	124.0 / 50.0		38	
55	Very stiff, brown-gray CLAY (CH); moist.	119.0 / 55.0		20	
60			ATT	26	LL = 64% PL = 26%
65	Dense, gray-orange, clayey SAND (SC), minor silt; moist.	109.0 / 65.0		34	
70	Hard, light gray-orange CLAY with sand (CL), minor silt, trace gravel; moist.	104.0 / 70.0		71	
	Very dense, orange-brown, clayey SAND (SC), minor silt; moist to wet.	103.0 / 71.0			Driller Comment: scattered gravel from 72.0 to 73.0 feet
75	Hard, light gray-orange SILT with fine sand (ML), some clay; moist.	99.0 / 75.0		59	
	Very dense, orange, clayey SAND with silt (SC); moist to wet.	98.0 / 76.0			Driller Comment: little softer at 78.0 feet
80					

Legend: ▲ BLOW COUNT ● MOISTURE CONTENT % RQD% CORE REC% (scale 0 – 50 – 100)

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BORING METHOD: mud rotary (see report text) BORING BIT DIAMETER: 4 7/8-inch

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BORING B-1
(continued)

ROSE BIGGI AVE (HALL BLVD - CRESCENT ST)
BEAVERTON, OR

FIGURE A-1

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION / DEPTH	BLOW COUNT
80	Stiff to very stiff, gray to dark gray SILT with fine sand (ML), some clay; moist.	94.0 / 80.0	15
85	grades to very stiff, minor fine sand, trace organics (rootlets); moist to wet, laminated to stratified beds of sand at 85.0 feet		16
90	Hard, gray-light gray CLAY (CH), trace fine sand; moist.	84.0 / 90.0	33
100	without sand at 100.0 feet		31
	Exploration completed at a depth of 101.5 feet.	72.5 / 101.5	

Column headings: DEPTH FEET; GRAPHIC LOG; MATERIAL DESCRIPTION; ELEVATION DEPTH; TESTING; SAMPLE; ▲ BLOW COUNT ● MOISTURE CONTENT % RQD% CORE REC% (scale 0, 50, 100); INSTALLATION AND COMMENTS

DRILLED BY: Western States Soil Conservation, Inc. LOGGED BY: JGH COMPLETED: 01/04/11

BORING METHOD: mud rotary (see report text) BORING BIT DIAMETER: 4 7/8-inch

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BORING B-1
(continued)

ROSE BIGGI AVE (HALL BLVD - CRESCENT ST)
BEAVERTON, OR

FIGURE A-1

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	BLOW COUNT	INSTALLATION AND COMMENTS
0	ASPHALT CONCRETE (3 inches).	174.0 / 173.7 / 0.3			
	AGGREGATE BASE (9 inches).	173.0 / 1.0			Driller Comment: smoother at 1.0 foot with scattered gravel
	Stiff, brown-gray SILT with sand (ML), minor gravel, trace organics (rootlets); moist - **FILL**.			12	
5	without gravel at 5.0 feet			8	
	Medium stiff to stiff, dark gray-orange CLAY (CL/CH), trace organics (roots, 1.4-inch diameter); moist.	168.3 / 5.7			
	Stiff to very stiff, brown-orange SILT with sand (ML); moist.	165.0 / 9.0	CON ATT		LL = 49% PL = 22%
10	becomes sandy; moist to wet at 11.0 feet		P200	15	P200 = 74%
					Color change(to dark gray-blue) in cuttings at 13.0 feet
15	grades to stiff, dark gray-blue, with fine sand; stratified beds of silt and silty sand (3 to 6 inches) at 15.0 feet			13	
			DD DS		DD = 84 pcf
20	Very stiff, dark gray-blue SILT with fine sand; moist (Missoula Flood Deposits).	154.0 / 20.0		20	
25	grades to gray SILT, minor fine sand to SILT with fine sand; moist, homogenous at 25.0 feet			25	
30	grades to hard at 30.0 feet			33	
35	grades to stiff, some clay, minor fine sand at 35.0 feet		ATT	11	LL = 32% PL = 25%
			DD		DD = 94 pcf
40					

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

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BORING METHOD: mud rotary (see report text) BORING BIT DIAMETER: 4 7/8-inch

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BORING B-2

ROSE BIGGI AVE (HALL BLVD - CRESCENT ST)
BEAVERTON, OR

FIGURE A-2

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE	▲ BLOW COUNT ● MOISTURE CONTENT % RQD% CORE REC% (0 – 50 – 100)	INSTALLATION AND COMMENTS
40		grades to moist to wet at 40.0 feet				11	
45		grades to very stiff, minor clay; laminated to stratified beds of orange-brown, medium sand and light-gray, silty sand to sandy silt at 45.0 feet		P200		25	P200 = 64%
50		Very stiff, light gray-brown SILT with fine sand (ML), some clay; moist.	124.0 / 50.0			26	
55		Hard, brown-light gray CLAY (CH); moist (Sandy River Mudstone).	119.0 / 55.0			80	
60		grades to very stiff, light gray-brown with orange mottles, trace sand at 60.0 feet				26	
65		Dense, orange-gray, clayey SAND with silt (SC); moist, laminated to stratified beds of sand with clay, clay, and silt.	109.0 / 65.0			45	
70						43	
75		Very stiff, light brown-orange CLAY (CH); moist.	99.0 / 75.0			23	
		Very stiff, gray-dark gray SILT (ML), some clay, trace sand; moist.	98.0 / 76.0				
80							

DRILLED BY: Western States Soil Conservation, Inc.   LOGGED BY: JGH   COMPLETED: 01/05/11

BORING METHOD: mud rotary (see report text)   BORING BIT DIAMETER: 4 7/8-inch

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BORING B-2
(continued)

ROSE BIGGI AVE (HALL BLVD - CRESCENT ST)
BEAVERTON, OR

FIGURE A-2

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	SAMPLE	▲ BLOW COUNT ● MOISTURE CONTENT % RQD% CORE REC% (0 50 100)	INSTALLATION AND COMMENTS
80		Very dense, dark gray-gray, fine to medium SAND with clay (SP), minor silt; moist to wet.	94.0 / 80.0			62	
85						29	
		Very stiff, light gray SILT (ML), minor fine sand; moist.	85.5 / 88.5				Driller Comment: clayey at 88.0 feet
90		Very stiff, gray-dark gray CLAY (CH); moist.	83.5 / 90.5			26	
95							
100		Very stiff, light gray-orange SILT with fine sand (ML), some clay; moist.	74.0 / 100.0			20	Driller Comment: consistent drilling to 100.0 feet
		Exploration completed at a depth of 101.5 feet.	72.5 / 101.5				
105							
110							
115							
120							

DRILLED BY: Western States Soil Conservation, Inc. LOGGED BY: JGH COMPLETED: 01/05/11

BORING METHOD: mud rotary (see report text) BORING BIT DIAMETER: 4 7/8-inch

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BORING B-2
(continued)

ROSE BIGGI AVE (HALL BLVD - CRESCENT ST)
BEAVERTON, OR

FIGURE A-2

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION / DEPTH	TESTING	BLOW COUNT	INSTALLATION AND COMMENTS
0	ASPHALT CONCRETE (2.75 inches).	174.0 / 173.8 / 0.2			
	AGGREGATE BASE (3.25 inches).	173.5 / 0.5			
	Medium stiff, gray to dark gray SILT with gravel (ML); moist - **FILL**.			7	Very smooth drilling below asphalt concrete
5	becomes soft to medium stiff, some clay, minor sand, trace organics (woody debris up to 1/4-inch diameter) at 5.0 feet			10	Driller Comment: something hard at 5.0 feet
	Soft to medium stiff, gray SILT with fine sand (ML), trace organics (woody debris up to 1/8-inch diameter); moist (alluvium).	166.5 / 7.5		4	
10	Soft to medium stiff, gray CLAY (CL), some silt, minor fine sand, trace organics (rootlets); moist.	165.5 / 8.5			
	Stiff, brown-gray SILT with fine sand (ML), some clay; moist to wet, stratified bed of wet silty sand (6 inches).	162.0 / 12.0		10	Color change from gray to brown in cuttings at 11.5 feet
15	Medium stiff to stiff, gray SILT with fine sand (ML) to fine, silty SAND (SM); wet, stratified beds of silt with sand and silty sand (up to 5 inches).	159.0 / 15.0		8	
			DD CON		DD = 89 pcf
20	Very stiff, gray SILT with fine sand (ML); moist to wet, stratified beds of wet sand (up to 4 inches).	154.0 / 20.0		27	
25	becomes light gray, some clay; moist at 25.0 feet			29	
30	grades to minor fine sand at 30.0 feet			24	
	Medium dense, gray, fine, silty SAND (SM); wet.	143.0 / 31.0			
35	Soft, gray SILT (ML); moist, homogenous.	139.0 / 35.0		3	
40					

▲ BLOW COUNT ● MOISTURE CONTENT % RQD% CORE REC% (scale 0 – 50 – 100)

DRILLED BY: Western States Soil Conservation, Inc. LOGGED BY: JGH COMPLETED: 06/14/11

BORING METHOD: mud rotary (see report text) BORING BIT DIAMETER: 3 7/8-inch

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BORING B-3

ROSE BIGGI AVE (HALL BLVD - CRESCENT ST)
BEAVERTON, OR

FIGURE A-3

DEPTH FEET	GRAPHIC LOG	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	SAMPLE	▲ BLOW COUNT ● MOISTURE CONTENT % RQD% CORE REC% (0 – 50 – 100)	INSTALLATION AND COMMENTS
40		Soft to medium stiff, gray SILT (ML), trace fine sand; moist, homogenous.			P	4	
45		Medium dense, brown-gray, fine to medium, silty SAND (SM); wet.	129.0 45.0			18	
		Exploration completed at a depth of 46.5 feet.	127.5 46.5				
50							
55							
60							
65							
70							
75							
80							

DRILLED BY: Western States Soil Conservation, Inc. LOGGED BY: JGH COMPLETED: 06/14/11

BORING METHOD: mud rotary (see report text) BORING BIT DIAMETER: 3 7/8-inch

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BORING B-3
(continued)

ROSE BIGGI AVE (HALL BLVD - CRESCENT ST)
BEAVERTON, OR

FIGURE A-3

C-1

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	BLOW COUNT / MOISTURE CONTENT %	COMMENTS
0.0	ASPHALT CONCRETE (3 inches).	0.3			
	AGGREGATE BASE.	1.2			
	Medium dense, brown-gray, silty GRAVEL with sand (GM); moist - **FILL**.			25	
2.5					
	Soft, gray SILT with fine sand (ML); moist.	3.5			Driller Comment: smooth, out of gravel at 3.5 feet
5.0			P200	12	P200 = 73% Hard-like concrete, smooth at 6.5 feet
	Exploration terminated due to possible utility/debris at a depth of 6.5 feet.	6.5			Surface elevation was not measured at the time of exploration.
7.5					
10.0					
12.5					

C-2

DEPTH FEET	MATERIAL DESCRIPTION	ELEVATION DEPTH	TESTING	BLOW COUNT / MOISTURE CONTENT %	COMMENTS
0.0	ASPHALT CONCRETE (5.5 inches).	0.5			
	AGGREGATE BASE.	1.0			
	Very stiff, gravelly SILT (ML), some clay, trace organics (woody debris, 1/4-inch diameter); moist - **FILL**.			18	
2.5					
	Medium stiff, gray-dark gray SILT (ML), minor sand and organics (woody debris, up to 1/4-inch diameter), trace gravel; moist - **FILL**.	4.5			Driller Comment: smoother and softer at 4.5 feet
5.0				5	
7.5	Medium stiff, dark gray CLAY (CH), trace organics (rootlets); moist (alluvium). some silt at 8.5 feet	7.5	ATT	6	LL = 49% PL = 19%
10.0	Medium stiff, orange-brown mottled gray SILT (ML), some clay, minor fine sand, trace organics (rootlets); moist.	10.0		7	Surface elevation was not measured at the time of exploration.
	Exploration completed at a depth of 11.5 feet.	11.5			
12.5					

9.0 feet, after drilling

DRILLED BY: Dan J. Fischer Excavating, Inc. LOGGED BY: JGH COMPLETED: 12/21/10

BORING METHOD: solid-stem auger (see report text) BORING BIT DIAMETER: 4-inch

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

BEAVERTON-28-01

SEPTEMBER 2011

BORING

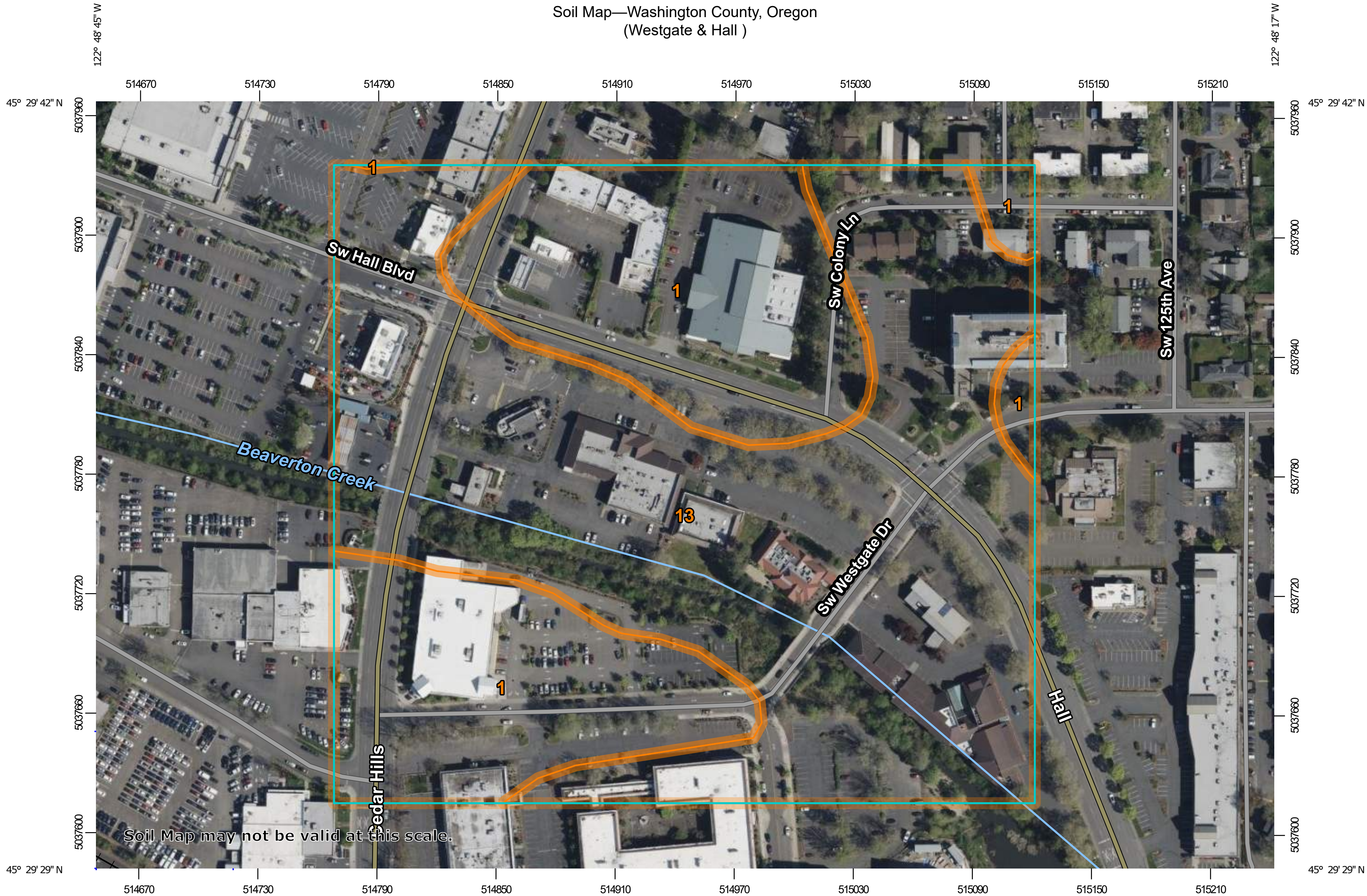
ROSE BIGGI AVE (HALL BLVD - CRESCENT ST)
BEAVERTON, OR

FIGURE A-5

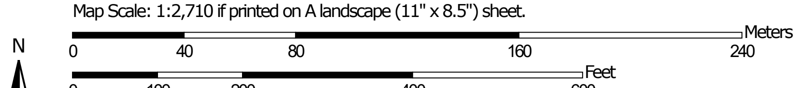
NV5

Delivering Solutions
Improving Lives

Soil Map—Washington County, Oregon
(Westgate & Hall)



Map Scale: 1:2,710 if printed on A landscape (11" x 8.5") sheet.



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

MAP LEGEND

Area of Interest (AOI)

Area of Interest (AOI)

Soils

Soil Map Unit Polygons

Soil Map Unit Lines

Soil Map Unit Points

Special Point Features

Blowout

Borrow Pit

Clay Spot

Closed Depression

Gravel Pit

Gravelly Spot

Landfill

Lava Flow

Marsh or swamp

Mine or Quarry

Miscellaneous Water

Perennial Water

Rock Outcrop

Saline Spot

Sandy Spot

Severely Eroded Spot

Sinkhole

Slide or Slip

Sodic Spot

Spoil Area

Stony Spot

Very Stony Spot

Wet Spot

Other

Special Line Features

Water Features

Streams and Canals

Transportation

Rails

Interstate Highways

US Routes

Major Roads

Local Roads

Background

Aerial Photography

MAP INFORMATION

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

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

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

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

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

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

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

Soil Survey Area: Washington County, Oregon
Survey Area Data: Version 22, Sep 14, 2022

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

Date(s) aerial images were photographed: Apr 16, 2021—Apr 18, 2021

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

Map Unit Legend

Map Unit Symbol	Map Unit Name	Acres in AOI	Percent of AOI
1	Aloha silt loam	10.6	37.6%
13	Cove silty clay loam	17.5	62.4%
Totals for Area of Interest		**28.1**	**100.0%**

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Appendix C:

C1 Assumptions

C2 Hydromodification Risk Level

C3 Hydraflow Report - Hydromodification Calculations

Appendix C1

Assumptions

Santa Barbara Unit Hydrograph (SBUH) Assumptions:

2 yr event	**2.5**	in/24-hours
5 yr event	**3.1**	in/24-hours
10-year	**3.45**	in/24-hours
25-yr	**3.90**	in/24-hours
Roughness Coefficient	**0.013**	

Curve Number Assumptions:

Impervious Area =	**98**	Per City of Beaverton Engineering Design manual - Pavement
Pervious Area (Post-Dveloped) Lawn =	**86**	Per City of Beaverton Engineering Design manual - Hydrologic soil group C, Open Space grass in good condition
Pervious Area (pre-developed) =	**79**	Per NRCS - USDA Hydrologic soil group D, Woods Fair condition
Pervious Area (post-developed) =	**73**	Due to enhanced plantings

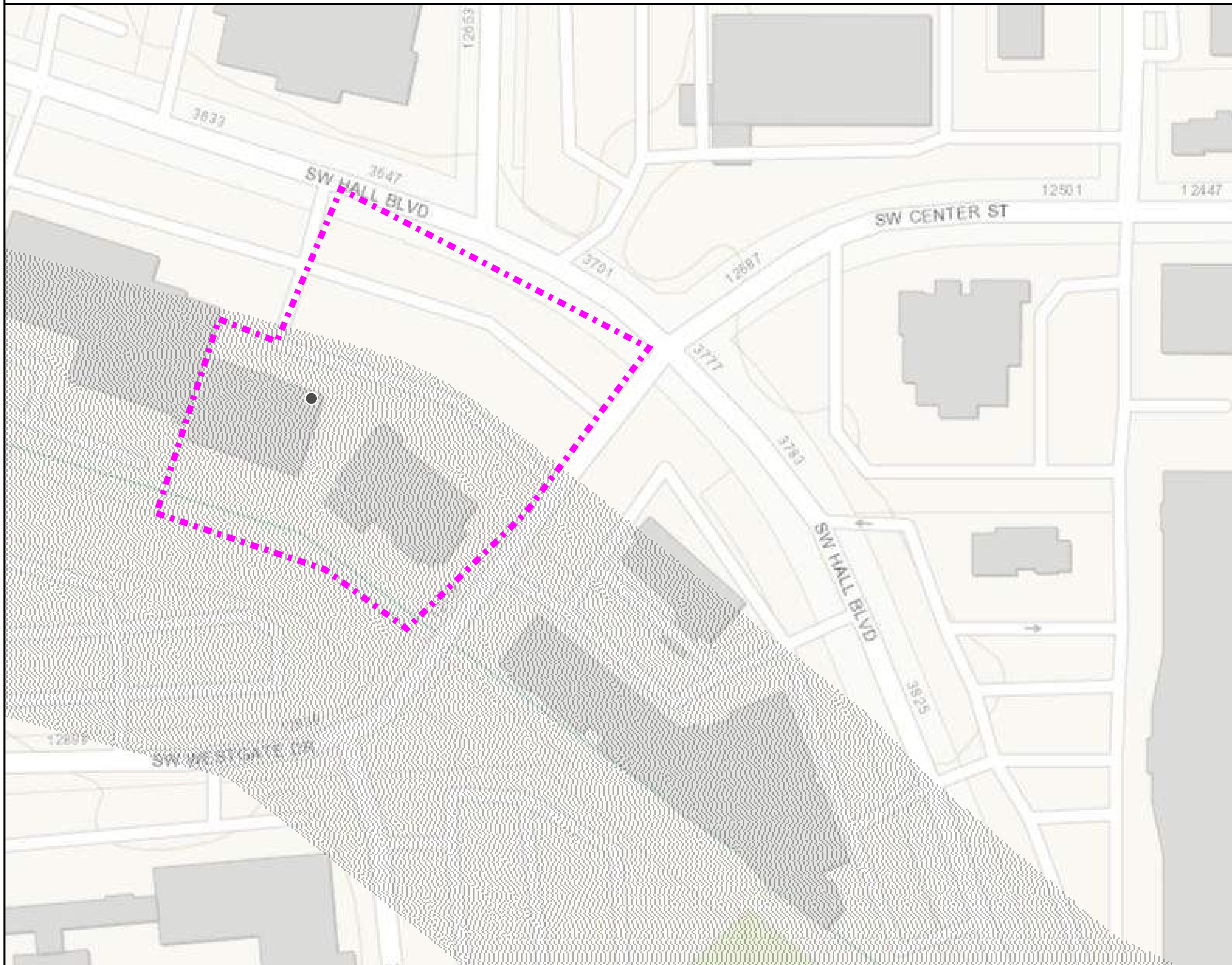
Rational Method Assumptions:
(used for conveyance calcs)

25-year Storm Event =	**2.6**	in/hours	Per odot zone 8
Roughness Coefficient	**0.013**		

Runoff Coefficient

Impervious Area =	**0.9**	Per BES SWMM Urban Areas - Sidewalk/Pavement
Pervious Area =	**0.35**	Per BES SWMM Urban Areas - Open Space

Westgate & Hall : Hydro Modification Risk Level Low



Legend

- Cleanout
- Fitting
- Valve
- Manhole
- Inlet
- Pond
- Vault
- Treatment Plant
- Pump Station
- Gravity Line
- Pressure Line
- Open Channel
- Virtual Flow
- Pond Outline
- CWS Boundary
- County Boundary
- Urban Growth Boundary

Hydromodification Protection Risk Levels

- Low
- Moderate
- High

PROJECT LIMITS

; $EDQGRQHG

CWS Sani | CWS Storm
Partner Sani | Partner Storm

'LVFODLPHU 7KLV SURGXFW DQG LWV DVVRFLDWHG GDWD LV IRU LQIRUPDWLRQDO SXUSRVHV RQO\ DQG ZDV GHULYHG IURP VHYHUDO GDWDEDVH
RI WKLV LQIRUPDWLRQ VKRXOG UHYLHZ RU FRQVXOW WKH SULPDU\ GDWD DQG LQIRUPDWLRQ VRXUFHV WR HQVXUH DFFXUDF\ &OHDQ :DWHU
ZDUUDQWLHV IRU WKLV SURGXFW 0DLQOLQH DQG VHUYLFH ODWHUDO ORFDWLRQV DUH GHSLFWHG XVLQJ EHVW DYDLODEOH LQIRUPDWLRQ
³8QORFDWDEOH XQGHUJURXQG IDFLOLWLHV´ DV GHILQHG LQ 2$5 (DVHPHQW GDWD LV QRW FXUUHQWO\ FRPSOHWHG 'LVWULFW Z
WKH H[FHSWLRQ RI VDQLWDU\ OLQHV ´ DQG ODUJHU ORFDWHG ZLWKLQ WKH FLW\ OLPLWV RI %HDYHUWRQ &RUQHOLXV)RUHVW *URYH +LOOVERUR
LQGLYLGXDO FLW\ 1RWLILFDWLRQ RI DQ\ HUURUV ZRXOG EH DSSUHFLDWHG &OHDQ :DWHU 6HUYLFHV 'HYHORSPHQW 6HUYLFHV 6: +LOOVERUR

EXHIBIT E-2

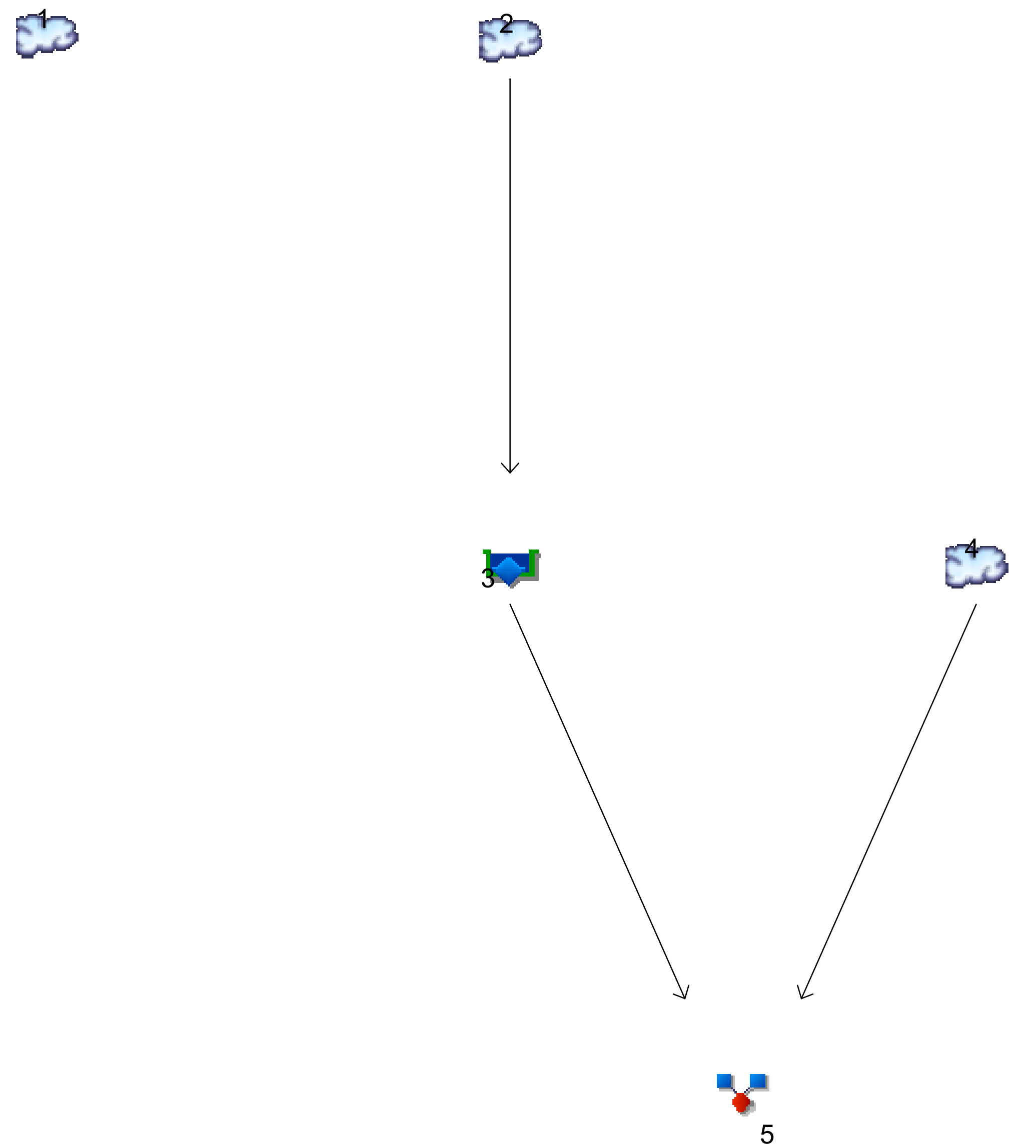
Clean Water Services

Hydraflow Table of Contents

Contents continued...

Watershed Model Schematic

Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2022



Project: westgate.gpw	Monday, 04 / 17 / 2023

Hydrograph Return Period Recap

Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2022

Hyd. No.	Hydrograph type (origin)	Inflow hyd(s)	Peak Outflow (cfs)								Hydrograph Description
			1-yr	2-yr	3-yr	5-yr	10-yr	25-yr	50-yr	100-yr	
1	SBUH Runoff	------	-------	0.317	-------	0.533	0.670	0.854	-------	1.070	Pre-Dev
2	SBUH Runoff	------	-------	0.680	-------	0.888	1.010	1.165	-------	1.337	Post-Dev-A
3	Reservoir	2	-------	0.080	-------	0.176	0.247	0.446	-------	0.858	Post-thru-DG
4	SBUH Runoff	------	-------	0.120	-------	0.190	0.234	0.292	-------	0.360	Post-Dev-B&C
5	Combine	3, 4	-------	0.161	-------	0.239	0.318	0.574	-------	1.102	Post-Dev-combined

Hydrograph Summary Report

Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2022

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	SBUH Runoff	0.317	2	480	5,989	------	------	------	Pre-Dev
2	SBUH Runoff	0.680	2	476	9,550	------	------	------	Post-Dev-A
3	Reservoir	0.080	2	1240	9,531	2	175.95	5,603	Post-thru-DG
4	SBUH Runoff	0.120	2	480	2,077	------	------	------	Post-Dev-B&C
5	Combine	0.161	2	480	11,609	3, 4	------	------	Post-Dev-combined

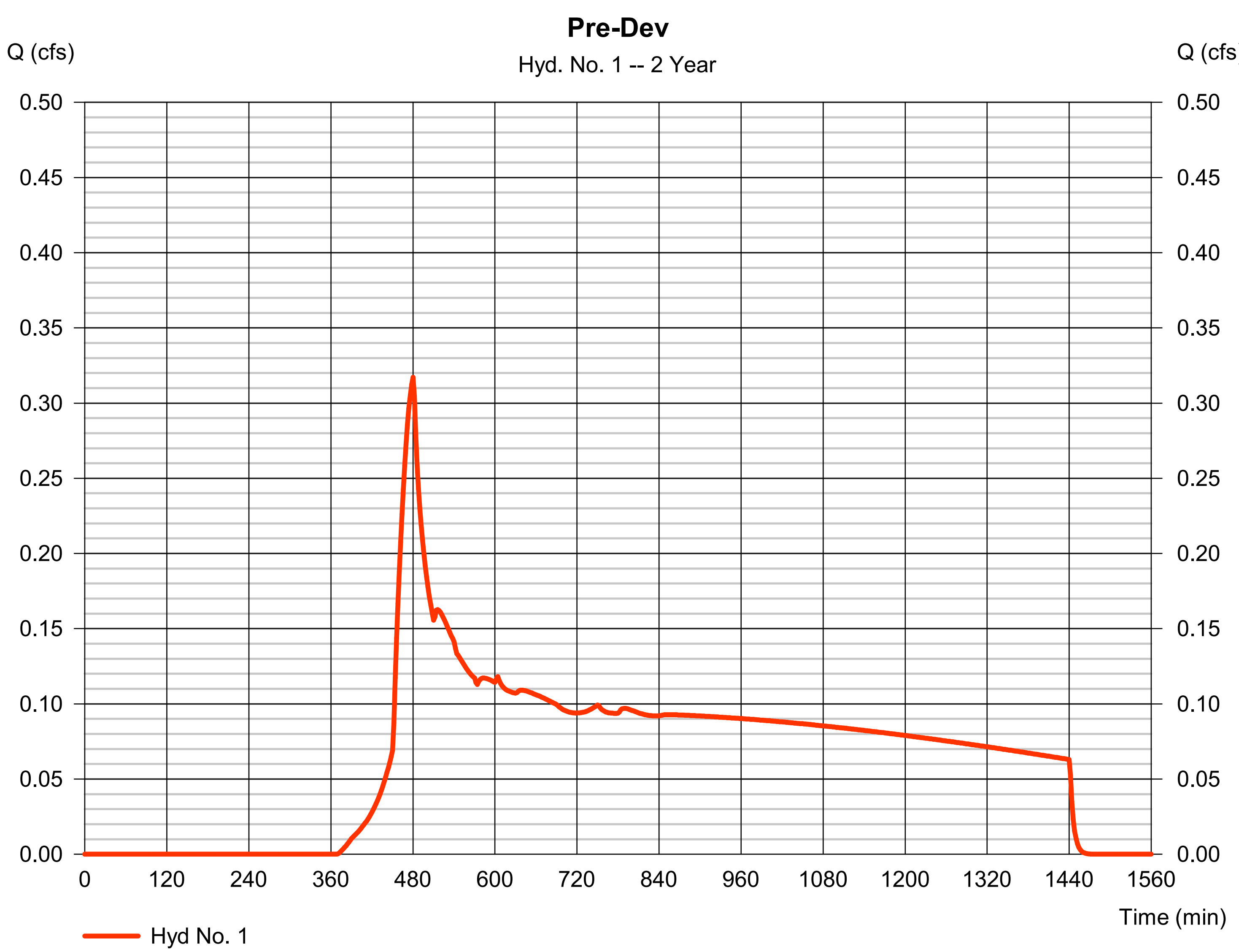
Hydrograph Report

Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2022 Monday, 04 / 17 / 2023

Hyd. No. 1

Pre-Dev

Hydrograph type	= SBUH Runoff	Peak discharge	= 0.317 cfs
Storm frequency	= 2 yrs	Time to peak	= 480 min
Time interval	= 2 min	Hyd. volume	= 5,989 cuft
Drainage area	= 1.970 ac	Curve number	= 79
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	= n/a



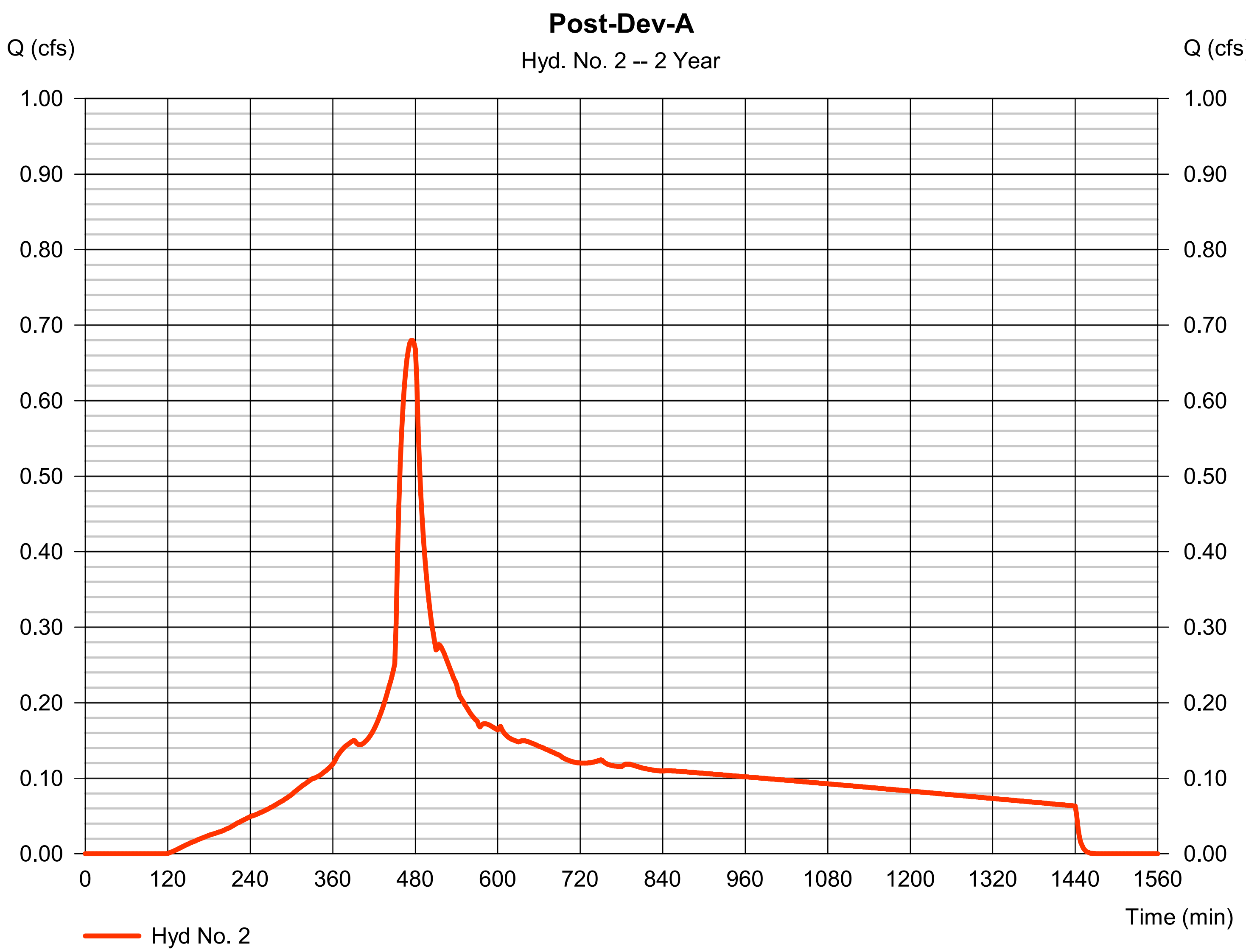
Hydrograph Report

Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2022 — Monday, 04 / 17 / 2023

Hyd. No. 2

Post-Dev-A

Hydrograph type	= SBUH Runoff	Peak discharge	= 0.680 cfs
Storm frequency	= 2 yrs	Time to peak	= 476 min
Time interval	= 2 min	Hyd. volume	= 9,550 cuft
Drainage area	= 1.380 ac	Curve number	= 94.4
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	= n/a



Hydrograph Report

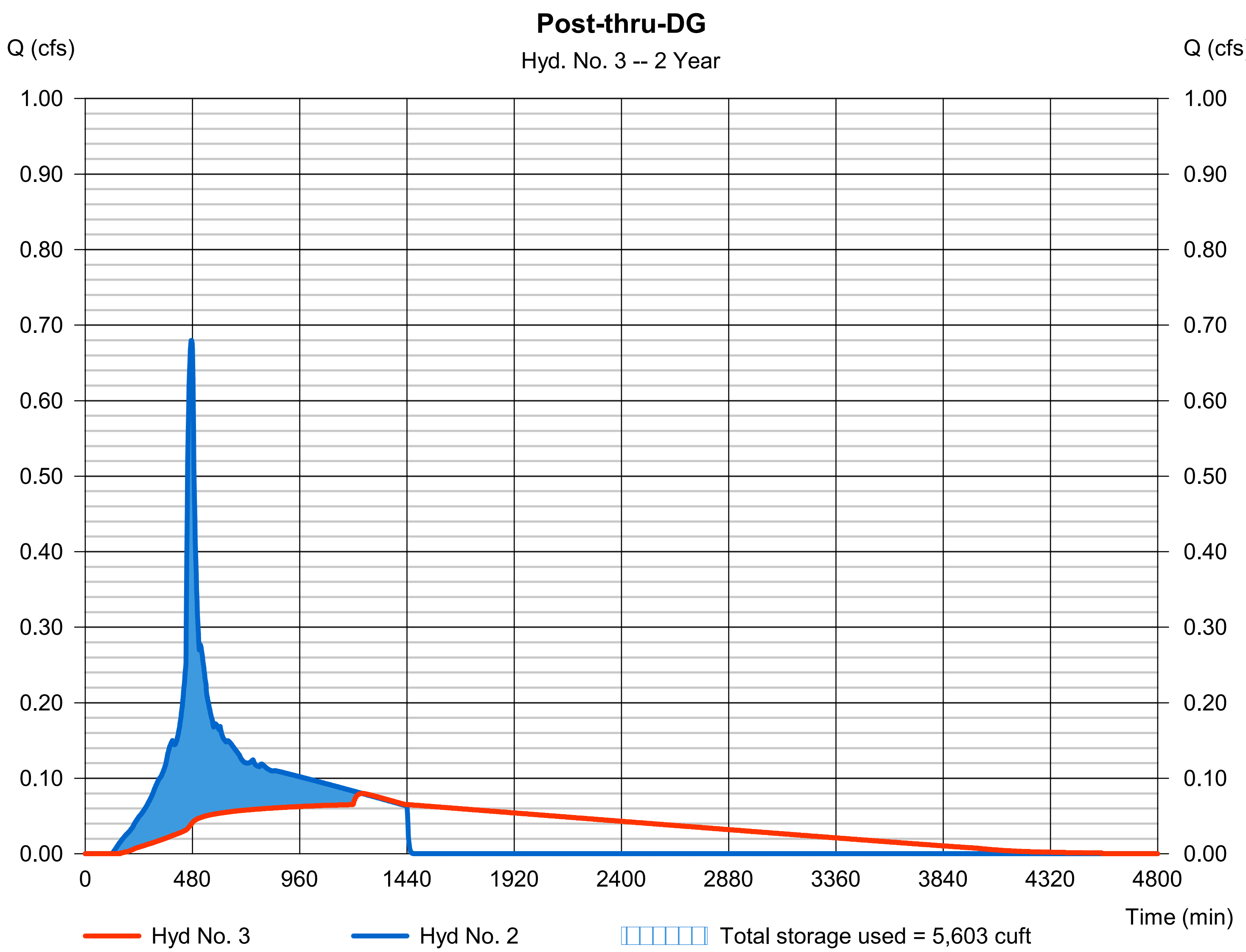
Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2022 Monday, 04 / 17 / 2023

Hyd. No. 3

Post-thru-DG

Hydrograph type	= Reservoir	Peak discharge	= 0.080 cfs
Storm frequency	= 2 yrs	Time to peak	= 1240 min
Time interval	= 2 min	Hyd. volume	= 9,531 cuft
Inflow hyd. No.	= 2 - Post-Dev-A	Max. Elevation	= 175.95 ft
Reservoir name	= DG	Max. Storage	= 5,603 cuft

Storage Indication method used.



Pond Report

Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2022 Monday, 04 / 17 / 2023

Pond No. 1 - DG

Pond Data

UG Chambers -Invert elev. = 170.75 ft, Rise x Span = 5.58 x 9.98 ft, Barrel Len = 107.91 ft, No. Barrels = 1, Slope = 0.00%, Headers = No

Stage / Storage Table

Stage (ft)	Elevation (ft)	Contour area (sqft)	Incr. Storage (cuft)	Total storage (cuft)
0.00	170.75	n/a	0	0
0.56	171.31	n/a	601	601
1.12	171.87	n/a	601	1,202
1.67	172.42	n/a	601	1,803
2.23	172.98	n/a	601	2,404
2.79	173.54	n/a	601	3,005
3.35	174.10	n/a	601	3,606
3.91	174.66	n/a	601	4,207
4.46	175.21	n/a	601	4,808
5.02	175.77	n/a	601	5,409
5.58	176.33	n/a	601	6,011

Culvert / Orifice Structures

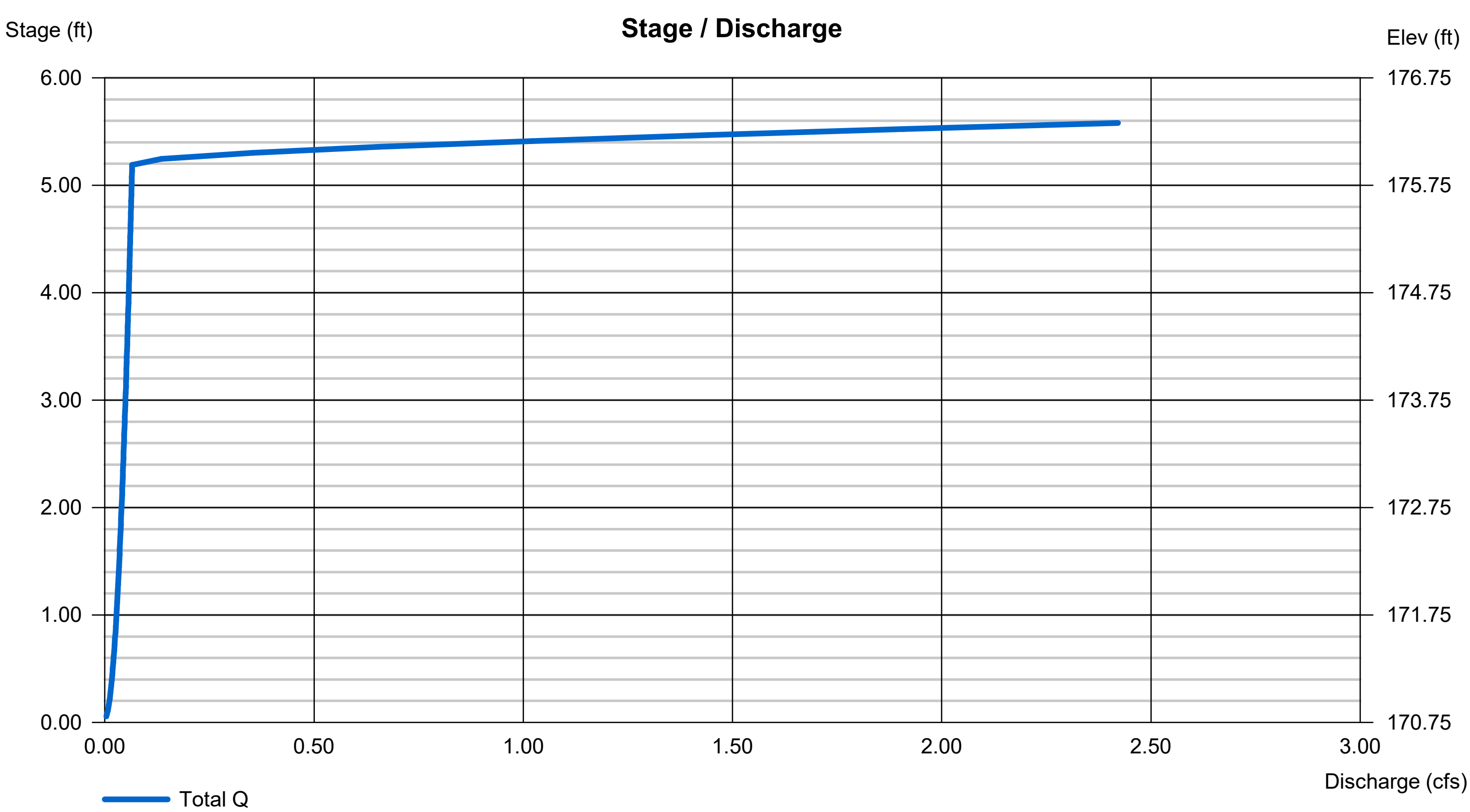
	[A]	[B]	[C]	[PrfRsr]
Rise (in)	= 8.00	1.05	0.00	0.00
Span (in)	= 8.00	1.05	0.00	0.00
No. Barrels	= 1	1	0	0
Invert El. (ft)	= 170.75	170.75	0.00	0.00
Length (ft)	= 0.00	0.00	0.00	0.00
Slope (%)	= 0.00	0.00	0.00	n/a
N-Value	= .013	.013	.013	n/a
Orifice Coeff.	= 0.60	0.60	0.60	0.60
Multi-Stage	= n/a	Yes	No	No

Weir Structures

	[A]	[B]	[C]	[D]
Crest Len (ft)	= 3.14	0.00	0.00	0.00
Crest El. (ft)	= 175.96	0.00	0.00	0.00
Weir Coeff.	= 3.33	3.33	3.33	3.33
Weir Type	= 1	---	---	---
Multi-Stage	= No	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).



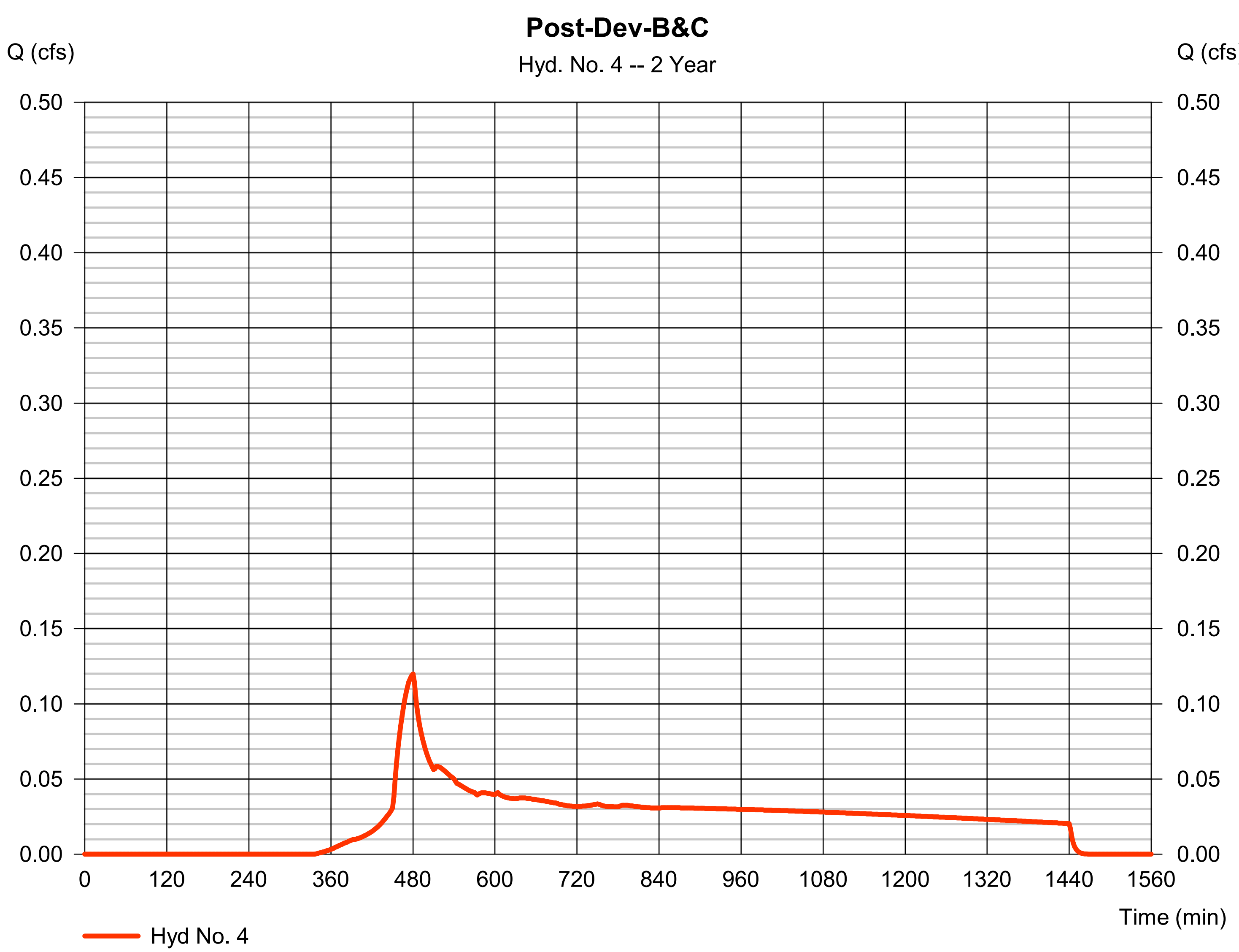
Hydrograph Report

Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2022 Monday, 04 / 17 / 2023

Hyd. No. 4

Post-Dev-B&C

Hydrograph type	= SBUH Runoff	Peak discharge	= 0.120 cfs
Storm frequency	= 2 yrs	Time to peak	= 480 min
Time interval	= 2 min	Hyd. volume	= 2,077 cuft
Drainage area	= 0.590 ac	Curve number	= 81.5
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	= n/a



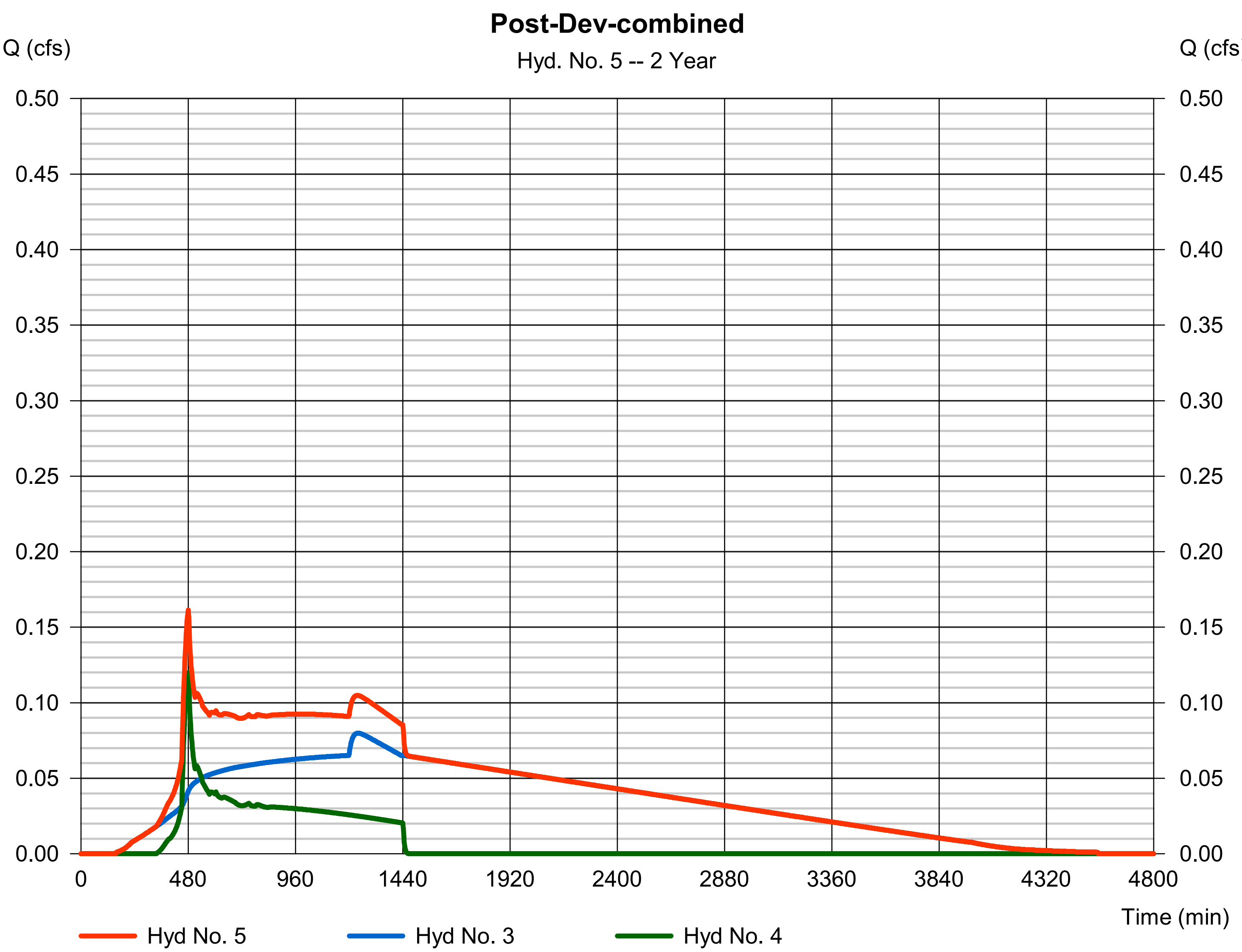
Hydrograph Report

Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2022 — Monday, 04 / 17 / 2023

Hyd. No. 5

Post-Dev-combined

Hydrograph type	= Combine	Peak discharge	= 0.161 cfs
Storm frequency	= 2 yrs	Time to peak	= 480 min
Time interval	= 2 min	Hyd. volume	= 11,609 cuft
Inflow hyds.	= 3, 4	Contrib. drain. area	= 0.590 ac



Hydrograph Summary Report

Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2022

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	SBUH Runoff	0.533	2	480	9,025	------	------	------	Pre-Dev
2	SBUH Runoff	0.888	2	474	12,456	------	------	------	Post-Dev-A
3	Reservoir	0.176	2	670	12,437	2	176.01	5,661	Post-thru-DG
4	SBUH Runoff	0.190	2	480	3,050	------	------	------	Post-Dev-B&C
5	Combine	0.239	2	480	15,488	3, 4	------	------	Post-Dev-combined

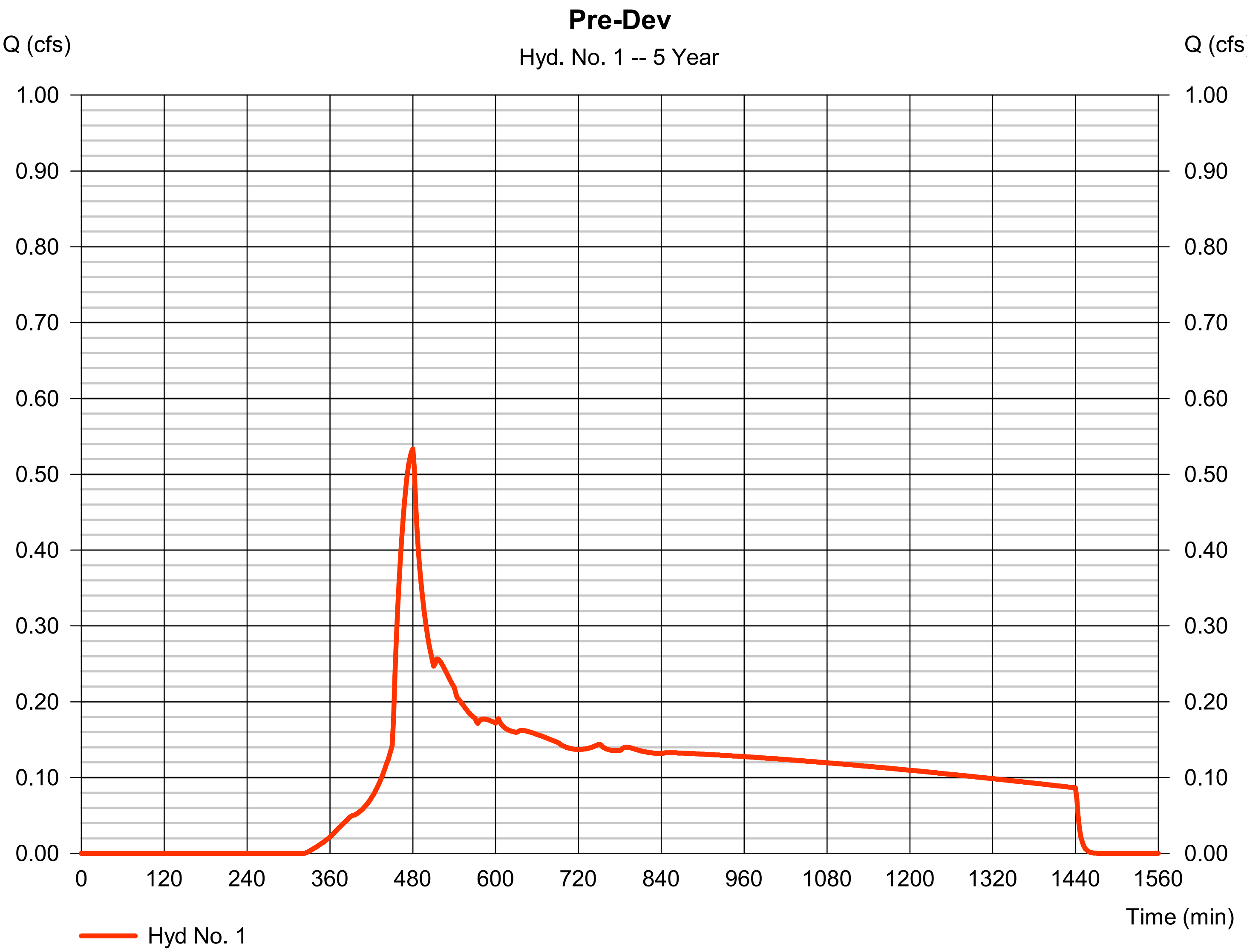
Hydrograph Report

Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2022 — Monday, 04 / 17 / 2023

Hyd. No. 1

Pre-Dev

Hydrograph type	= SBUH Runoff	Peak discharge	= 0.533 cfs
Storm frequency	= 5 yrs	Time to peak	= 480 min
Time interval	= 2 min	Hyd. volume	= 9,025 cuft
Drainage area	= 1.970 ac	Curve number	= 79
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 5.00 min
Total precip.	= 3.10 in	Distribution	= Type IA
Storm duration	= 24 hrs	Shape factor	= n/a



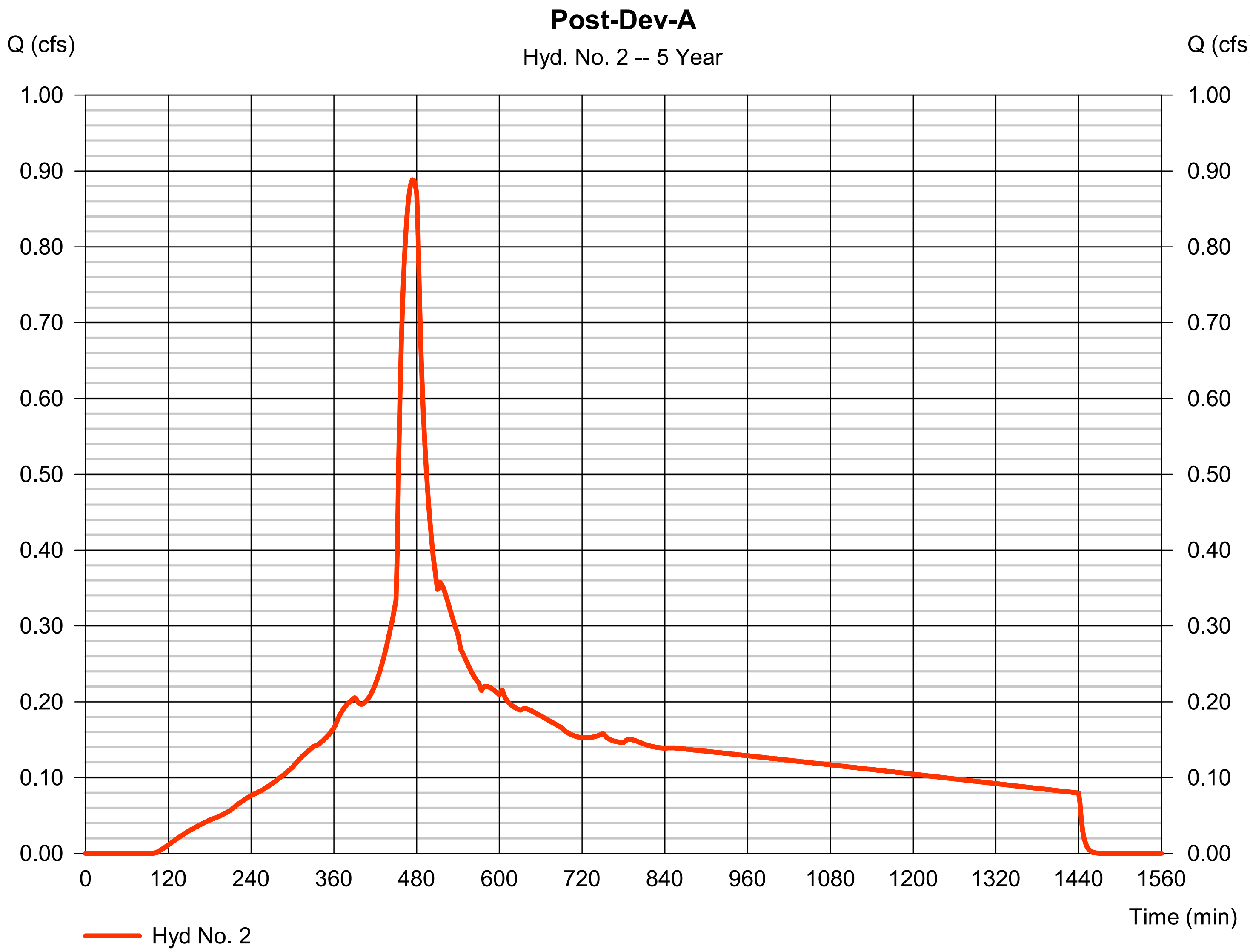
Hydrograph Report

Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2022 — Monday, 04 / 17 / 2023

Hyd. No. 2

Post-Dev-A

Hydrograph type	= SBUH Runoff	Peak discharge	= 0.888 cfs
Storm frequency	= 5 yrs	Time to peak	= 474 min
Time interval	= 2 min	Hyd. volume	= 12,456 cuft
Drainage area	= 1.380 ac	Curve number	= 94.4
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 5.00 min
Total precip.	= 3.10 in	Distribution	= Type IA
Storm duration	= 24 hrs	Shape factor	= n/a



Hydrograph Report

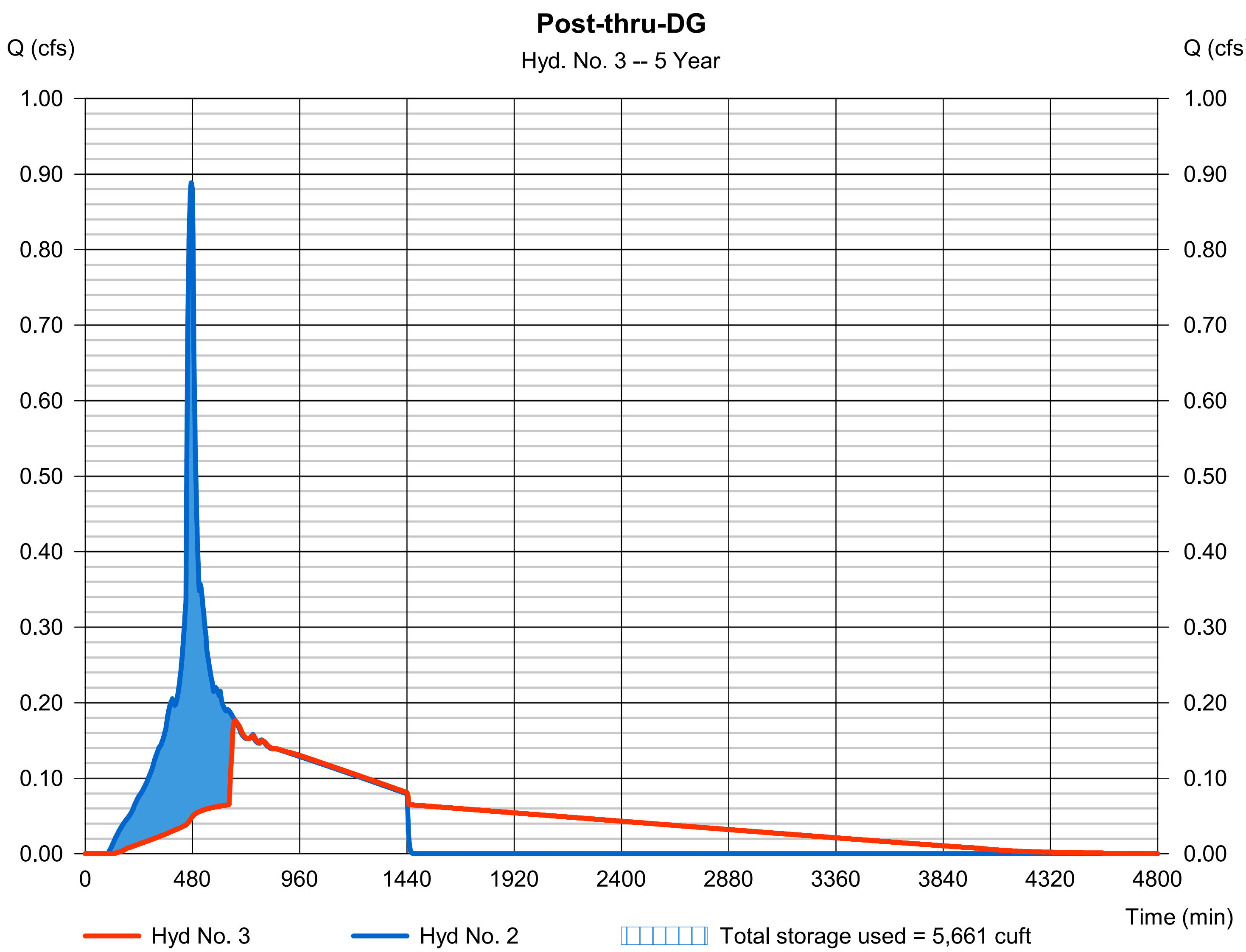
Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2022 Monday, 04 / 17 / 2023

Hyd. No. 3

Post-thru-DG

Hydrograph type	= Reservoir	Peak discharge	= 0.176 cfs
Storm frequency	= 5 yrs	Time to peak	= 670 min
Time interval	= 2 min	Hyd. volume	= 12,437 cuft
Inflow hyd. No.	= 2 - Post-Dev-A	Max. Elevation	= 176.01 ft
Reservoir name	= DG	Max. Storage	= 5,661 cuft

Storage Indication method used.



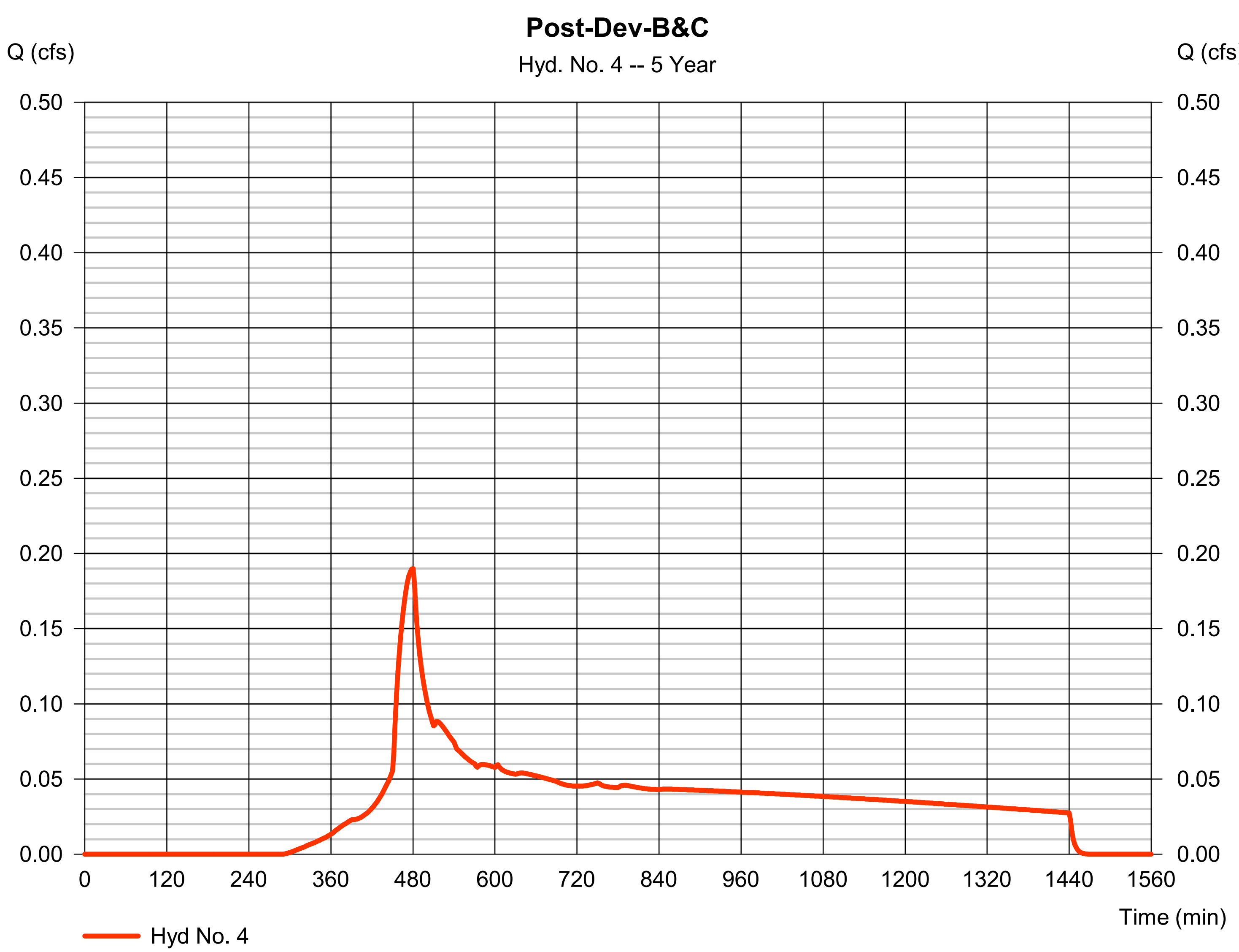
Hydrograph Report

Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2022 Monday, 04 / 17 / 2023

Hyd. No. 4

Post-Dev-B&C

Hydrograph type	= SBUH Runoff	Peak discharge	= 0.190 cfs
Storm frequency	= 5 yrs	Time to peak	= 480 min
Time interval	= 2 min	Hyd. volume	= 3,050 cuft
Drainage area	= 0.590 ac	Curve number	= 81.5
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 5.00 min
Total precip.	= 3.10 in	Distribution	= Type IA
Storm duration	= 24 hrs	Shape factor	= n/a



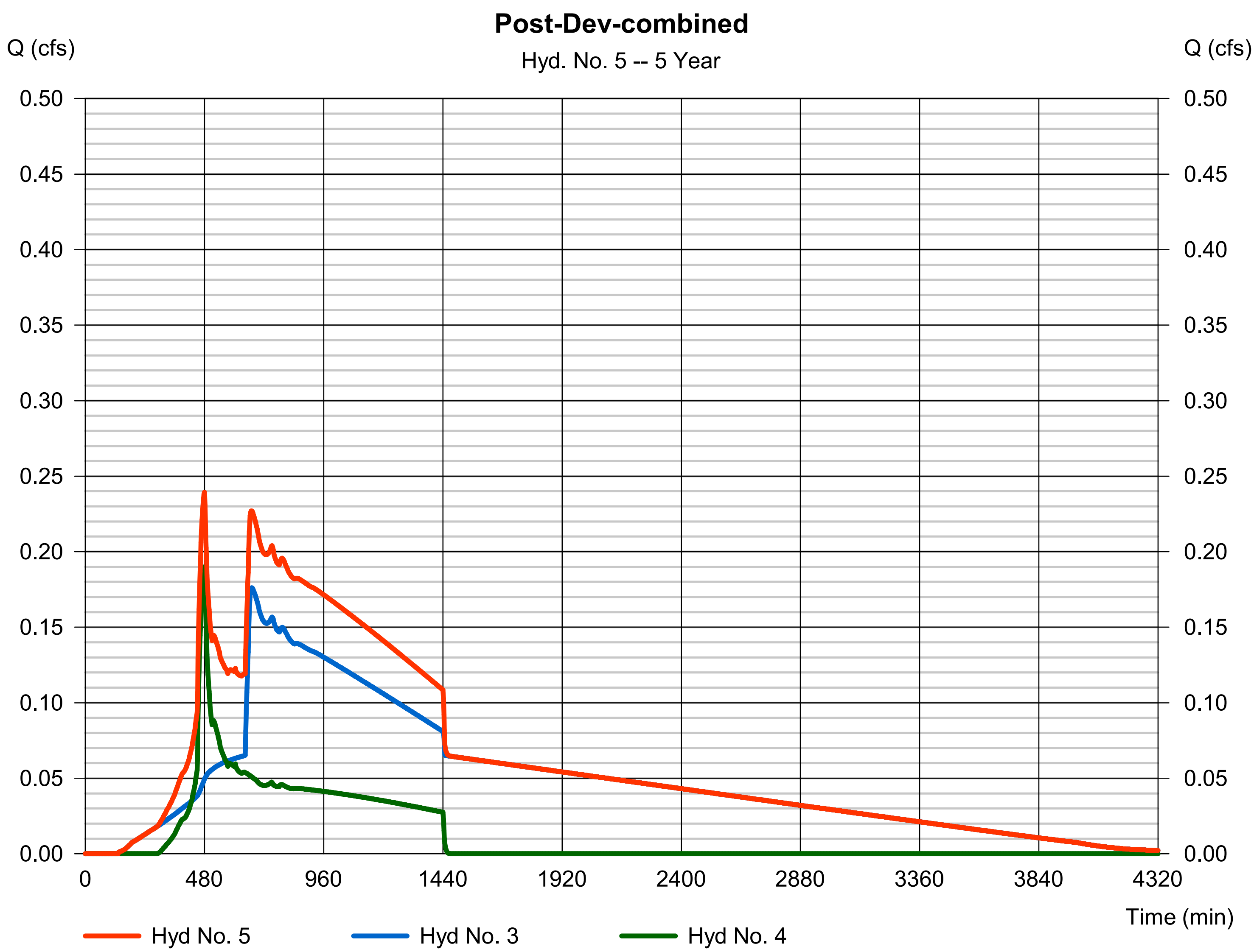
Hydrograph Report

Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2022 Monday, 04 / 17 / 2023

Hyd. No. 5

Post-Dev-combined

Hydrograph type	= Combine	Peak discharge	= 0.239 cfs
Storm frequency	= 5 yrs	Time to peak	= 480 min
Time interval	= 2 min	Hyd. volume	= 15,488 cuft
Inflow hyds.	= 3, 4	Contrib. drain. area	= 0.590 ac



Hydrograph Summary Report

Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2022

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	SBUH Runoff	0.670	2	480	10,921	------	------	------	Pre-Dev
2	SBUH Runoff	1.010	2	474	14,166	------	------	------	Post-Dev-A
3	Reservoir	0.247	2	584	14,147	2	176.02	5,681	Post-thru-DG
4	SBUH Runoff	0.234	2	480	3,651	------	------	------	Post-Dev-B&C
5	Combine	0.318	2	584	17,797	3, 4	------	------	Post-Dev-combined

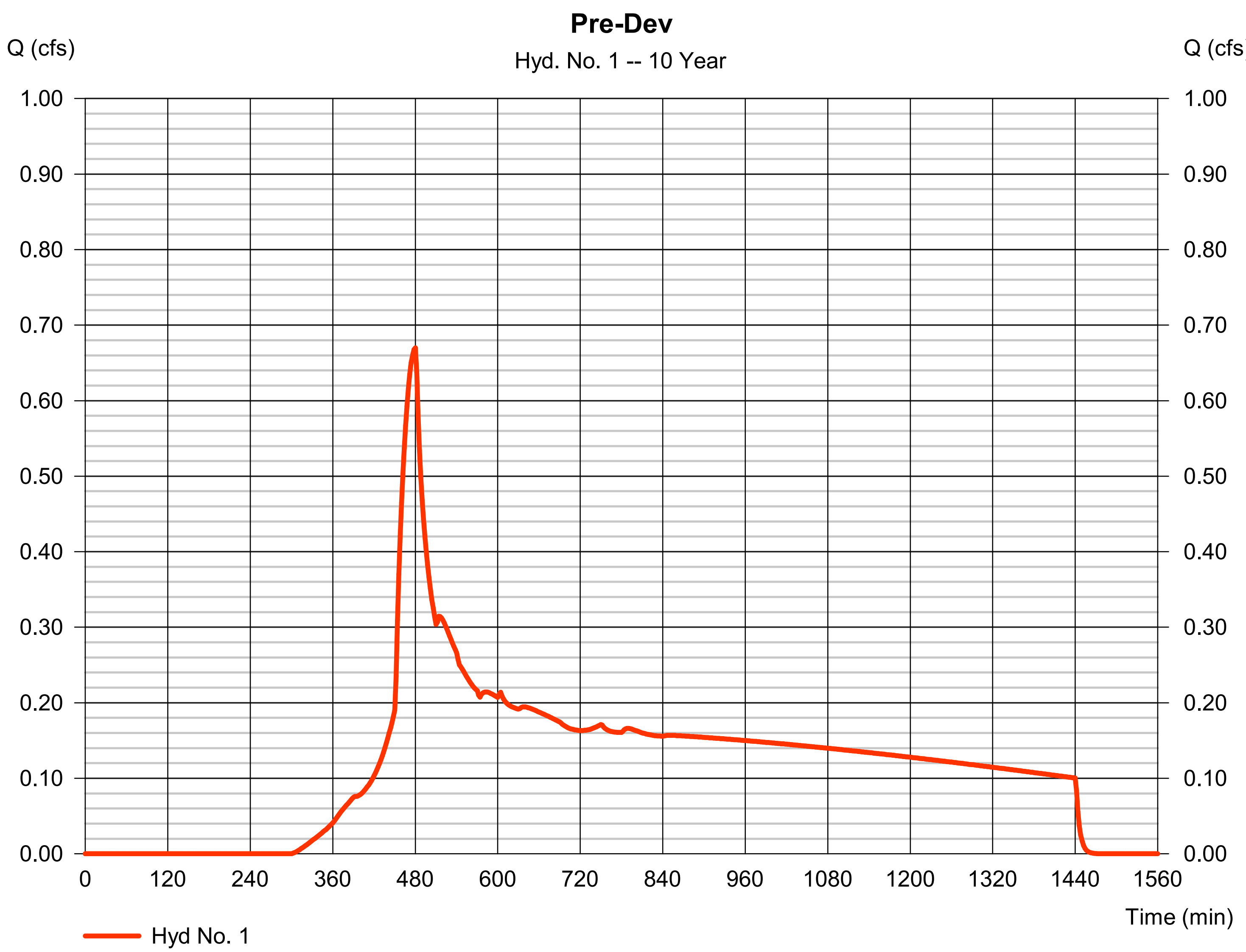
Hydrograph Report

Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2022 — Monday, 04 / 17 / 2023

Hyd. No. 1

Pre-Dev

Hydrograph type	= SBUH Runoff	Peak discharge	= 0.670 cfs
Storm frequency	= 10 yrs	Time to peak	= 480 min
Time interval	= 2 min	Hyd. volume	= 10,921 cuft
Drainage area	= 1.970 ac	Curve number	= 79
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	= n/a



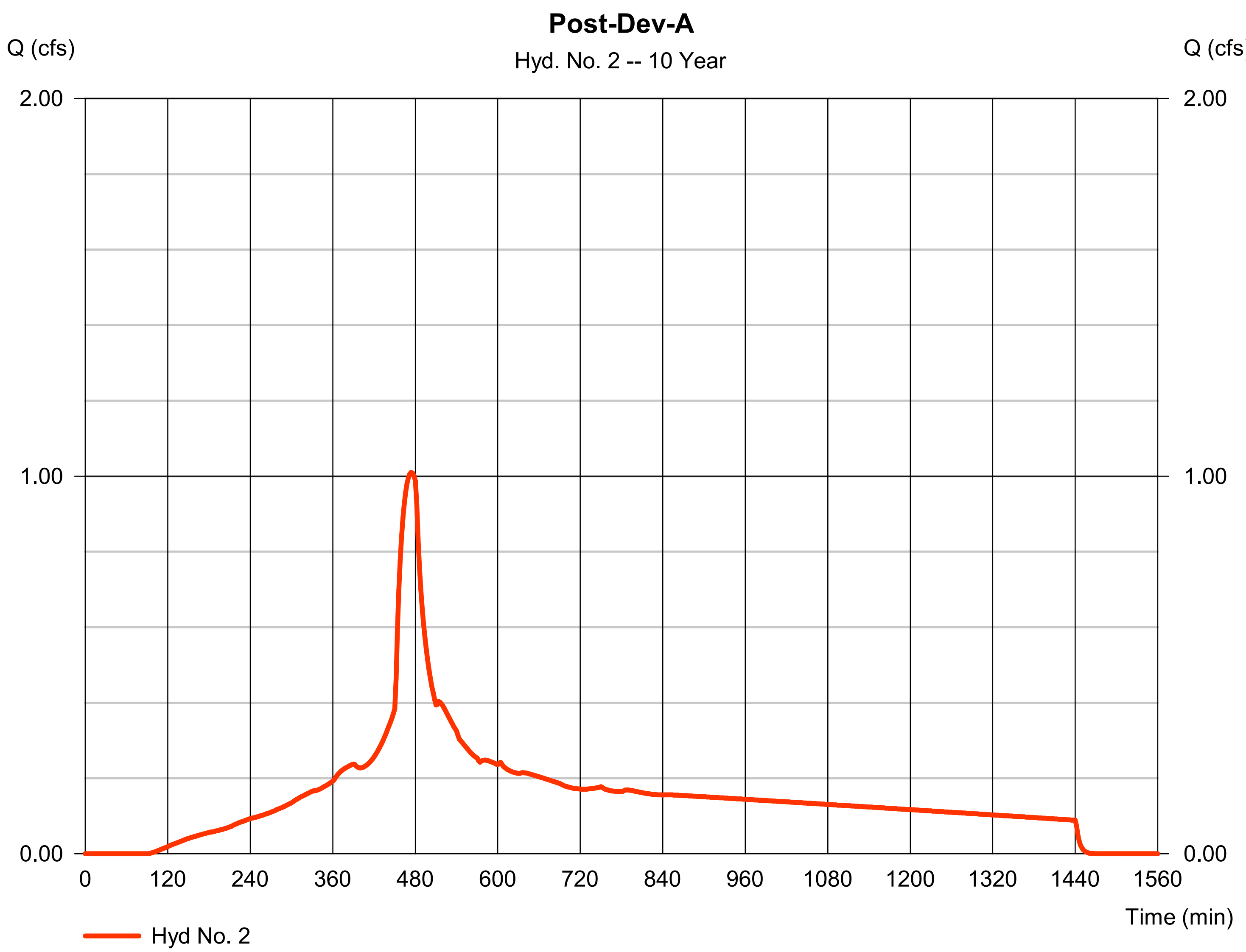
Hydrograph Report

Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2022 Monday, 04 / 17 / 2023

Hyd. No. 2

Post-Dev-A

Hydrograph type	= SBUH Runoff	Peak discharge	= 1.010 cfs
Storm frequency	= 10 yrs	Time to peak	= 474 min
Time interval	= 2 min	Hyd. volume	= 14,166 cuft
Drainage area	= 1.380 ac	Curve number	= 94.4
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	= n/a



Hydrograph Report

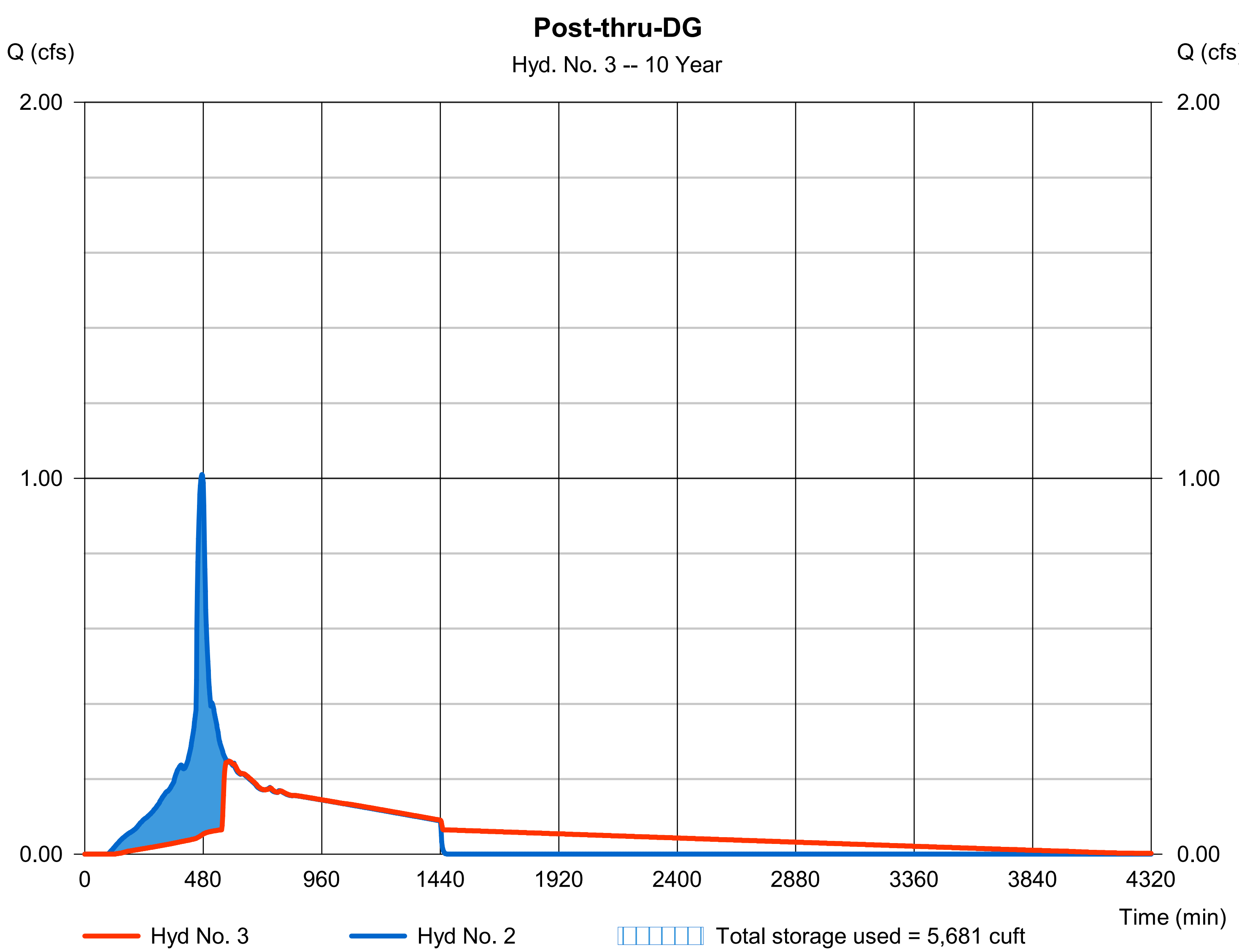
Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2022 Monday, 04 / 17 / 2023

Hyd. No. 3

Post-thru-DG

Hydrograph type	= Reservoir	Peak discharge	= 0.247 cfs
Storm frequency	= 10 yrs	Time to peak	= 584 min
Time interval	= 2 min	Hyd. volume	= 14,147 cuft
Inflow hyd. No.	= 2 - Post-Dev-A	Max. Elevation	= 176.02 ft
Reservoir name	= DG	Max. Storage	= 5,681 cuft

Storage Indication method used.



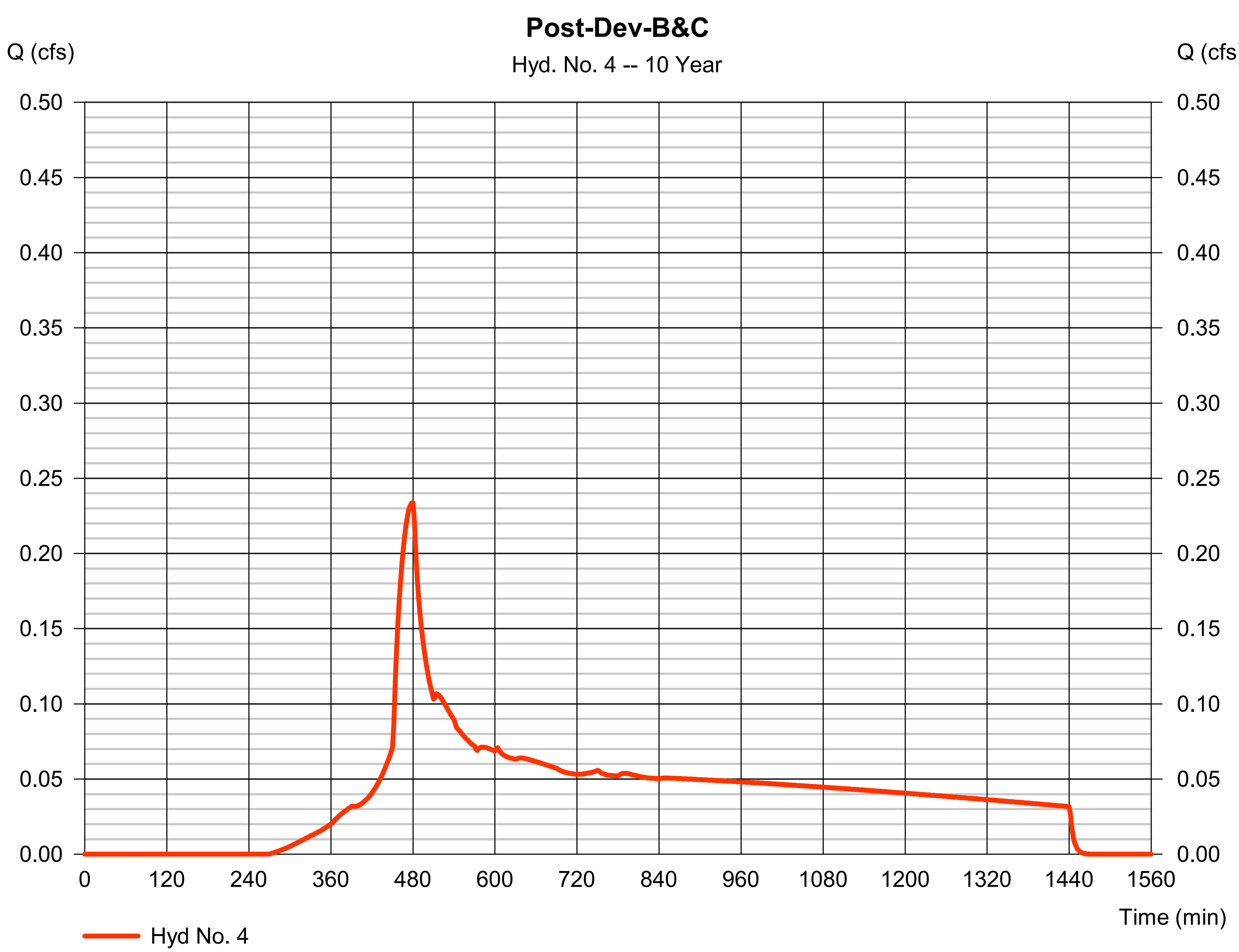
Hydrograph Report

Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2022 Monday, 04 / 17 / 2023

Hyd. No. 4

Post-Dev-B&C

Hydrograph type	= SBUH Runoff	Peak discharge	= 0.234 cfs
Storm frequency	= 10 yrs	Time to peak	= 480 min
Time interval	= 2 min	Hyd. volume	= 3,651 cuft
Drainage area	= 0.590 ac	Curve number	= 81.5
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	= n/a



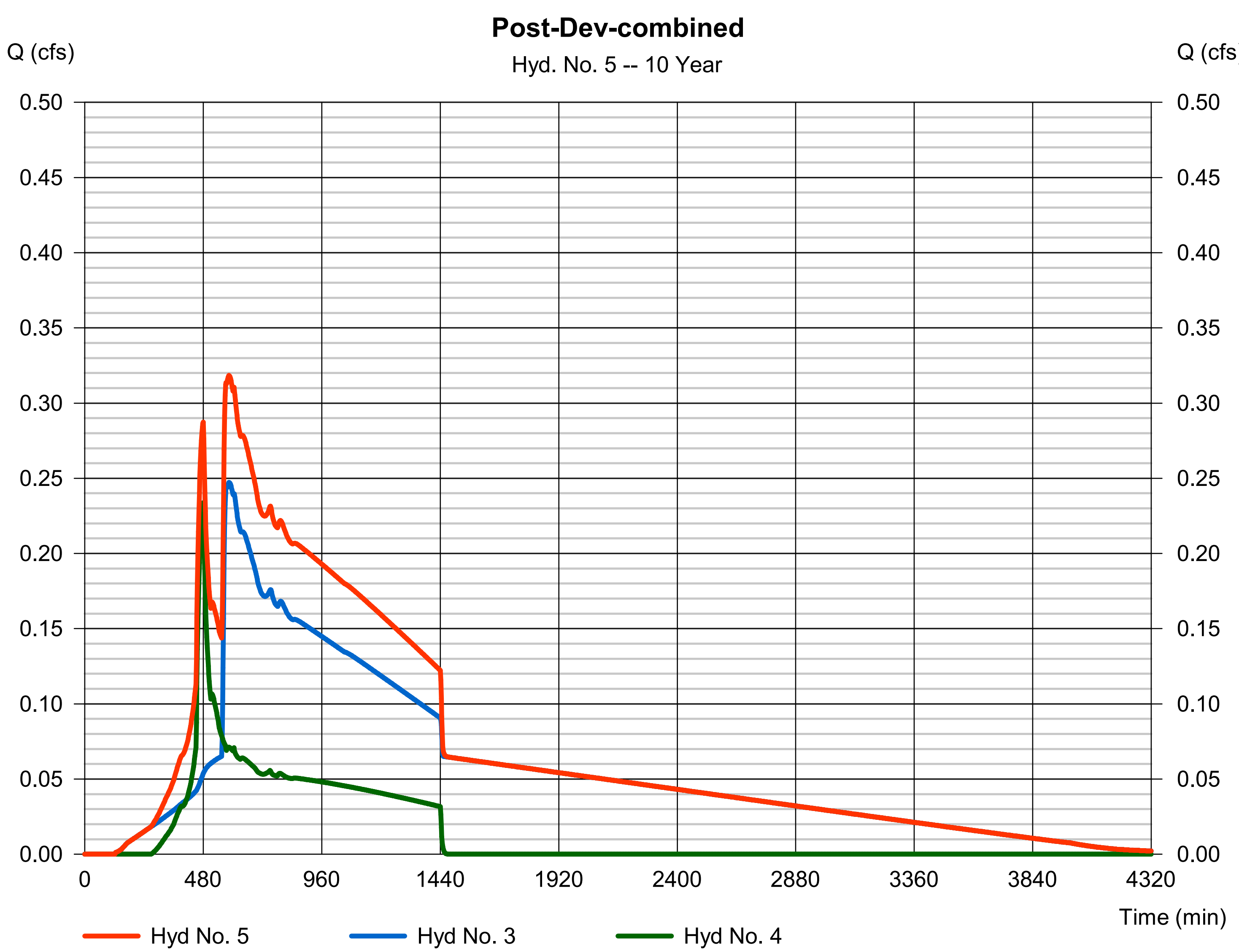
Hydrograph Report

Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2022 | Monday, 04 / 17 / 2023

Hyd. No. 5

Post-Dev-combined

Hydrograph type	= Combine	Peak discharge	= 0.318 cfs
Storm frequency	= 10 yrs	Time to peak	= 584 min
Time interval	= 2 min	Hyd. volume	= 17,797 cuft
Inflow hyds.	= 3, 4	Contrib. drain. area	= 0.590 ac



Hydrograph Summary Report

Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2022

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	SBUH Runoff	0.854	2	480	13,463	------	------	------	Pre-Dev
2	SBUH Runoff	1.165	2	474	16,374	------	------	------	Post-Dev-A
3	Reservoir	0.446	2	520	16,355	2	176.07	5,729	Post-thru-DG
4	SBUH Runoff	0.292	2	478	4,449	------	------	------	Post-Dev-B&C
5	Combine	0.574	2	520	20,804	3, 4	------	------	Post-Dev-combined

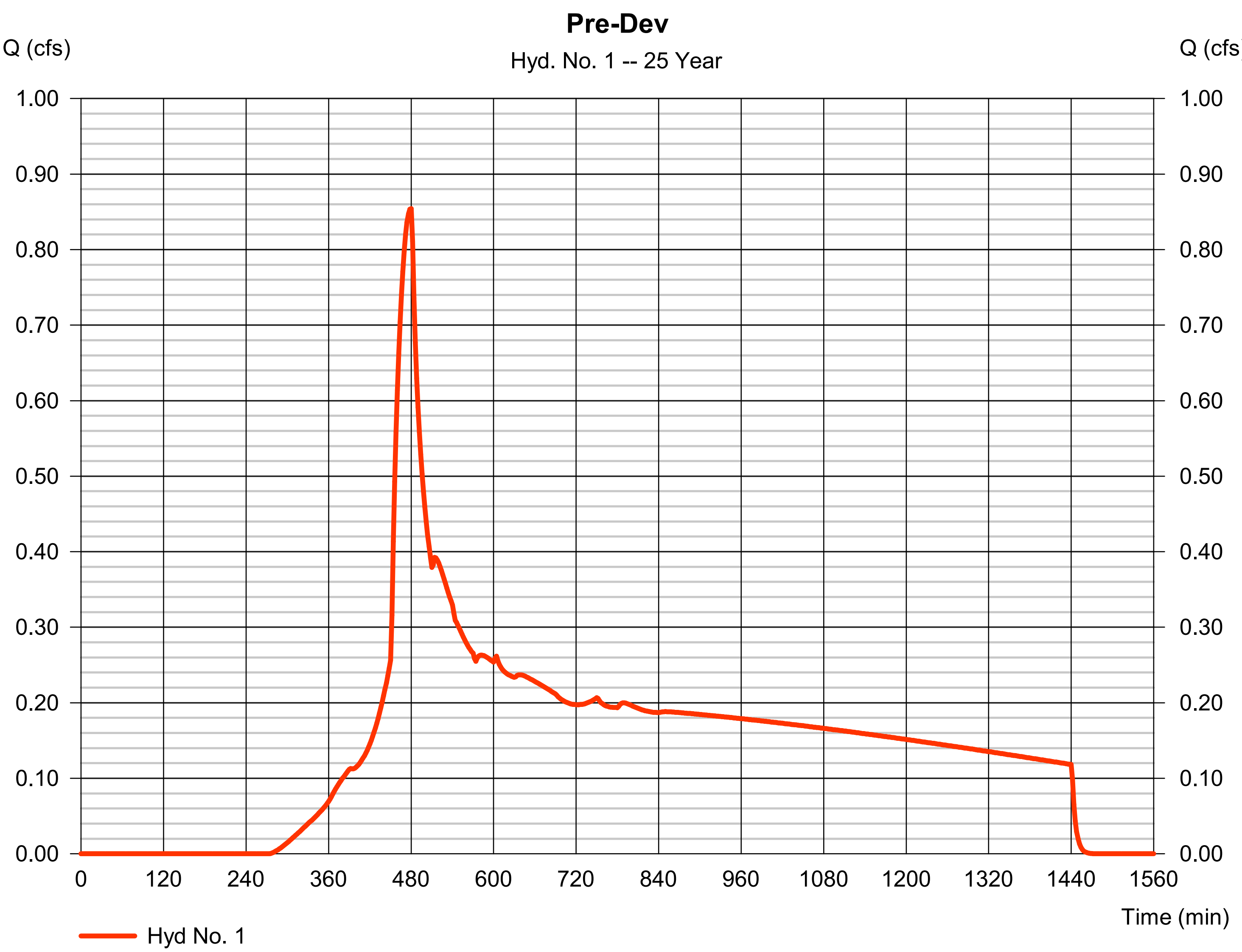
Hydrograph Report

Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2022 Monday, 04 / 17 / 2023

Hyd. No. 1

Pre-Dev

Hydrograph type	= SBUH Runoff	Peak discharge	= 0.854 cfs
Storm frequency	= 25 yrs	Time to peak	= 480 min
Time interval	= 2 min	Hyd. volume	= 13,463 cuft
Drainage area	= 1.970 ac	Curve number	= 79
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	= n/a



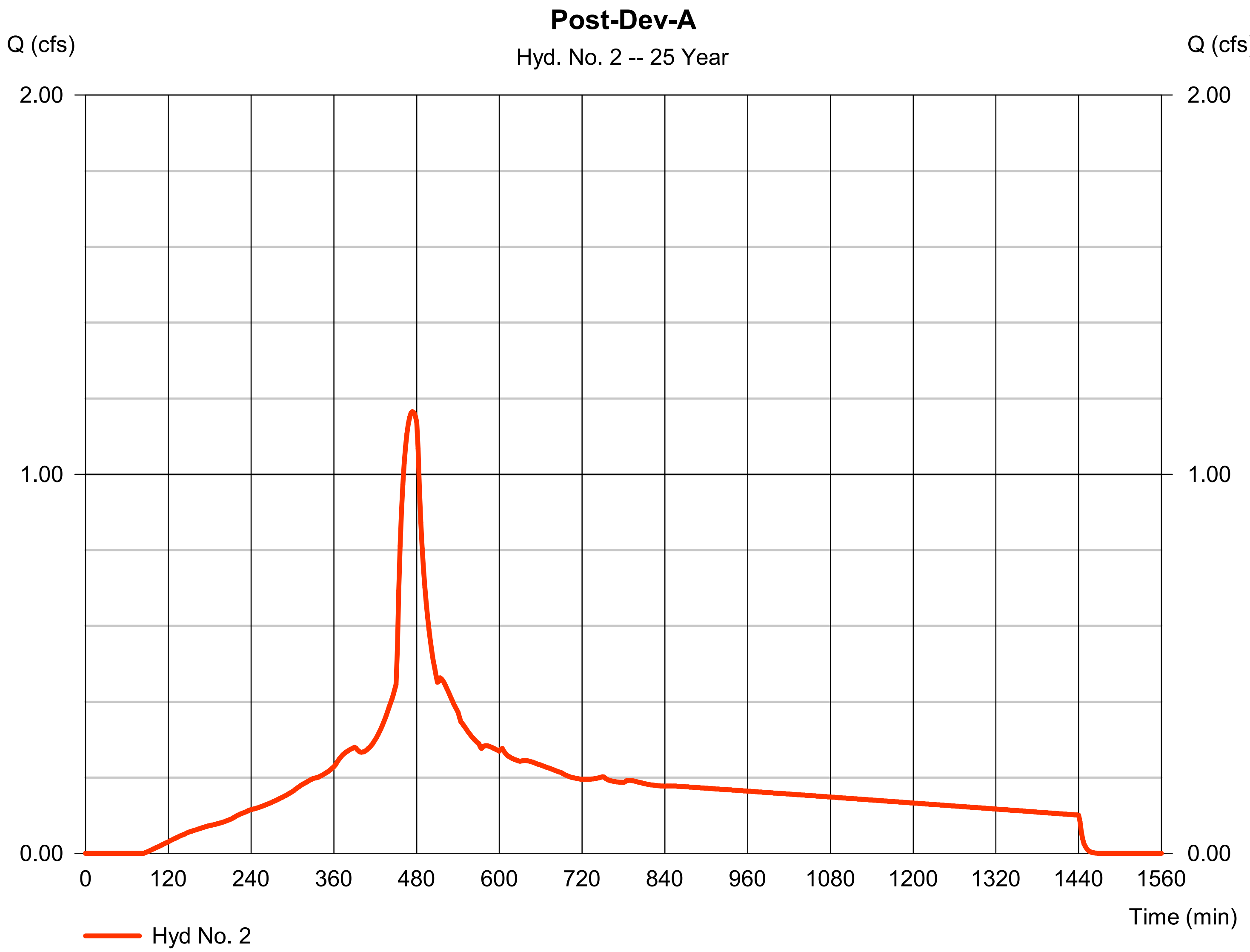
Hydrograph Report

Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2022 Monday, 04 / 17 / 2023

Hyd. No. 2

Post-Dev-A

Hydrograph type	= SBUH Runoff	Peak discharge	= 1.165 cfs
Storm frequency	= 25 yrs	Time to peak	= 474 min
Time interval	= 2 min	Hyd. volume	= 16,374 cuft
Drainage area	= 1.380 ac	Curve number	= 94.4
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	= n/a



Hydrograph Report

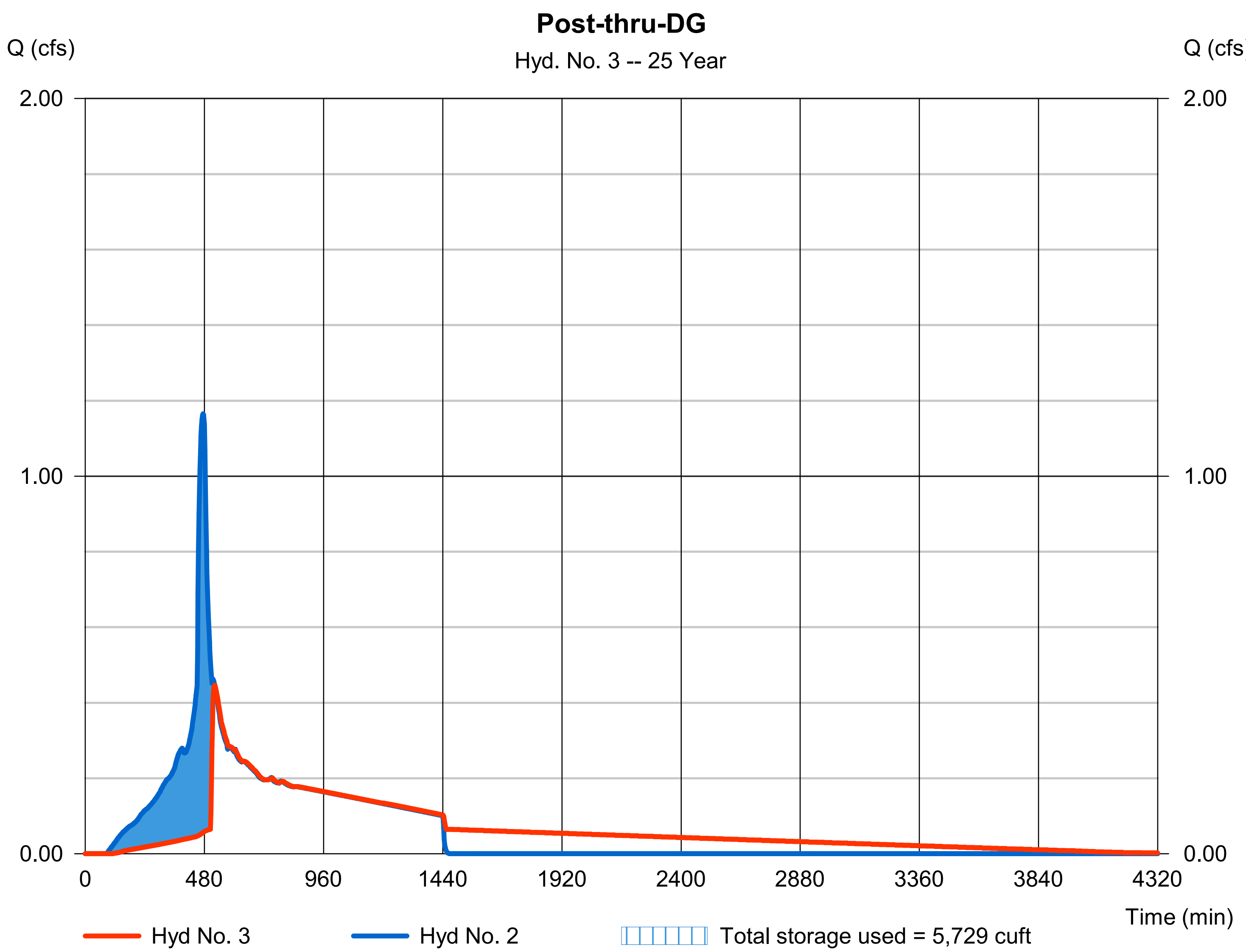
Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2022 Monday, 04 / 17 / 2023

Hyd. No. 3

Post-thru-DG

Hydrograph type	= Reservoir	Peak discharge	= 0.446 cfs
Storm frequency	= 25 yrs	Time to peak	= 520 min
Time interval	= 2 min	Hyd. volume	= 16,355 cuft
Inflow hyd. No.	= 2 - Post-Dev-A	Max. Elevation	= 176.07 ft
Reservoir name	= DG	Max. Storage	= 5,729 cuft

Storage Indication method used.



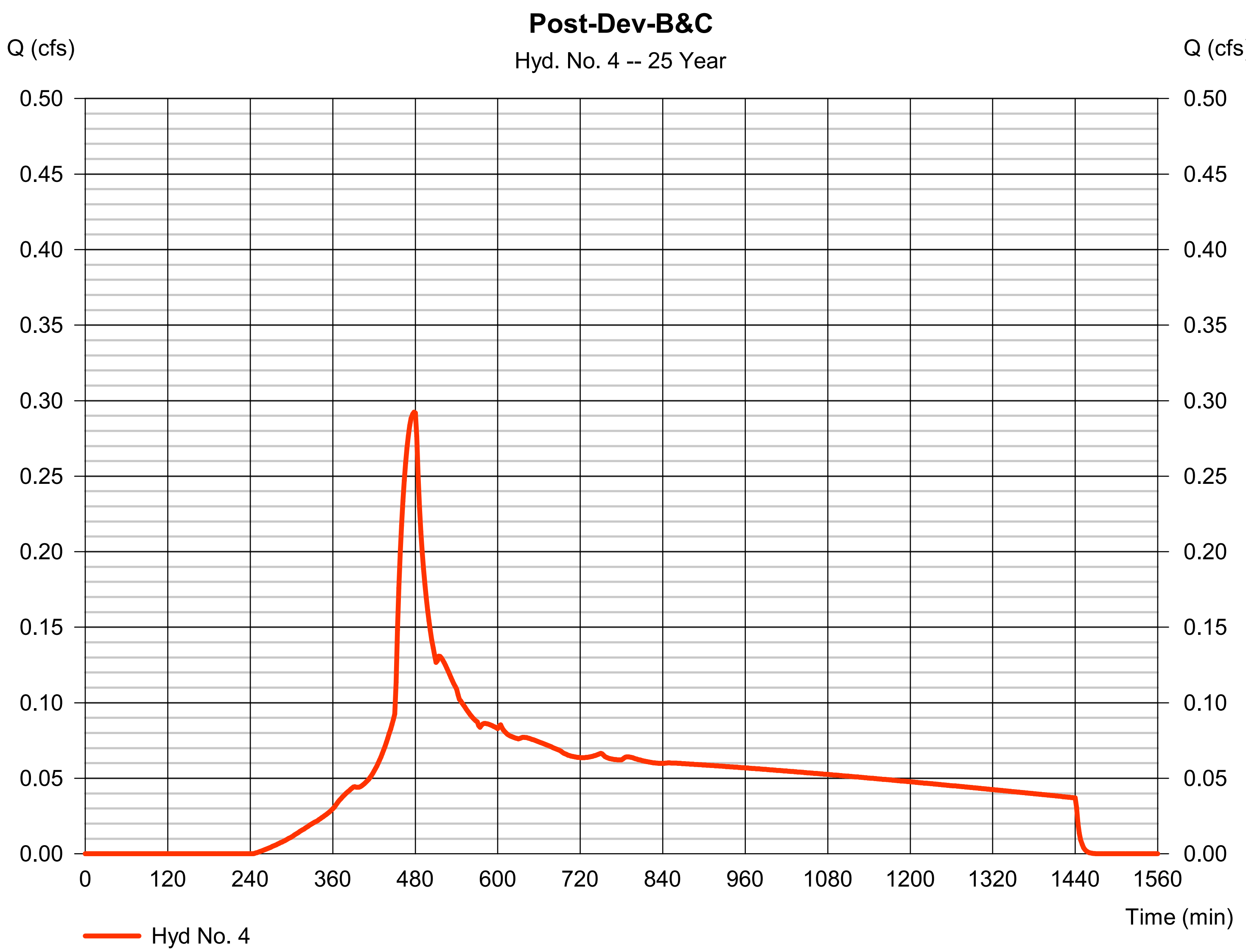
Hydrograph Report

Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2022 | Monday, 04 / 17 / 2023

Hyd. No. 4

Post-Dev-B&C

Hydrograph type	= SBUH Runoff	Peak discharge	= 0.292 cfs
Storm frequency	= 25 yrs	Time to peak	= 478 min
Time interval	= 2 min	Hyd. volume	= 4,449 cuft
Drainage area	= 0.590 ac	Curve number	= 81.5
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	= n/a



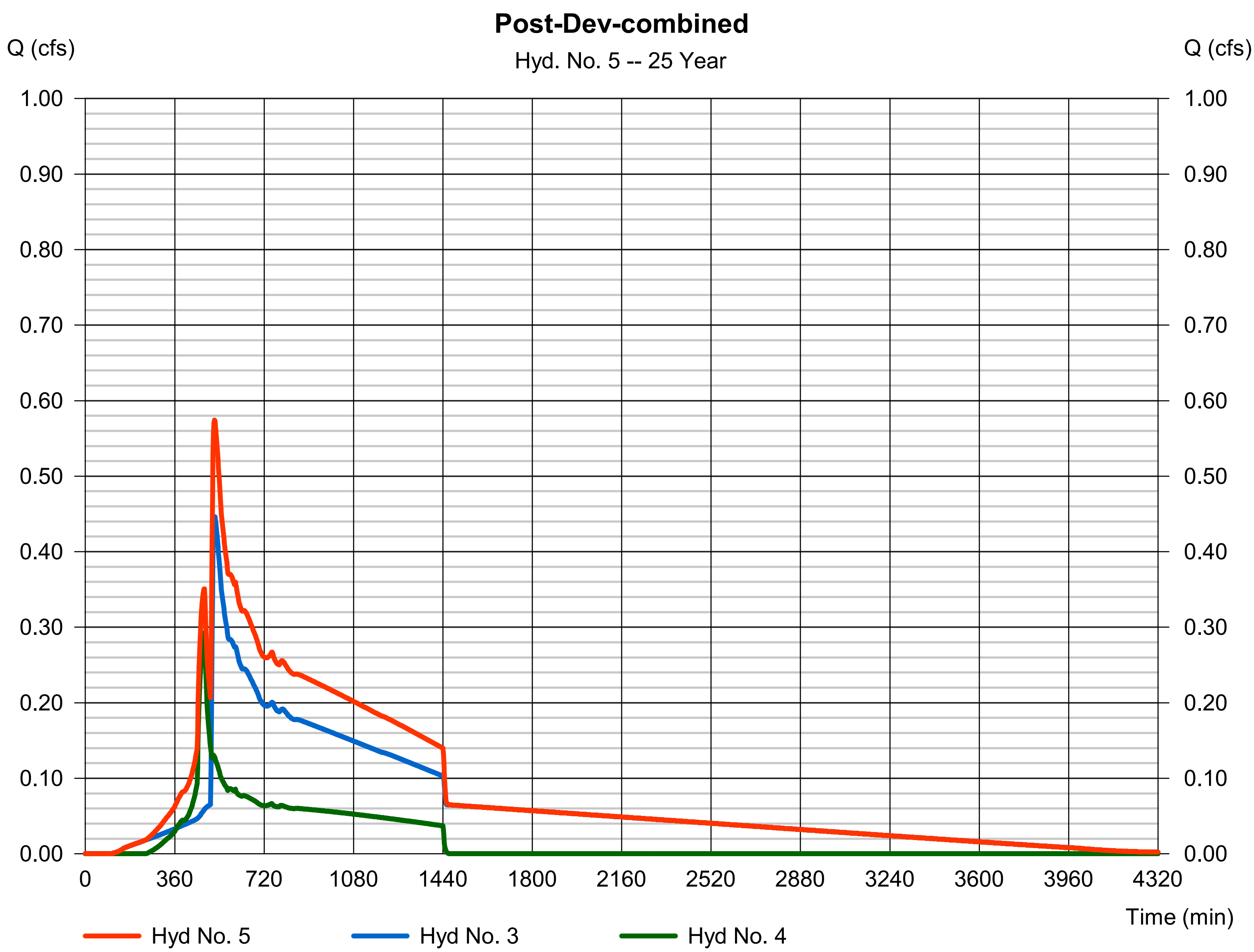
Hydrograph Report

Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2022 — Monday, 04 / 17 / 2023

Hyd. No. 5

Post-Dev-combined

Hydrograph type	= Combine	Peak discharge	= 0.574 cfs
Storm frequency	= 25 yrs	Time to peak	= 520 min
Time interval	= 2 min	Hyd. volume	= 20,804 cuft
Inflow hyds.	= 3, 4	Contrib. drain. area	= 0.590 ac



Hydrograph Summary Report

Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2022

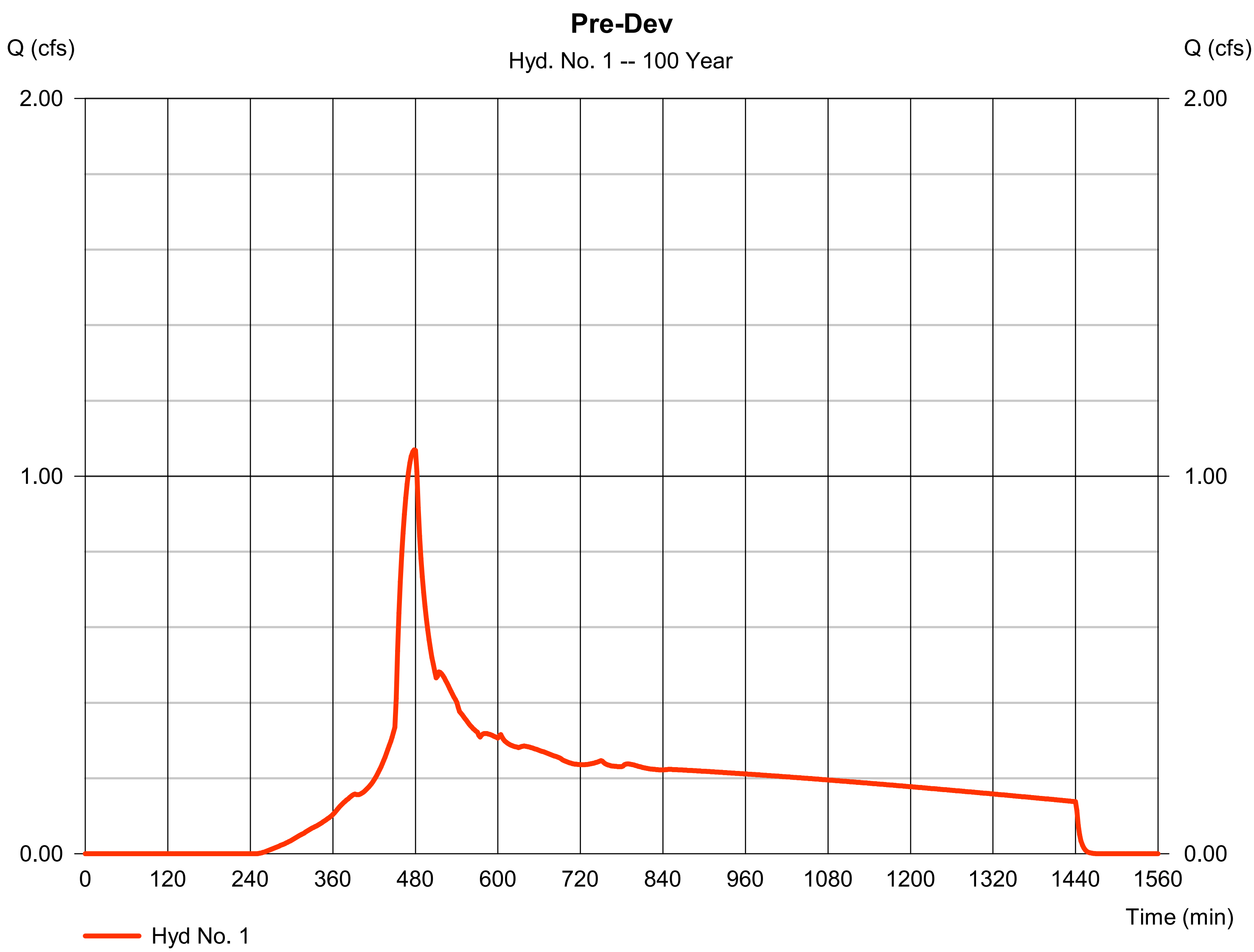
Hyd. No.	Hydrograph type (origin)	Peak flow (cfs)	Time interval (min)	Time to Peak (min)	Hyd. volume (cuft)	Inflow hyd(s)	Maximum elevation (ft)	Total strge used (cuft)	Hydrograph Description
1	SBUH Runoff	1.070	2	478	16,396	------	------	------	Pre-Dev
2	SBUH Runoff	1.337	2	474	18,837	------	------	------	Post-Dev-A
3	Reservoir	0.858	2	490	18,818	2	176.14	5,804	Post-thru-DG
4	SBUH Runoff	0.360	2	478	5,365	------	------	------	Post-Dev-B&C
5	Combine	1.102	2	490	24,183	3, 4	------	------	Post-Dev-combined

Hydrograph Report

Hyd. No. 1

Pre-Dev

Hydrograph type	= SBUH Runoff	Peak discharge	= 1.070 cfs
Storm frequency	= 100 yrs	Time to peak	= 478 min
Time interval	= 2 min	Hyd. volume	= 16,396 cuft
Drainage area	= 1.970 ac	Curve number	= 79
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 5.00 min
Total precip.	= 4.40 in	Distribution	= Type IA
Storm duration	= 24 hrs	Shape factor	= n/a



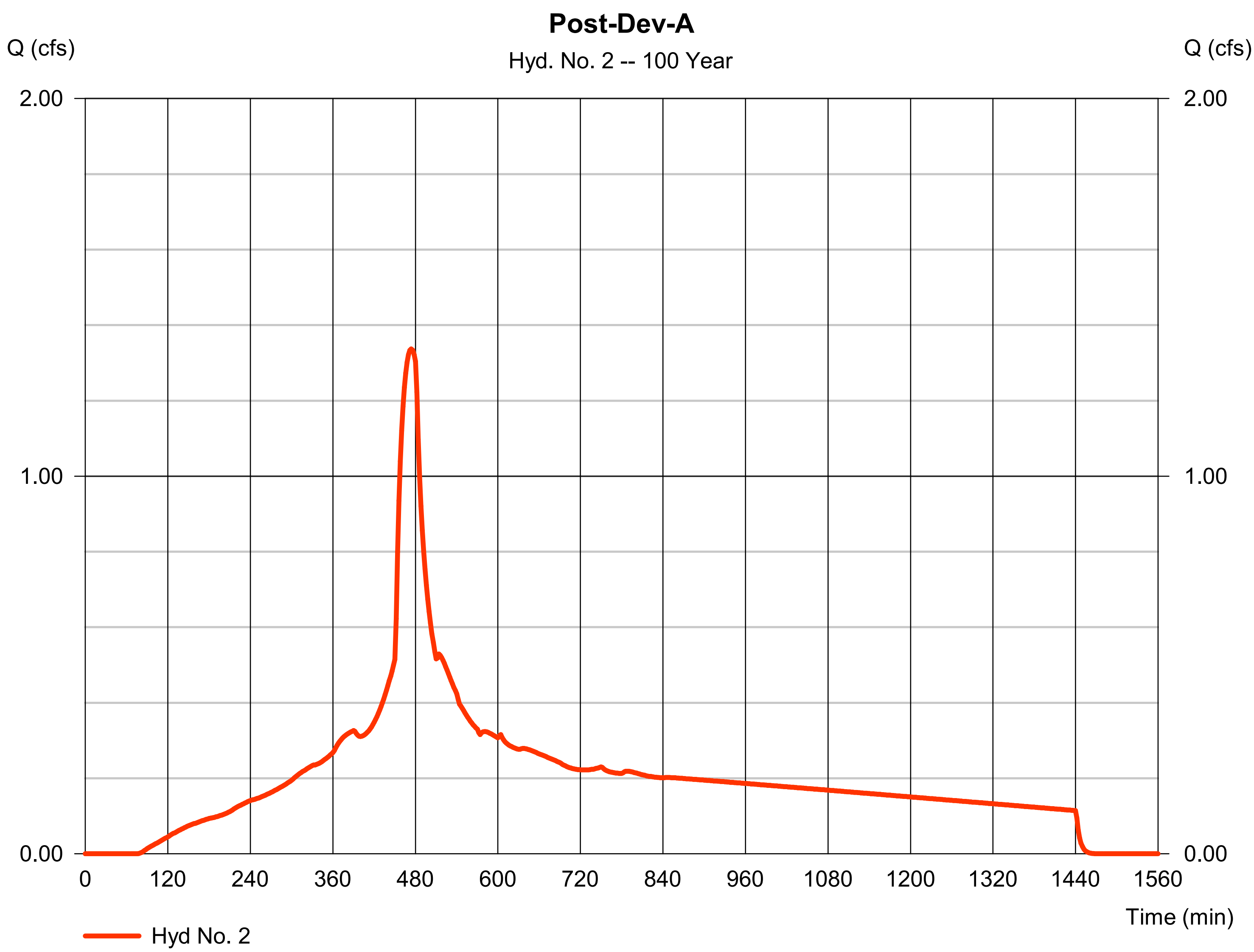
Hydrograph Report

Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2022 — Monday, 04 / 17 / 2023

Hyd. No. 2

Post-Dev-A

Hydrograph type	= SBUH Runoff	Peak discharge	= 1.337 cfs
Storm frequency	= 100 yrs	Time to peak	= 474 min
Time interval	= 2 min	Hyd. volume	= 18,837 cuft
Drainage area	= 1.380 ac	Curve number	= 94.4
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 5.00 min
Total precip.	= 4.40 in	Distribution	= Type IA
Storm duration	= 24 hrs	Shape factor	= n/a



Hydrograph Report

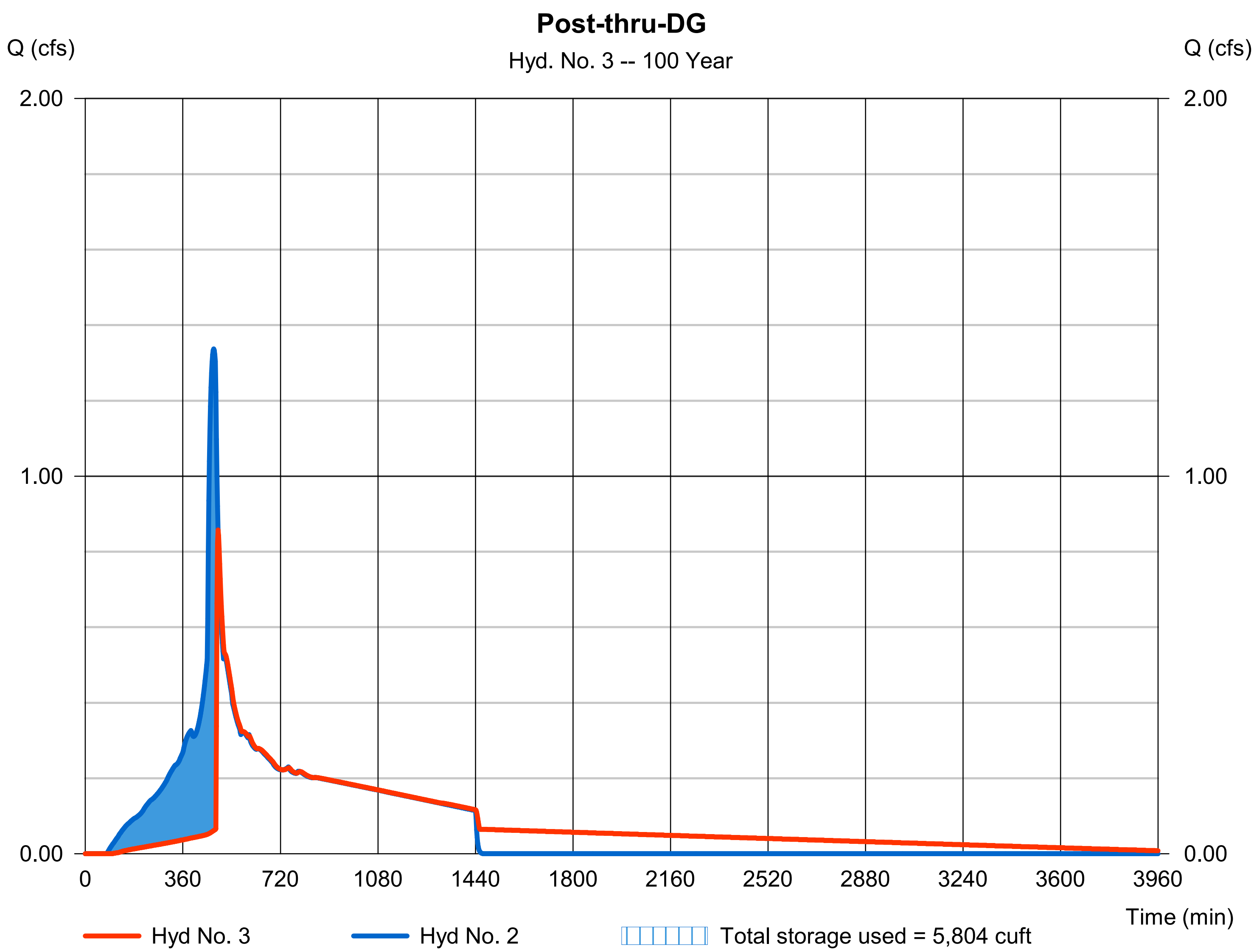
Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2022 Monday, 04 / 17 / 2023

Hyd. No. 3

Post-thru-DG

Hydrograph type	= Reservoir	Peak discharge	= 0.858 cfs
Storm frequency	= 100 yrs	Time to peak	= 490 min
Time interval	= 2 min	Hyd. volume	= 18,818 cuft
Inflow hyd. No.	= 2 - Post-Dev-A	Max. Elevation	= 176.14 ft
Reservoir name	= DG	Max. Storage	= 5,804 cuft

Storage Indication method used.



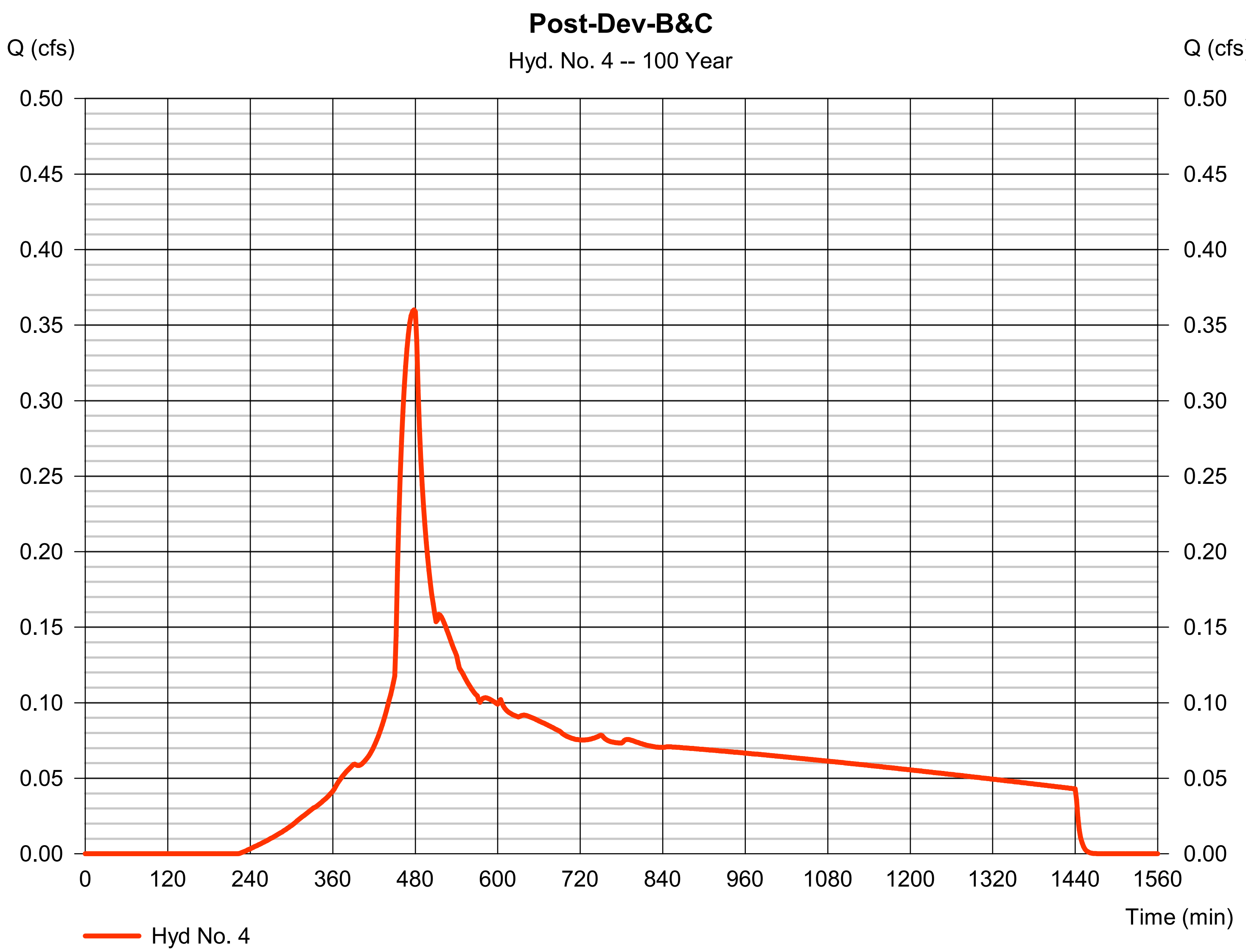
Hydrograph Report

Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2022 Monday, 04 / 17 / 2023

Hyd. No. 4

Post-Dev-B&C

Hydrograph type	= SBUH Runoff	Peak discharge	= 0.360 cfs
Storm frequency	= 100 yrs	Time to peak	= 478 min
Time interval	= 2 min	Hyd. volume	= 5,365 cuft
Drainage area	= 0.590 ac	Curve number	= 81.5
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 5.00 min
Total precip.	= 4.40 in	Distribution	= Type IA
Storm duration	= 24 hrs	Shape factor	= n/a



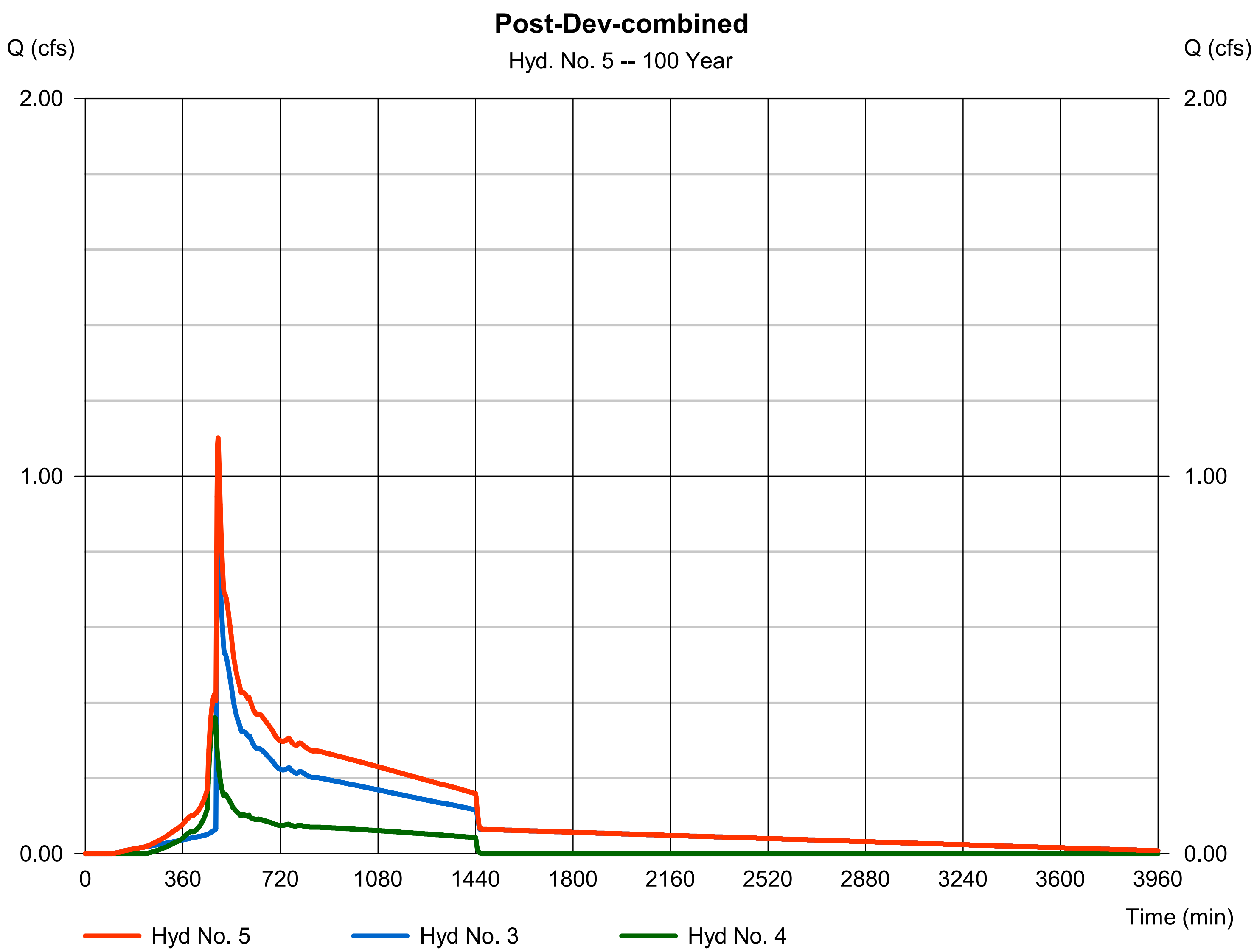
Hydrograph Report

Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2022 Monday, 04 / 17 / 2023

Hyd. No. 5

Post-Dev-combined

Hydrograph type	= Combine	Peak discharge	= 1.102 cfs
Storm frequency	= 100 yrs	Time to peak	= 490 min
Time interval	= 2 min	Hyd. volume	= 24,183 cuft
Inflow hyds.	= 3, 4	Contrib. drain. area	= 0.590 ac



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Appendix D:

D1 Landuse Plans

D2 SW Westgate Drive – As-built Storm Plans

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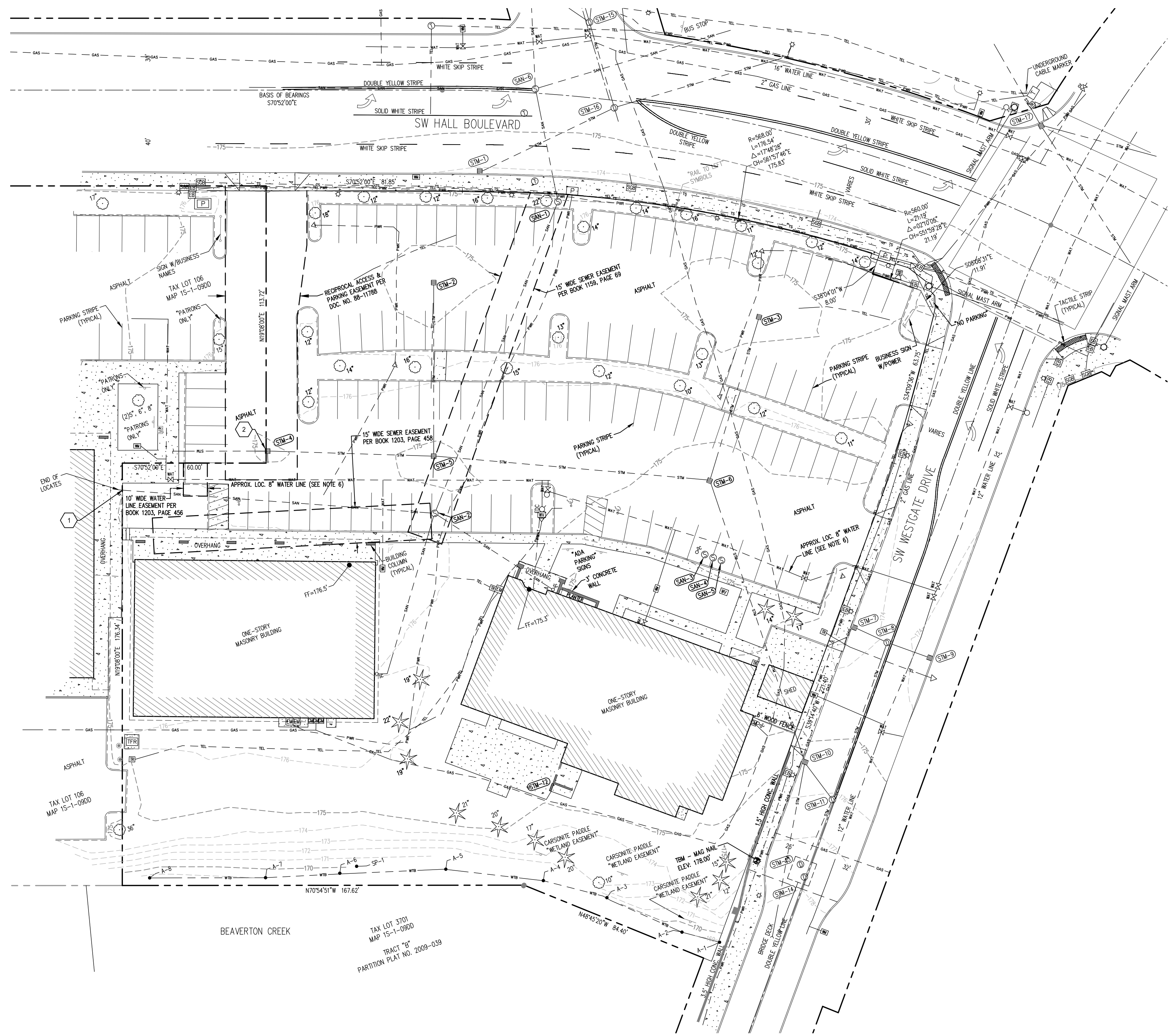


NOT FOR CONSTRUCTION

(X) UTILITY INVESTIGATION KEY NOTES

NOTE	DESCRIPTION	DETAIL REF.
1	PER SCOPING INVESTION BY PACIFIC INTERTECH ON 1/23/2023, ADJACENT PROPERTY SANITARY DOES NOT CONNECT TO ONSITE PUBLIC SANITARY MAIN TO BE REMOVED.	
2	XXXXXXX (RESPONSE PENDING NEW SCOPING WORK UNDERWAY)	

STORM & SANITARY SEWER INFORMATION

STM-1 CATCH BASIN
RIM 173.79'
15" OUT (E) 170.2'

STM-2 CATCH BASIN
RIM 174.49'
TRAP (S)

STM-3 CATCH BASIN
RIM 174.34'
TRAP (S)

STM-4 CATCH BASIN
RIM 174.82'
6" IN (S) 173.1'
6" IN (W) 172.8'
TRAP (E)

STM-5 CATCH BASIN
RIM 174.70'
8" IN (W) 172.4'
8" IN (S) 172.4'
TRAP (N) & (E)

STM-6 CATCH BASIN
RIM 174.30'
18" IN (NE) 171.5'
21" IN (NW) 171.6'
TRAP (S)

STM-7 CATCH BASIN
RIM 173.53'
10" OUT (SE) 170.9'

STM-8 MANHOLE
RIM 174.08'
10" IN (NW) 170.8'
10" IN (SE) 170.7'
12" OUT (SW) 170.6'

STM-9 CATCH BASIN
RIM 173.51'
12" OUT (NW) 170.8'

STM-10 CATCH BASIN
RIM 175.21'
24" IN (N) 170.6'
24" OUT (SE) 170.5'
24" OUT (SW) 170.5'

STM-11 MANHOLE
RIM 176.16'
12" IN (NE) 170.1'
15" IN NW 170.0'
12" OUT (SW) 169.8'

STM-12 SLOT DRAIN
RIM 175.13'

STM-13 VAULT
RIM 177.65'
12" IN (NE) 169.5'

STM-14 VAULT
RIM 177.89
NOT OUTLET OBSERVED

STM-15 MANHOLE
RIM = 175.77'
10" I.E. IN (NE) = 171.7'
10" I.E. IN (NW) = 171.4'
18" I.E. IN (N) = 170.6'
18" I.E. OUT (S) = 168.8'

STM-16 MANHOLE
RIM = 175.45'
15" I.E. IN (W) = 168.0'
18" I.E. IN (S) = 167.3' (POSSIBLE STUB)
18" IN (N) PARTIALLY
BLOCKED WITH CONCRETE = 167.3'
18" I.E. OUT (E) = 167.1'

STM-17 CATCH BASIN
RIM = 175.30'
15" I.E. IN (SE) = 168.8'
36" I.E. IN (N) = 166.6'
30" I.E. OUT (S) = 166.6'

SAN-1 MANHOLE
RIM 175.22'
8" IN (SW) 166.5'
8" OUT (N) 166.4'

SAN-2 MANHOLE
RIM 175.35'
4" IN (SE) 168.7'
4" IN (SW) 168.6'
8" IN (NW) 168.5'
8" OUT (NE) 168.3'

SAN-3 GREASE TRAP
RIM 174.64'
BOLTED LID

SAN-4 GREASE TRAP
RIM 174.63'
BOLTED LID

SAN-5 GREASE TRAP
RIM 174.58'
BOLTED LID

SAN-6 MANHOLE
RIM = 175.60'
8" IN (N & S)
12" IN (W)
12" OUT (E)
NOT MEASURED DUE TO TRAFFIC CONFLICTS

LEGEND

DECIDUOUS TREE W/SIZE NOTED
PINE TREE W/SIZE NOTED
FIRE HYDRANT
FIRE DEPARTMENT CONNECTION
WATER METER
WATER VALVE
WATER VAULT
SANITARY SEWER CLEAN OUT
SANITARY SEWER MANHOLE
STORM SEWER CLEAN OUT
STORM SEWER CATCH BASIN
STORM SEWER MANHOLE
MAILBOX
BOLLARD

GAS METER
POWER VAULT
ELECTRICAL METER
POWER JUNCTION BOX
POWER TRANSFORMER
STREET LIGHT
PRIVATE LIGHT
TELEPHONE/TELEVISION VAULT
TELEPHONE/TELEVISION JUNCTION BOX
TELEPHONE/TELEVISION RISER
TELEPHONE MANHOLE
TRAFFIC SIGNAL POLE
SIGNAL JUNCTION BOX
SIGN
FOUND SURVEY MONUMENT

RIGHT-OF-WAY LINE
PROPERTY LINE
CENTERLINE
CURB
EXTRUDED CURB
EDGE OF PAVEMENT
EASEMENT
FENCE LINE
POWER LINE
TELEPHONE LINE
GAS LINE
STORM SEWER LINE
SANITARY SEWER LINE
WATER LINE
TRAFFIC SIGNAL WIRE
WETLAND BOUNDARY

NOTES

1) THE FIELD SURVEY FOR THIS MAP WAS COMPLETED ON AUGUST 12, 2022

2) ELEVATIONS ARE BASED ON WASHINGTON COUNTY BENCHMARK NO. 191. IT IS A BRASS DISK SET IN THE SIDEWALK AT THE SOUTHEAST CORNER OF THE BRIDGE ON SW CEDAR HILLS BOULEVARD CROSSING BEAVERTON CREEK. THE BENCHMARK ELEVATION IS 172.55 FEET ON THE NGVD 1929 VERTICAL DATUM. THE HORIZONTAL BASIS OF BEARINGS IS ALONG THE NORTHWESTERLY TANGENT CENTERLINE OF SW HALL BOULEVARD PER WASHINGTON COUNTY SURVEY NO. 21,149.

3) EASEMENTS ARE PER A PRELIMINARY TITLE REPORT PREPARED BY FIRST AMERICAN TITLE INSURANCE COMPANY WITH AN EFFECTIVE DATE OF JUNE 22, 2022 AND ORDER NO. 7000-3966747.

4) UNDERGROUND UTILITY MARKINGS ARE BASED ON LOCATE TICKET NO. 22198911, 22214873, 22214874 AND 22240279.

5) SEVERAL CARSONITE POSTS LABELED AS MARKING A WETLAND EASEMENT WERE OBSERVED ADJACENT TO THE CREEK WHERE SHOWN. OTHERS MAY EXIST BUT THERE IS A THICK BRUSH IN PLACES. THERE IS NO EASEMENT FOR WETLAND PURPOSES DISCLOSED IN THE TITLE REPORT.

6) A CITY OF BEAVERTON WATER LINE CROSSES THROUGH THE PROPERTY APPROXIMATELY WHERE SHOWN. THE UTILITY LOCATOR COULD NOT DETERMINE ITS EXACT LOCATION. THERE IS NOT AN EASEMENT FOR THIS USE DISCLOSED IN THE TITLE REPORT.

7) UTILITY LOCATE MARKINGS ON THE PROPERTY WERE INCOMPLETE AND ARE MAPPED TO THE EXTENTS AS THEY WERE DEPICTED ON THE GROUND.

8) THE SEWER LINES TO THE WEST AND SOUTHWEST OF MANHOLE "SAN-2" ARE BASED ON LOCATE MARKINGS BY OTHERS.

UTILITY STATEMENT

THE UNDERGROUND UTILITIES SHOWN HAVE BEEN LOCATED FROM FIELD SURVEY INFORMATION AND EXISTING DRAWINGS. THE SURVEYOR MAKES NO GUARANTEE THAT THE UNDERGROUND UTILITIES SHOWN COMPRISE ALL SUCH UTILITIES IN THE AREA, EITHER IN SERVICE OR ABANDONED. THE SURVEYOR FURTHER DOES NOT WARRANT THAT THE UNDERGROUND UTILITIES SHOWN ARE IN THE EXACT LOCATION INDICATED ALTHOUGH HE DOES CERTIFY THAT THEY ARE LOCATED AS ACCURATELY AS POSSIBLE FROM INFORMATION AVAILABLE. THE SURVEYOR HAS NOT PHYSICALLY LOCATED THE UNDERGROUND UTILITIES.

PORTIONS OF LOTS 17 & 18, STEEL'S
ADDITION TO BEAVERTON,
LOCATED IN THE SOUTHEAST
ONE-QUARTER OF SECTION 9,
TOWNSHIP 1 SOUTH, RANGE 1 WEST, W.M.,
CITY OF BEAVERTON,
WASHINGTON COUNTY, OREGON

NORTHWEST SURVEYING, INC.
1815 NW 169th PLACE, SUITE 2090
BEAVERTON, OR 97006
PH:(503) 848-2127 FAX:(503) 848-2179
EMAIL: nwsurveying@nwsrvy.com

SCALE 1 INCH = 20 FEET
0 20 40

Ankrom Moisan
38 NORTHWEST DAVIS, SUITE 300
PORTLAND, OR 97209
503.245.7100

1505 5TH AVE, SUITE 300
SEATTLE, WA 98101
206.576.1600

1014 HOWARD STREET
SAN FRANCISCO, CA 94103
415.252.7063

kpff
111 SW Fifth Ave., Suite 2600
Portland, OR 97204
O: 503.542.3860
F: 503.274.4681
www.kpff.com

WESTGATE AND HALL
3775 SW HALL BOULEVARD, BEAVERTON, OR 97005

CEDARst

REVISION	DATE	REASON FOR ISSUE

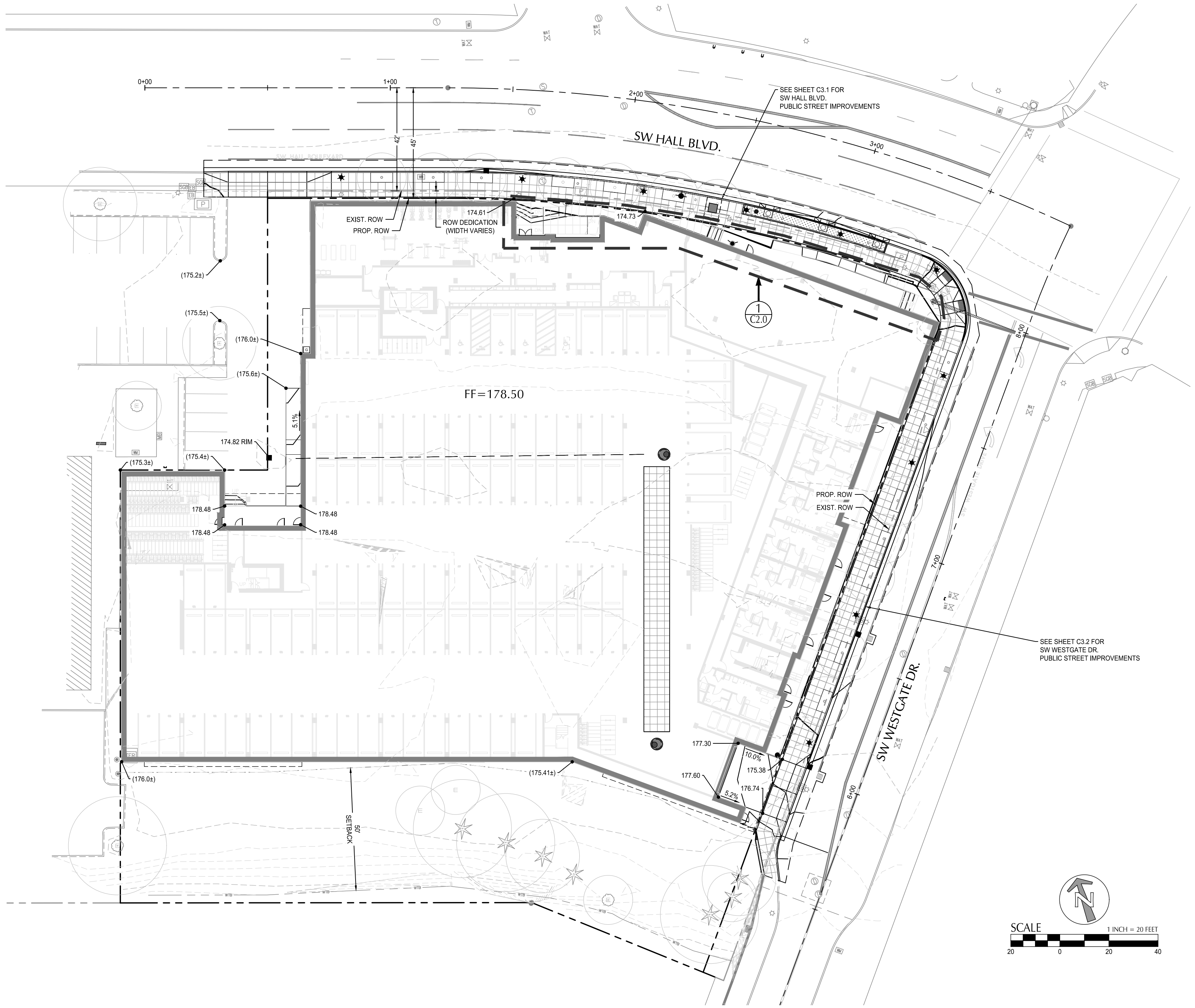
EXISTING CONDITIONS PLAN

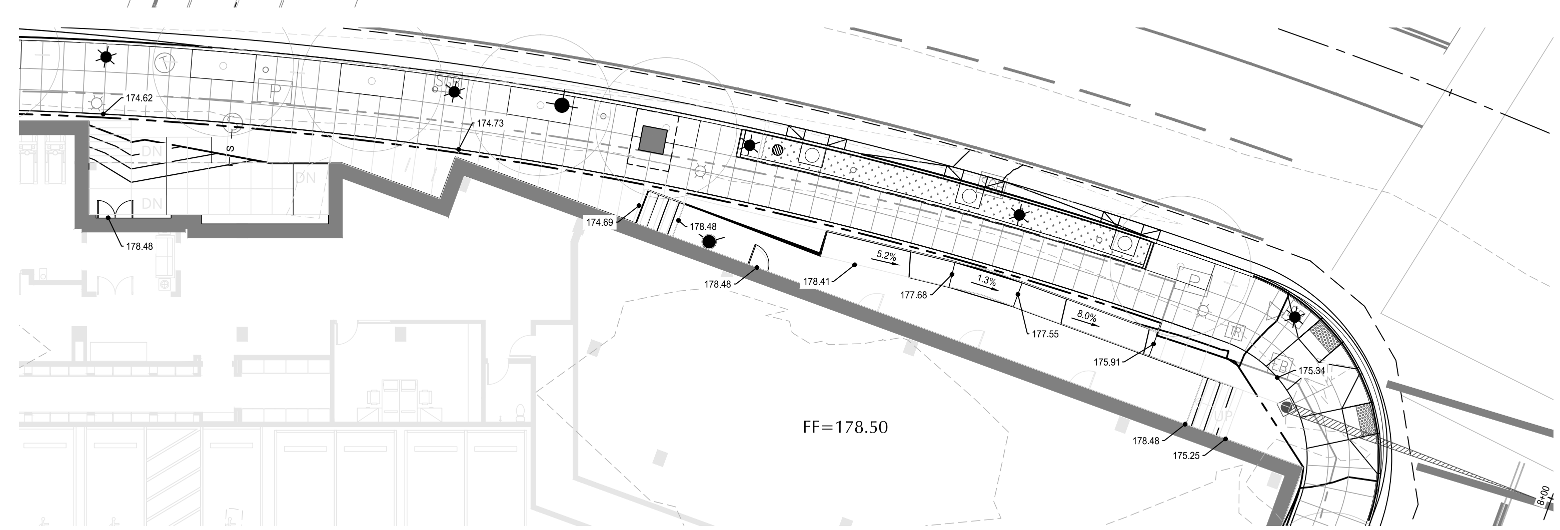
SCHEMATIC DESIGN

DATE 01/20/2023
PROJECT NUMBER 221970

SHEET NUMBER **C0.2**

File: \\civilpdx1\civil-projects\c\p\2022\2200347-westgate-hall-mixed-use\CAD\PLOT\22347-00-EX.dwg TAB:C0.2
Plotted: 4/17/23 at 12:33pm By: RCadua





1 ENLARGEMENT
SCALE: 1" = 10'

SHEET NOTES

1. SLOPES PROVIDED ON SLOPE ARROW ARE FOR REFERENCE ONLY.
2. LANDINGS ON ACCESSIBLE ROUTES SHALL NOT EXCEED 2% IN ANY DIRECTION.
3. ALL ACCESSIBLE ROUTES SHALL COMPLY WITH CURRENT ADA ACCESSIBILITY GUIDELINES FOR BUILDING AND FACILITIES (ADAAG).

GRADING LABEL LEGEND

CALLOUT	DESCRIPTION
X.X%	GRADING SLOPE AND DIRECTION (DOWNHILL)
[X.X%]	SLOPE ORIENTATION INDICATING DIRECTION OF MAXIMUM GRADE (DOWNHILL)
XX.XX XX	SPOT ELEVATION; DESCRIPTION LISTED BELOW. NO DESCRIPTION MEANS TP OR TG
BOW	BACK OF WALK
BS	BOTTOM OF STEP
BW	BOTTOM OF WALL
EG	EXISTING GRADE
FF	FINISHED FLOOR
FL	FLOW LINE
G	GUTTER
HP	HIGH POINT
LP	LOW POINT
RIM	RIM OF STRUCTURE
TC	TOP OF CURB
TG	TOP OF GROUND
TP	TOP OF PAVEMENT
TS	TOP OF STEP
TW	TOP WALL
(XXX.X±)	EXISTING GRADE (MATCH WHERE APPLICABLE)

SHEET LEGEND

- DRAINAGE FLOW DIRECTION
- GRADE BREAK
- SAWCUT LINE
- 49 EX. CONTOUR MINOR
- 50 EX. CONTOUR MAJOR
- 49 CONTOUR MINOR (FG)
- 50 CONTOUR MAJOR (FG)

NOT FOR CONSTRUCTION

Ankrom Moisan
38 NORTHWEST DAVIS, SUITE 300
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www.kpff.com

WESTGATE AND HALL
3775 SW HALL BOULEVARD, BEAVERTON, OR 97005

CEDARst

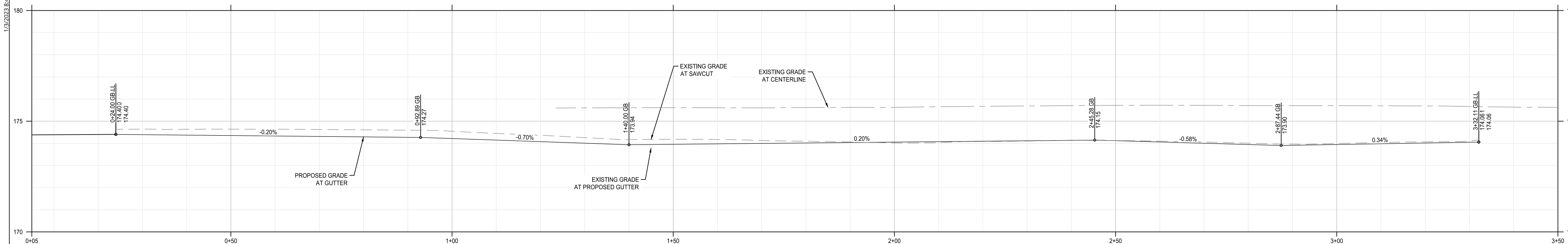
REVISION	DATE	REASON FOR ISSUE

SITE GRADING PLAN

SCHEMATIC DESIGN

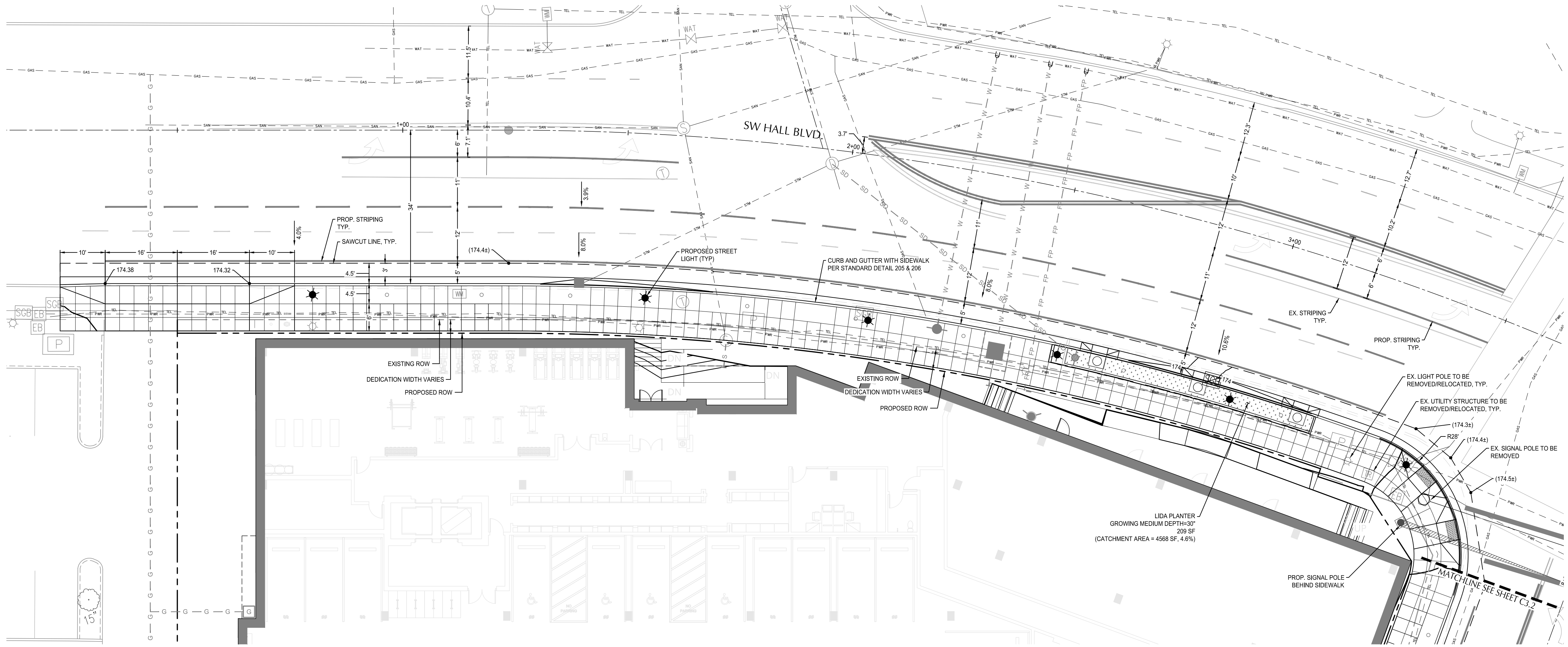
DATE 01/20/2023
PROJECT NUMBER 221970
SHEET NUMBER
C2.0

File: \\civil\pdx1\civil-projects\cp\2022\2200347-westgate-hall-mixed-use\CAD\PLOT\2200347-20-ST.dwg TAB:C2.0
Plotted: 4/17/23 at 12:34pm By: RCadua



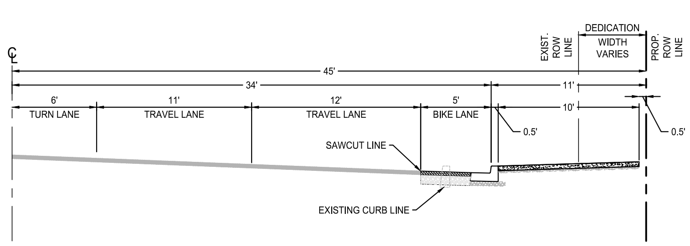
PROFILE VIEW - SW HALL BLVD.

SCALE: HORZ: 1" = 10'
VERT: 1" = 2'



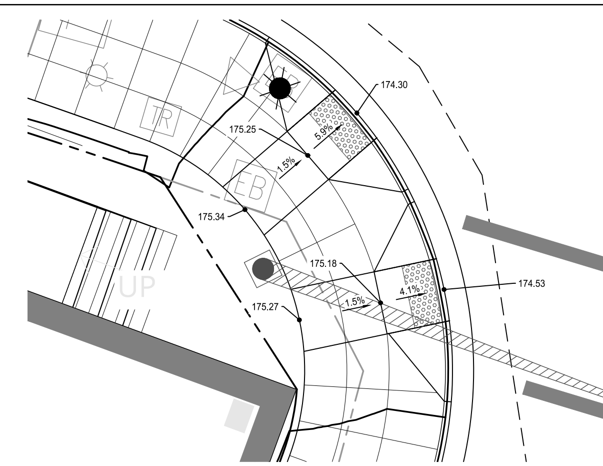
PLAN VIEW - SW HALL BLVD.

SCALE: 1" = 10'



A TYPICAL SECTION - SW HALL BLVD

SCALE: 1" = 5'



A CORNER GRADING

SCALE: 1" = 5'

SCALE 1 INCH = 10 FEET

10 0 10 20

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WESTGATE AND HALL

3775 SW HALL BOULEVARD, BEAVERTON, OR 97005

CEDARst

REVISION	DATE	REASON FOR ISSUE

PUBLIC STREET IMPROVEMENT PLAN - SW HALL

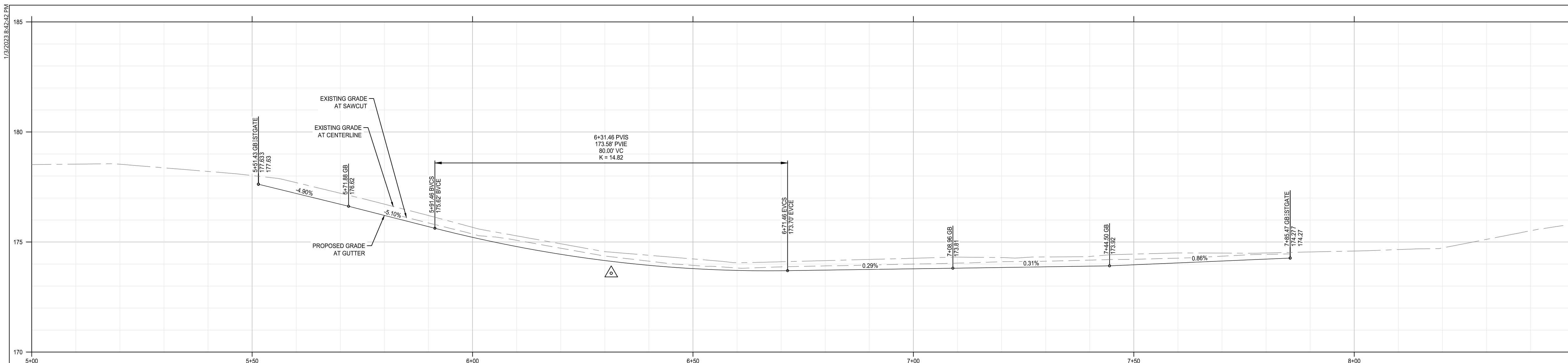
SCHEMATIC DESIGN

DATE	PROJECT NUMBER
01/20/2023	221970

SHEET NUMBER

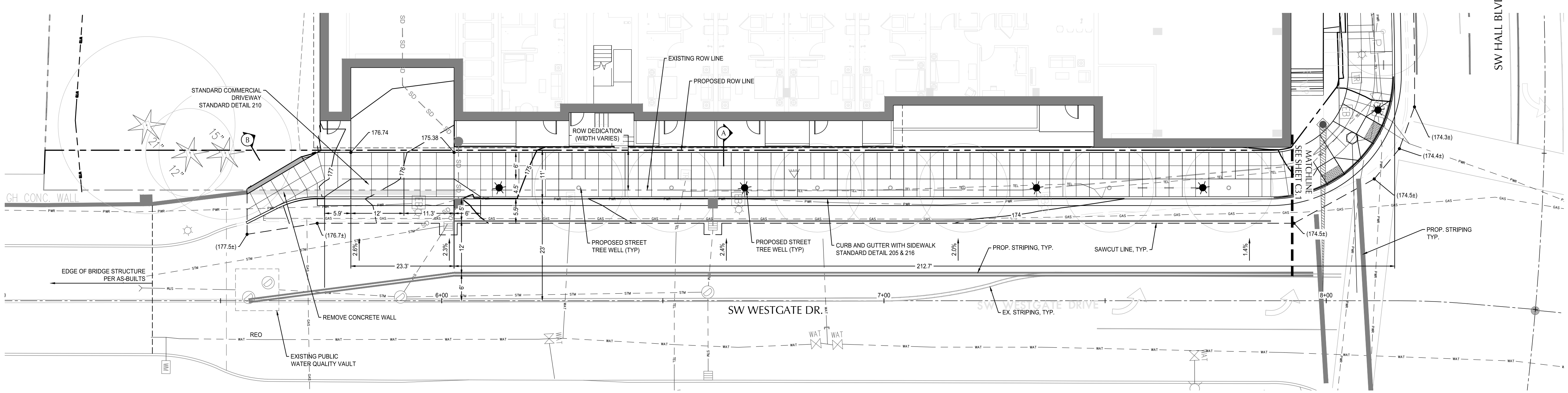
C3.1

File: \\cnf\pdx1\Civil-Projects\cp\2022\2200347-westgate-hall-mixed-use\CAD\PLOT\22347-31-P&P.dwg TAB:C3.1
Plotted: 4/17/23 at 12:34pm By: RCadua



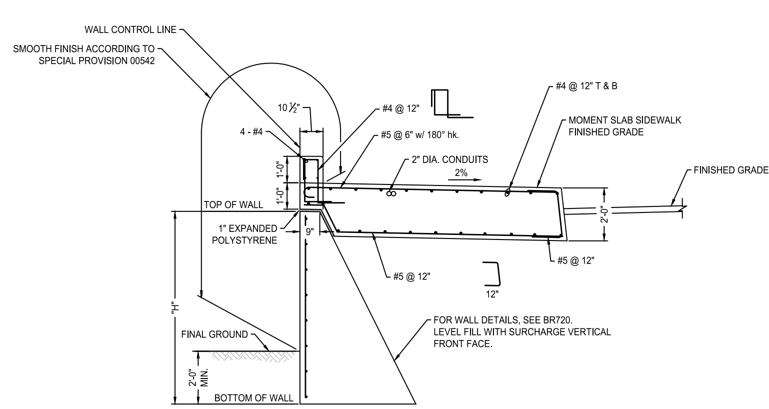
PROFILE VIEW - SW WESTGATE DR.

SCALE: HORZ: 1" = 10'
VERT: 1" = 2'



PLAN VIEW - SW WESTGATE DR.

SCALE: 1" = 10'

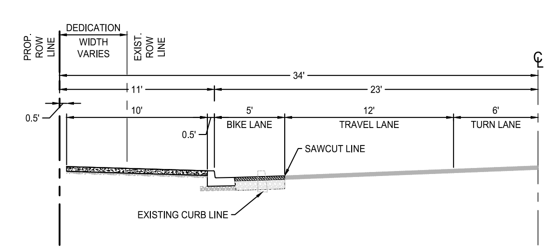


LEFT WALL SHOWN
(RIGHT WALL SIMILAR, OPPOSITE HAND)

TYPICAL WALL SECTION 12" CURB

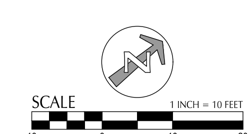
B TYPICAL SIDEWALK/WALL SECTION
(MATCHING ORIGINAL AS-BUILT DESIGN)

SCALE: NTS



A TYPICAL SECTION - SW WESTGATE DR.

SCALE: 1" = 5'



File: \\civilpdx1\Civil-Projects\cjb\2022\220347-westgate-hall-mixed-use\CAD\PLOT\22347-31-P&P.dwg TAB:C3.2
Plotted: 4/17/23 at 12:34pm By: RCadua

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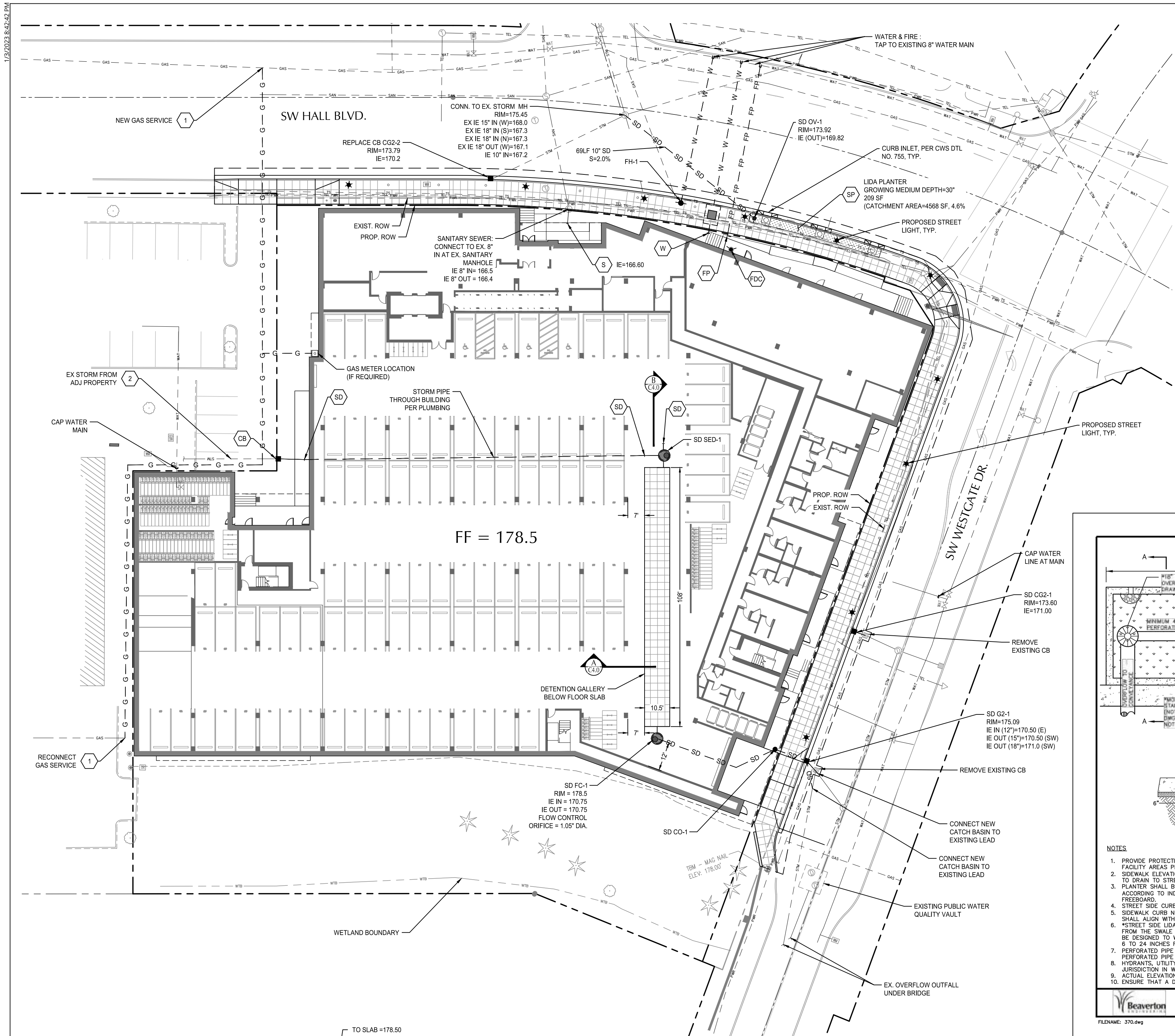
REVISION	DATE	REASON FOR ISSUE

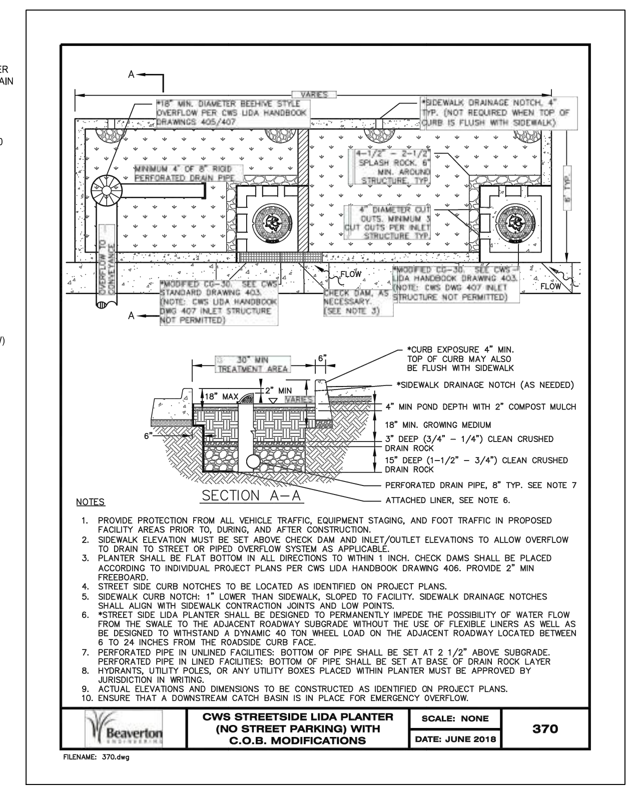
PUBLIC STREET IMPROVEMENT PLAN - SW WESTGATE

SCHEMATIC DESIGN

DATE 01/20/2023
PROJECT NUMBER 221970

SHEET NUMBER
C3.2





SHEET NOTES

1. UTILITIES ON HALL BLVD ARE APPROXIMATED BASED ON AVAILABLE CITY GIS MAPPING. FIELD VERIFY LOCATION AND ELEVATION OF CONNECTION POINTS
2. PIPE BEDDING AND BACKFILL FOR ALL UTILITIES SHALL BE DONE PER DETAIL 1/C501.
3. STATIONS AND OFFSETS SHOWN ON STRUCTURES ARE SHOWN AT CENTER OF STRUCTURE.

X UTILITY KEY NOTES

NOTE	DESCRIPTION	DETAIL REF.
1	CONTRACTOR TO COORDINATE WITH NW TO RECONNECT GAS SERVICE TO ADJACENT PROPERTY	
2	EX. STORM PENDING SCOPING CONFIRMATION TO DETERMINE EXTENT OF EX. STORM TO ADJACENT PROPERTY	
DG	DETENTION GALLERY, COMPOSED OF QUAD HEIGHT R-TANK MODULES	1/C4.0
FP	FIRE PROTECTION STUB FOR BUILDING	
FDC	FIRE DEPARTMENT CONNECTION (FDC), BUILDING MOUNTED	
SD	STORM STUB FOR BUILDING CONNECTION, SEE PLUMBING FOR CONTINUATION	
S	SANITARY STUB FOR BUILDING ,SEE PLUMBING FOR CONTINUATION	
SP	CWS STREET SIDE LIDA PLANTER (NO STREET PARKING) WITH C.O.B. MODIFICATIONS	COB DTL NO. 370
W	WATER SERVICE POINT OF CONNECTION	
!!	UTILITY CROSSING. PROVIDE 12" MIN. CLEARANCE, U.N.O.	

UTILITY LABEL LEGEND

STRUCTURE LABEL

UTILITY TYPE (SD=STORM DRAINAGE)
STRUCTURE TYPE CALLOUT
ID NUMBER (WHERE APPLICABLE)

XX XX-XX
RIM=
IE IN = XX.X
IE OUT = XX.X

STRUCTURE INFO (WHERE APPLICABLE)

PIPE LABEL

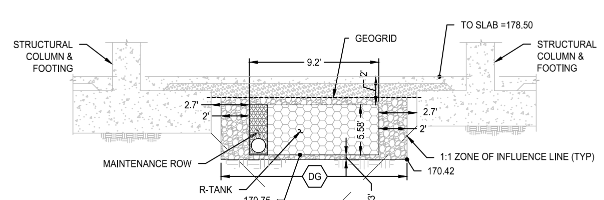
UTILITY LENGTH
UTILITY SIZE
UTILITY TYPE

XXLF - XX" XX
S=X.XX%

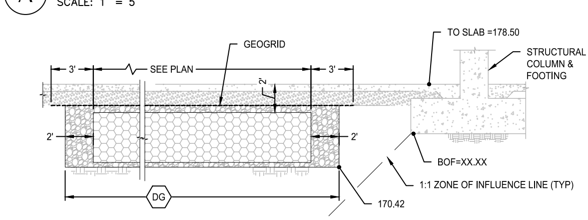
SLOPE (WHERE APPLICABLE)

STRUCTURE TYPE

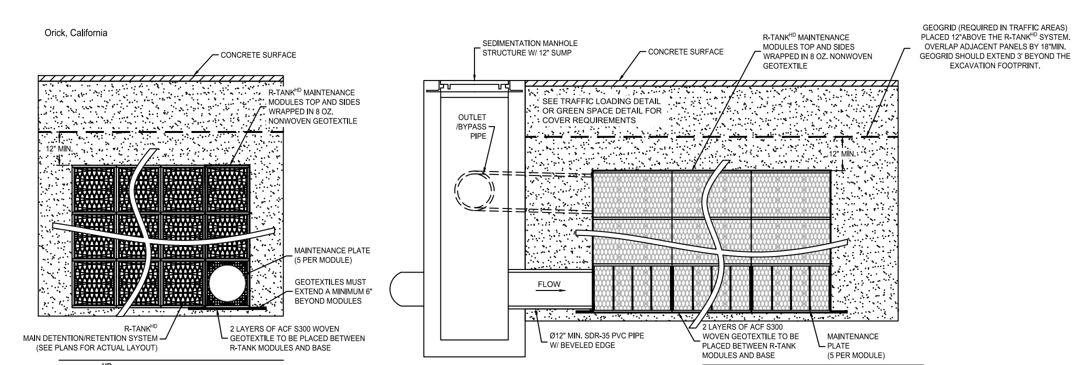
CALLOUT	DESCRIPTION	DETAIL REF.
45BND	45DEG BEND	
BWV	BACKWATER VALVE	X/C501
CB	CATCH BASIN	X/C501
CG-2	ODOT CONCRETE INLET TYPE CG-2	RD366/C501
G-2	ODOT CONCRETE INLET TYPE G-2	RD364/C501
CO	CLEANOUT TO GRADE	X/C501
DS	DOWNSPOUT	X/C501
FC	FLOW CONTROL MANHOLE	X/C501
FDC	FIRE DEPARTMENT CONNECTION	X/C501
FH	FIRE HYDRANT	X/C501
MH	MANHOLE	X/C501
SED	SEDIMENTATION MANHOLE	X/C501
OV	OVERFLOW INLET	CWS DWG NO. 760



A INFILTRATION GALLERY SECTION
SCALE: 1" = 5'



B INFILTRATION GALLERY SECTION
SCALE: 1" = 5'



5 INFILTRATION GALLERY MAINTENANCE ROW
SCALE: NTS

SCALE 1 INCH = 20 FEET

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REVISION	DATE	REASON FOR ISSUE

UTILITY PLAN

SCHEMATIC DESIGN

DATE 01/20/2023
PROJECT NUMBER 221970
SHEET NUMBER C4.0

Appendix E:

E1 R-Tank Email

E2 Tank R-Tank Design Submittal

E3 O&M for R-Tank

This page deliberately left blank to facilitate double-sided printing.

Josh Lighthipe

From: jason.bailey1@ferguson.com
Sent: Friday, April 7, 2023 4:01 AM
To: joseph.cotton@ferguson.com; Josh Lighthipe
Cc: Tyson Leggate; Justin Collinson; rob.woodman@ferguson.com
Subject: RE: Westgate & Hall project - Maintenance for R-tank under garage floor slab
Attachments: Westgate and Hall RTank HD 4-7-23.pdf

Categories: Filed by Newforma

Good Morning Team

Thanks for sending this over for our review.

We are moving away from under building designs because of the increased complexity and lack of coordination we were seeing from regions of the country. In addition to these issues, we wanted to ensure all systems are constructable and installed with the respect necessary for these types of situations. Therefore, going forward, prior to proposing an under building system, we would prefer to review them all to ensure we are comfortable with their size, location, complexity, etc.

In the case of this project, this is a simple rectangle where the engineer clearly demonstrates the understanding that minimum separation distances are required. I have attached a drawing packet based upon this layout. I recommend the treatment row extend the length of the units. Most maintenance will be done at the inlet locations, so it would be important to coordinate with your local maintenance department or contractor to ensure their vacuum truck has a hose length capacity that is not exceeded by this design. To limit in system maintenance, we include a pre-treatment screen at the last upstream structure, a detail is shown in the packet.

Additionally, you will note an acknowledgement and a series of notes on the cover sheet. Since this is under the building, no material will ship without a submittal review and approval. The contractor is required to participate in a pre-construction meeting and sign the pre-construction checklist. Additionally, documentation must be recorded by the engineer or contractor showing proper installation of the units and a focus on the proper installation of all connections.

If you have any questions, please let me know.

Thanks
Jason Bailey, PE
Engineering Services Manager

Ferguson Waterworks
C: 484-793-3018
E: jason.bailey1@ferguson.com
W: Ferguson Waterworks Geo & Stormwater

From: Joseph Cotton <joseph.cotton@ferguson.com>
Sent: Thursday, April 6, 2023 5:49 PM
To: Josh Lighthipe <Josh.Lighthipe@kpff.com>
Cc: Tyson Leggate <tyson.leggate@kpff.com>; justin.collinson@kpff.com; Jason Bailey <jason.bailey1@ferguson.com>;

Rob Woodman <rob.woodman@ferguson.com>
Subject: FW: Westgate & Hall project - Maintenance for R-tank under garage floor slab

Afternoon Josh,

Thanks for sending. I've copied in our Engineering Services Manager Jason Bailey as well as our National Urban Green Infrastructure Manager Rob Woodman on this project. We're taking a more conservative approach with R-Tank going under buildings at Ferguson, so please be sure to get Jason and Rob involved early on with any potential designs going under a building. The green light on utilizing R-Tank in these applications will need to come from Jason and Rob, not any of our distribution partners.

Jason/Rob – See below request from Josh along with his preliminary plans (attached). Josh is very knowledgeable about the R-Tank product and has specified R-Tank multiple times in the past. Josh is a great resource to discuss creative designs with. Please take a look at this one and get back to Josh with some direction.

Josh – Jason and Rob are on the East Coast so it may be tomorrow before you get a reply. Thanks for this opportunity and I'll talk to you soon.

Regards,

Joe Cotton
Geo / Stormwater Sales (Pacific Northwest)
Ferguson Waterworks | Geo & Stormwater Solutions
9129 N. Tyndall Ave., Portland, OR 97217
C: (503) 528-6190
E: joseph.cotton@ferguson.com
www.ferguson.com/waterworks

Josh Lighthipe

From: Josh Lighthipe
Sent: Thursday, April 13, 2023 1:50 PM
To: Josh Lighthipe
Subject: FW: R-Tank under building photos

Josh Lighthipe Associate, PE (OR, WA), LEED AP

Senior Project Manager | KPFF Portland Civil + Survey
O 503.542.3860 D 503.542.3840 M 971.235.2317

From: joseph.cotton@ferguson.com <joseph.cotton@ferguson.com>
Sent: Friday, April 7, 2023 8:28 AM
To: Josh Lighthipe <Josh.Lighthipe@kpff.com>
Subject: R-Tank under building photos

Morning Josh,

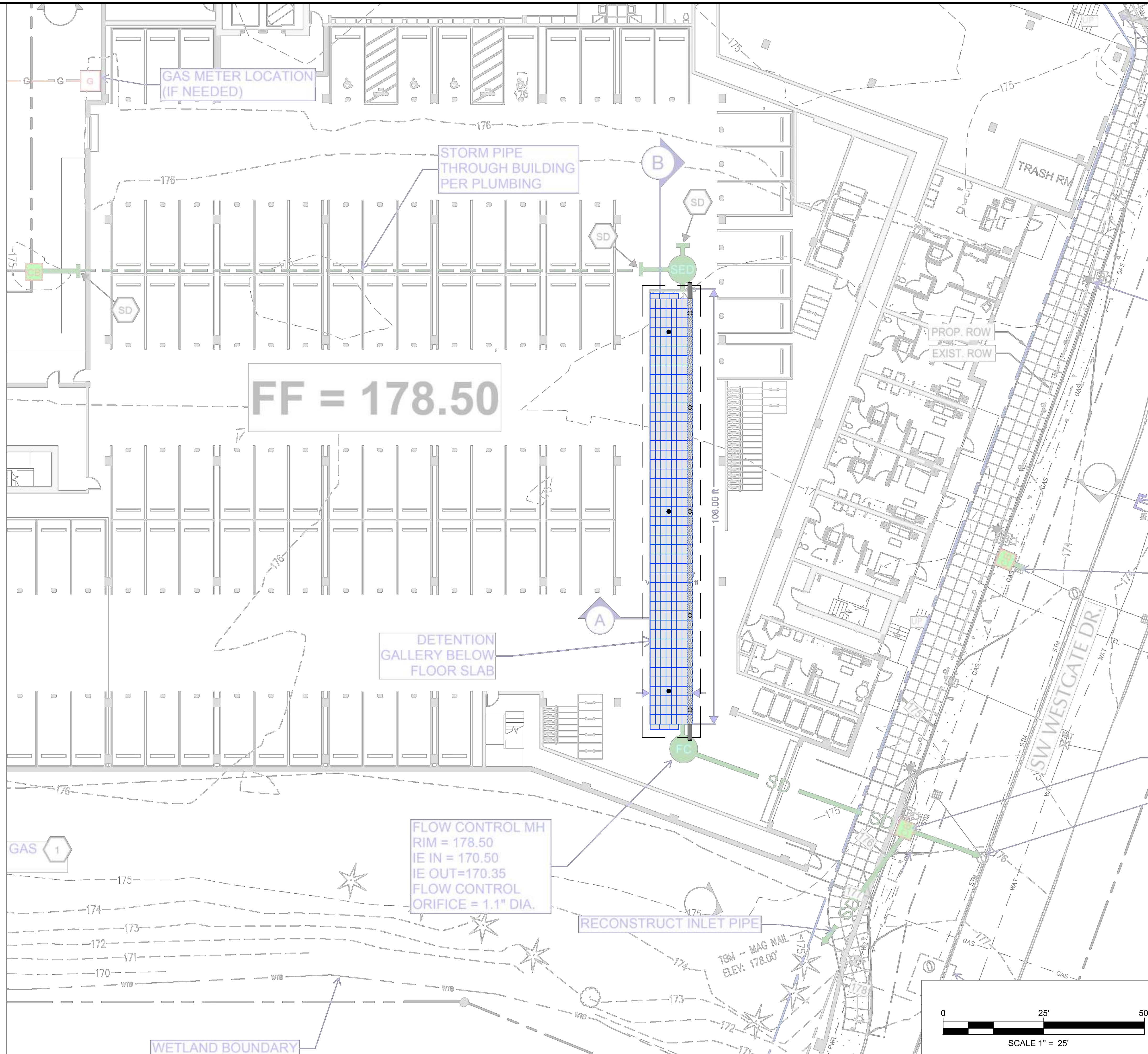
Here are some photos of the UD Single system under an apartment building at the Hillsboro 35 Apartments project installed in February of this year. No maintenance has been performed yet but I thought the photos may help your cause with a local example. A small system with a Treatment Row…


CAT

Sent from my iPhone

ACKNOWLEDGEMENT:
AN APPROVAL OF THE SUBMITTAL PLANS IS REQUIRED PRIOR TO MATERIAL ORDER. AS PART OF THE SUBMITTAL APPROVAL, THE ENGINEER OF RECORD HEREBY ACKNOWLEDGES THAT THE R-TANK SYSTEM IS NOT DESIGNED TO SUPPORT LOADS FROM BUILDINGS OR STRUCTURES. THEREFORE, THE ENGINEER OF RECORD HAS COORDINATED WITH THE PROPER DISCIPLINES TO ENSURE NO STRUCTURAL LOADS ARE IMPARTED UPON THE SYSTEM AND ANY INFILTRATION FROM THE SYSTEM HAS BEEN ACCOUNTED FOR IN THE FOUNDATION DESIGN.

NOTES:
- THE CONTRACTOR SHALL PARTICIPATE IN A PRECONSTRUCTION MEETING AND SIGN THE PRECONSTRUCTION CHECKLIST PRIOR TO MATERIAL INSTALLATION.
- DOCUMENTATION SHALL BE RECORDED BY THE CONTRACTOR OR ENGINEER OF RECORD SHOWING PROPER INSTALLATION OF THE SYSTEM AND ALL CONNECTIONS, IN ACCORDANCE WITH MANUFACTURER SPECIFICATIONS.
- IT IS HEREBY RECOMMENDED THAT THE R-TANK SYSTEM BE INSTALLED AFTER THE FOUNDATIONS HAVE BEEN INSTALLED TO ENSURE PROPER SEPARATION DISTANCES ARE MAINTAINED.



ENGINEER OF RECORD TO REVIEW, APPROVE AND ENDORSE FINAL SITE SPECIFIC DESIGN.

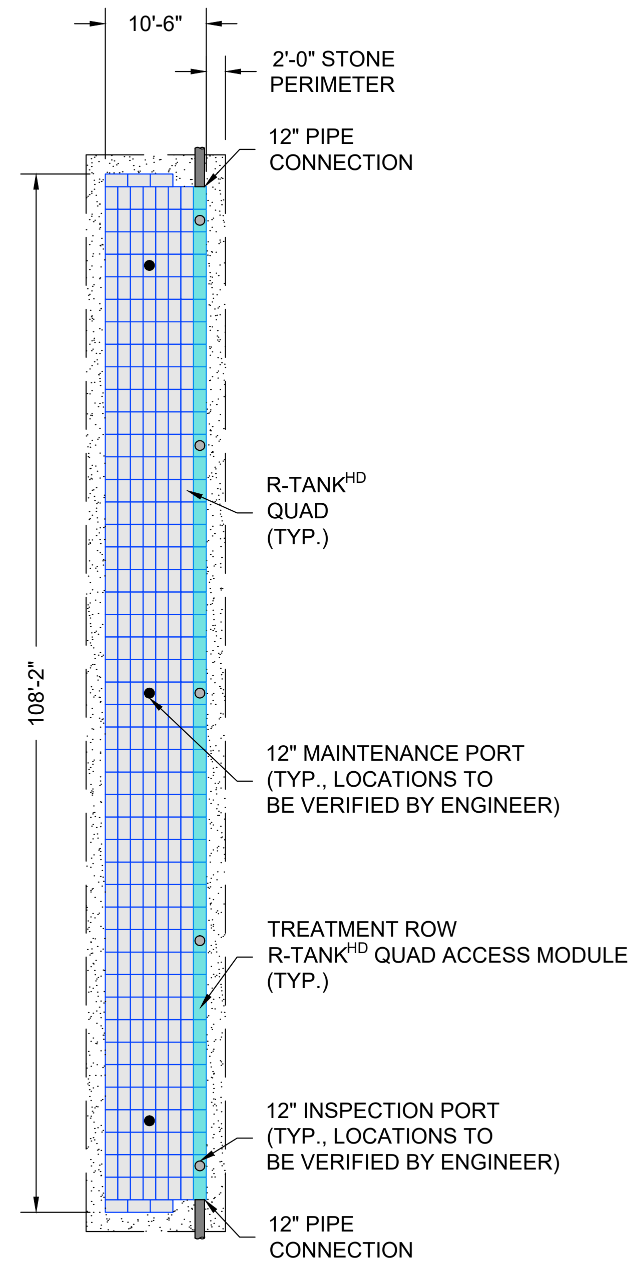
R-TANK HD

FERGUSON WATERWORKS

FOR ADDITIONAL INFORMATION PLEASE CONTACT: FERGUSON WATERWORKS, 1-800-448-3636, www.ferguson.com

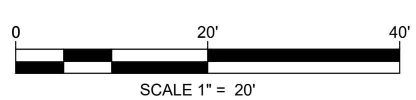
R-TANK^{HD} SYSTEM OVERLAY
WESTGATE AND HALL
BEAVERTON, OR

SCALE	1" = 25'
DRAWN BY	JKB
DATE	04/07/2023
SHEET NO.	1 of 7

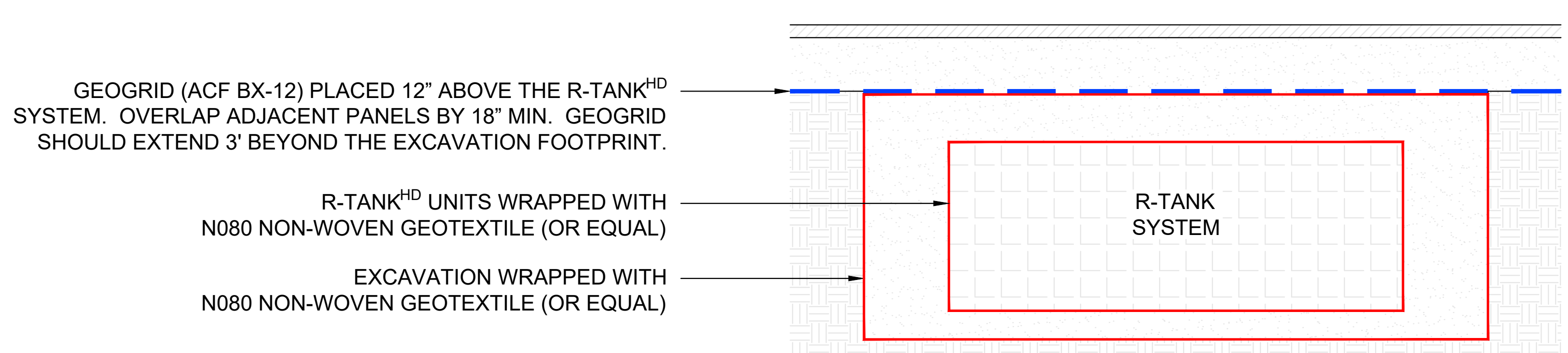


R-TANK QUANTITIES	
R-TANK^{HD} MODULE TYPE	QUAD
TRAFFIC LOAD	HS-25
# OF QUAD R-TANKS	321
# OF QUAD R-TANK ACCESS MODULES	45
TOTAL SYSTEM STORAGE	7,899 CF
R-TANK STORAGE VOLUME	5,971 CF
STONE STORAGE VOLUME (40% VOID RATIO)	1,929 CF
STONE BED FOOTPRINT	1,627 SF
STONE QUANTITY	179 CY
N080 NON-WOVEN GEOTEXTILE TANK WRAP	457 SY
N080 NON-WOVEN GEOTEXTILE EXCAVATION WRAP	637 SY
8 OZ. NON-WOVEN GEOTEXTILE TREAT. ROW WRAP	170 SY
S300 WOVEN GEOTEXTILE TREAT. ROW BASE FABRIC	38 SY
ACF BX-12 GEOGRID	310 SY
12" MAINTENANCE PORTS	3
12" INSPECTION PORTS	5
GEOTEXTILE PIPE BOOTS (8" | 12" | 15" | 18" | 24")	2
STORMRING CPS (6" | 8" | 12" | 15" | 18")	2
NOTE: STONE QUANTITY INCLUDES 12" OF COVER AND 3" OF BASE.	
NOTE: GEOTEXTILE / LINER QUANTITIES INCLUDE A 15% WASTE FACTOR.	
SEE SHEETS 3 - 7 FOR DETAILS AND ADDITIONAL INFORMATION	

GRAPHIC SCALE



R-TANK ELEVATIONS	
DESCRIPTION	ELEVATION
BASE INV.	170.25
TANK INV.	170.50
TOP OF TANK	176.08
GEOGRID	177.08
MIN. ALLOW. FINAL GRADE	178.08
MAX. ALLOW. FINAL GRADE	183.07



R-TANK^{HD} TANK WRAP & EXCAVATION ENVELOPE DETAIL

R-TANK HD

ENGINEER OF RECORD TO REVIEW, APPROVE AND ENDORSE FINAL SITE SPECIFIC DESIGN.

FERGUSON WATERWORKS

FOR ADDITIONAL INFORMATION PLEASE CONTACT: FERGUSON WATERWORKS, 1-800-448-3636, www.ferguson.com

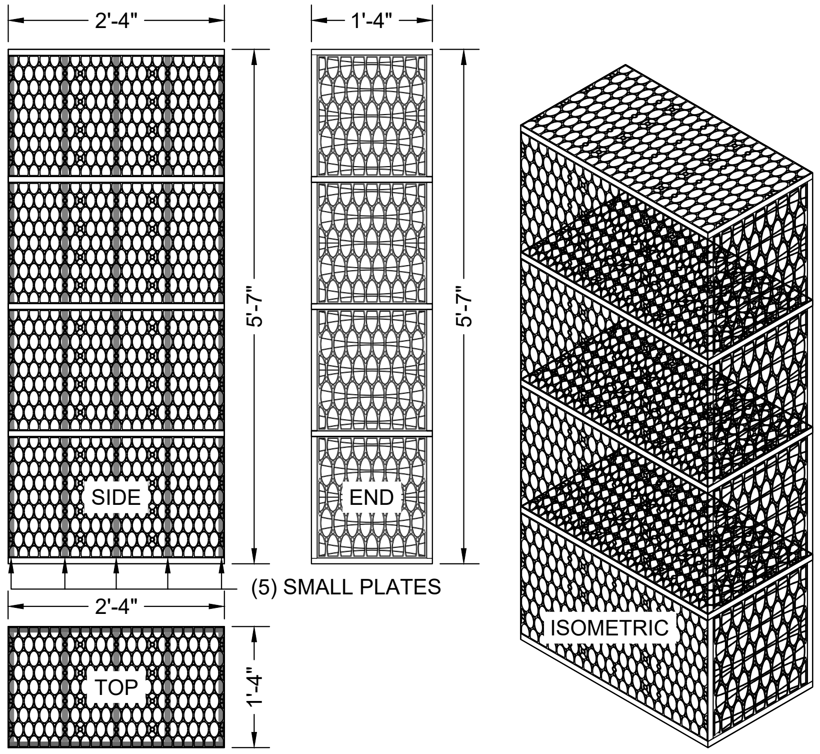
R-TANK^{HD} SYSTEM LAYOUT
WESTGATE AND HALL
BEAVERTON, OR

SCALE 1" = 20'

DRAWN BY JKB

DATE 04/07/2023

SHEET NO.



MODULE DATA

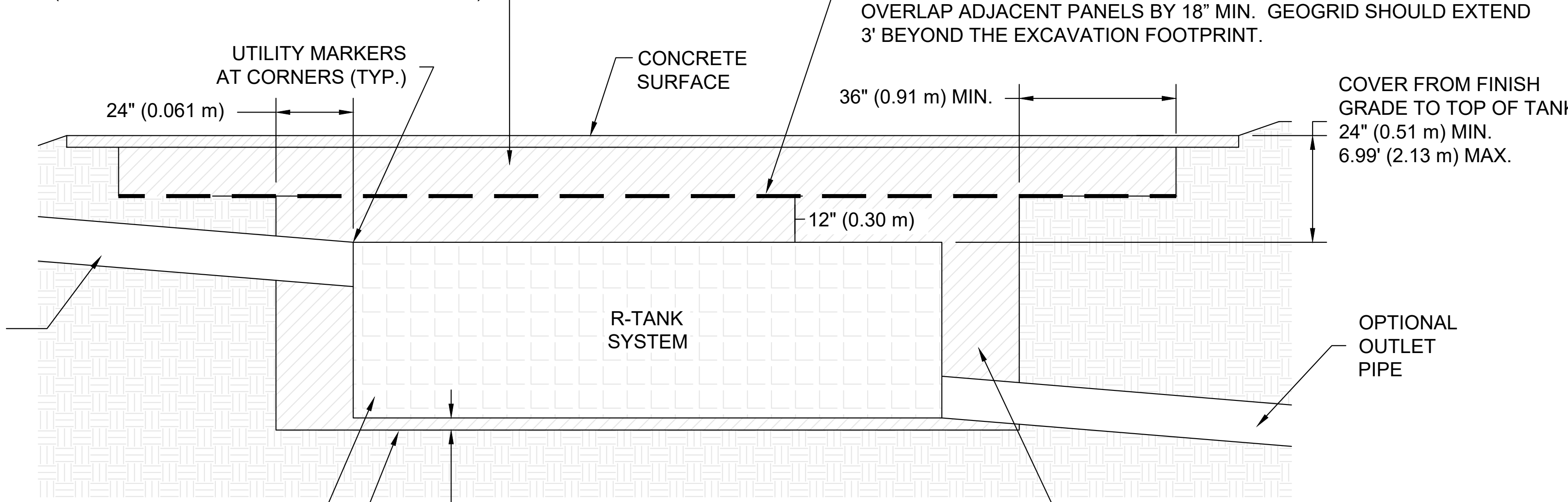
GEOMETRY:	**LOAD RATING:**
LENGTH = 28.15 IN. (715 MM)	33.4 PSI, (MODULE ONLY)
WIDTH = 15.75 IN. (400 MM)	HS20/HS25 - SEE SPEC FOR
HEIGHT = 66.93 IN. (1700 MM)	COVER REQUIREMENTS
TANK VOLUME = 17.17 CF	**MATERIAL:**
STORAGE VOLUME = 16.31 CF	100% RECYCLED POLYPROPYLENE
VOID INTERNAL VOLUME: 95%	**SMALL PLATES REQUIRED:**
VOID SURFACE AREA: 90%	5/SEGMENT, 20/MODULE

QUAD R-TANK^{HD} - MODULE DETAIL

TOTAL COVER: 20" MINIMUM AND 84" MAXIMUM. FIRST 12" MUST BE FREE DRAINING BACKFILL (SPEC SECTION 2.03B): STONE <1.5" OR SOIL (USCS CLASS GW, GP, SW OR SP). ADDITIONAL FILL MAY BE STRUCTURAL FILL (SPEC SECTION 2.03C): STONE OR SOIL (USCS CLASS SM, SP, SW, GM, GP OR GW) WITH MAX CLAY CONTENT<10%, MAX 25% PASSING NO. 200 SIEVE, AND MAX PLASTICITY INDEX OF 4. A MIN. 12" COVER MUST BE MAINTAINED BETWEEN BACKFILL EQUIPMENT AND THE TOP OF THE R-TANK™ SYSTEM AT ALL TIMES. TOTAL HEIGHT OF TOP BACKFILL SHOULD NOT EXCEED 7'. CONTACT FERGUSON WATERWORKS IF MORE THAN 7' OR LESS THAN 20" OF TOP BACKFILL IS REQUIRED (FROM TOP OF TANK TO TOP OF PAVEMENT).

NOTES:
1. FOR COMPLETE MODULE DATA, SEE APPROPRIATE R-TANK^{HD} MODULE SHEET .
2. INSTALLATIONS PER THIS DETAIL MEET GUIDELINES OF HL-93 LOADING PER THE AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS, CUSTOMARY U.S. UNITS, 7TH EDITION, 2014 WITH 2015 AND 2016 INTERIM REVISIONS.
3. PRE-TREATMENT STRUCTURES NOT SHOWN.
4. FOR INFILTRATION APPLICATIONS, GEOTEXTILE ENVELOPING R-TANK SHALL BE ACF M200 (PER SPEC SECTION 2.02A) AND BASE SHALL BE 4" MIN. UNCOMPACTED FREE DRAINING BACKFILL (SPEC SECTION 2.03A) TO PROVIDE A LEVEL BASE. SURFACE MUST BE SMOOTH, FREE OF LUMPS OR DEBRIS, AND EXTEND 2' BEYOND R-TANK^{HD} FOOTPRINT.

GEOGRID (ACF BX-12) PLACED 12" ABOVE THE R-TANK^{HD} SYSTEM. OVERLAP ADJACENT PANELS BY 18" MIN. GEOGRID SHOULD EXTEND 3' BEYOND THE EXCAVATION FOOTPRINT.



R-TANK^{HD} UNITS
LOAD RATING: 33.4 PSI (MODULE ONLY)

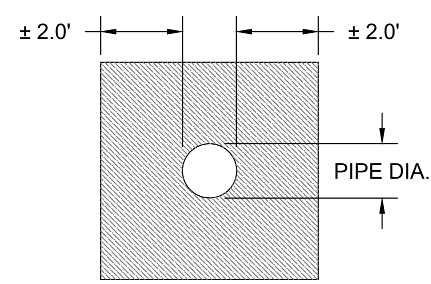
SUBGRADE / EXCAVATION LINE: COMPACT PER SPEC SECTION 3.02 D. A BEARING CAPACITY OF 2,000 PSF MUST BE ACHIEVED PRIOR TO INSTALLING R-TANK^{HD}

BASE: 3" (0.08 m) MIN. FREE DRAINING BACKFILL (SPEC SECTION 2.03B) COMPACTED TO 95% STANDARD PROCTOR DENSITY IS REQUIRED TO PROVIDE A LEVEL BASE SURFACE. MUST BE SMOOTH, FREE OF LUMPS OR DEBRIS, AND EXTEND 2' BEYOND R-TANK^{HD} FOOTPRINT. A BEARING CAPACITY OF 2,000 PSF MUST BE ACHIEVED PRIOR TO INSTALLING R-TANK^{HD}. NATIVE SOILS MAY BE ACCEPTABLE IF DETERMINED TO BE STABLE BY OWNER'S ENGINEER.

SIDE BACKFILL: FREE DRAINING BACKFILL (SPEC SECTION 2.03B): STONE <1.5" OR SOIL (USCS CLASS GW, GP, SW OR SP). MUST BE FREE FROM LUMPS, DEBRIS AND OTHER SHARP OBJECTS. SPREAD EVENLY TO PREVENT R-TANK^{HD} MOVEMENT. COMPACT SIDE BACKFILL WITH POWERED MECHANICAL COMPACTOR IN 12" LIFTS (PER SPEC SECTION 3.05 A2).

R-TANK^{HD} & HS-25 LOADS - SECTION VIEW

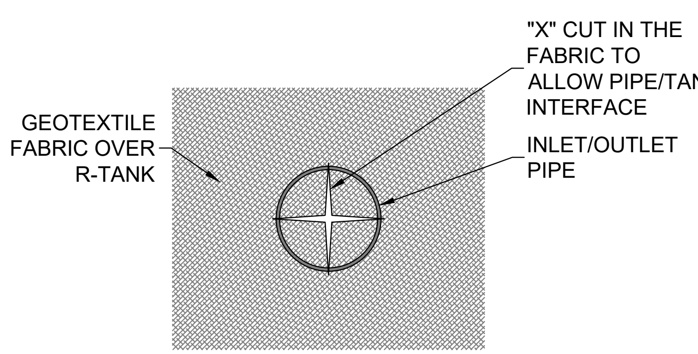
NOTE:
- PIPE BOOTS ARE AVAILABLE IN THE FOLLOWING STANDARD SIZES: 8" | 12" | 15" | 18" | 24".
- LARGER SPECIAL ORDER, CUSTOM SIZES ARE AVAILABLE.



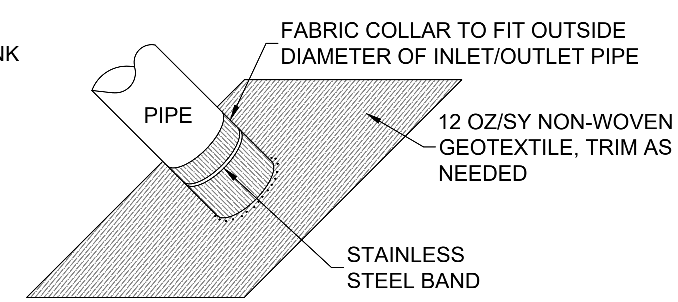
FRONT VIEW OF GEOTEXTILE BOOT

CUT AN "X" IN THE FABRIC ENVELOPE THAT IS SLIGHTLY LARGER THAN THE PIPE. PULL THE FABRIC FLAPS AROUND THE PIPE, AND SEAL WITH A STAINLESS STEEL BAND.

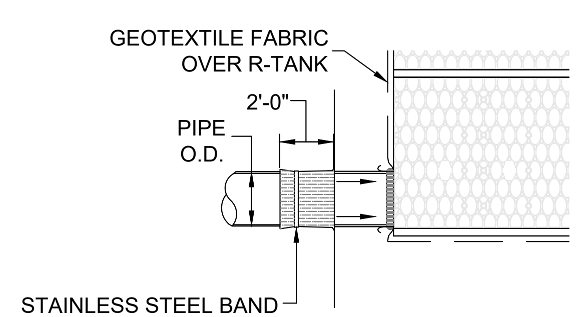
AFTER TANK WRAP IS SECURED TO PIPE, SLIDE BOOT AGAINST R-TANK AND SECURE WITH SECOND STAINLESS STEEL BAND, THEN ATTACH BOOT FLAP TO TANK ENVELOPE FABRIC WITH DUCT TAPE OR OTHER ADHESIVE.



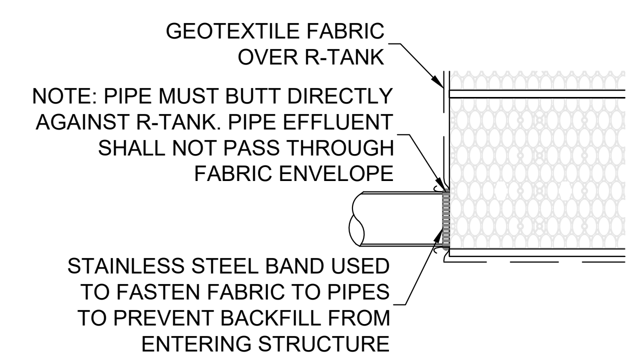
END VIEW OF PIPE/FABRIC CONNECTION



GEOTEXTILE BOOT



SIDE VIEW OF GEOTEXTILE BOOT



SIDE VIEW OF PIPE/FABRIC CONNECTION

R-TANK TYPICAL TANK INLET/OUTLET W/ GEOTEXTILE PIPE BOOT DETAIL

R TANK HD

ENGINEER OF RECORD TO REVIEW, APPROVE AND ENDORSE FINAL SITE SPECIFIC DESIGN.

FERGUSON WATERWORKS

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R-TANK^{HD} SYSTEM DETAILS
WESTGATE AND HALL
BEAVERTON, OR

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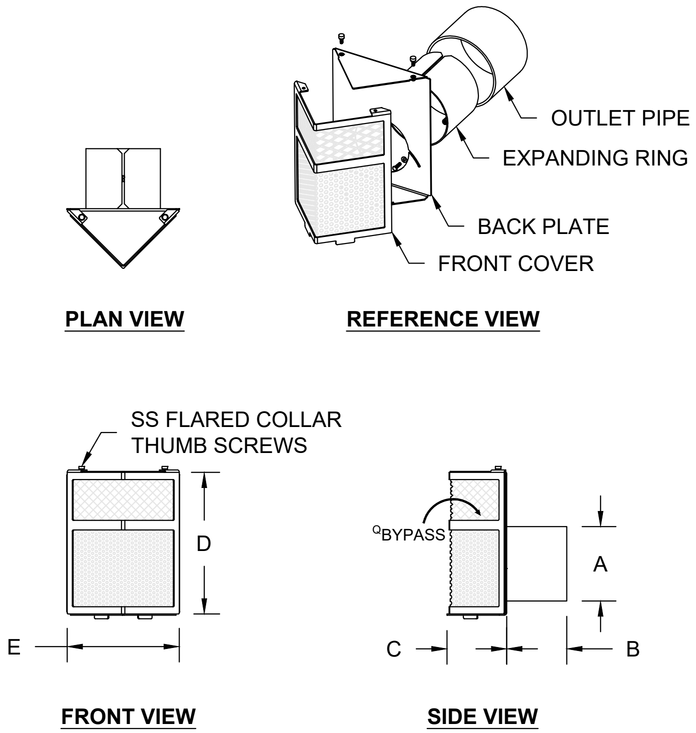
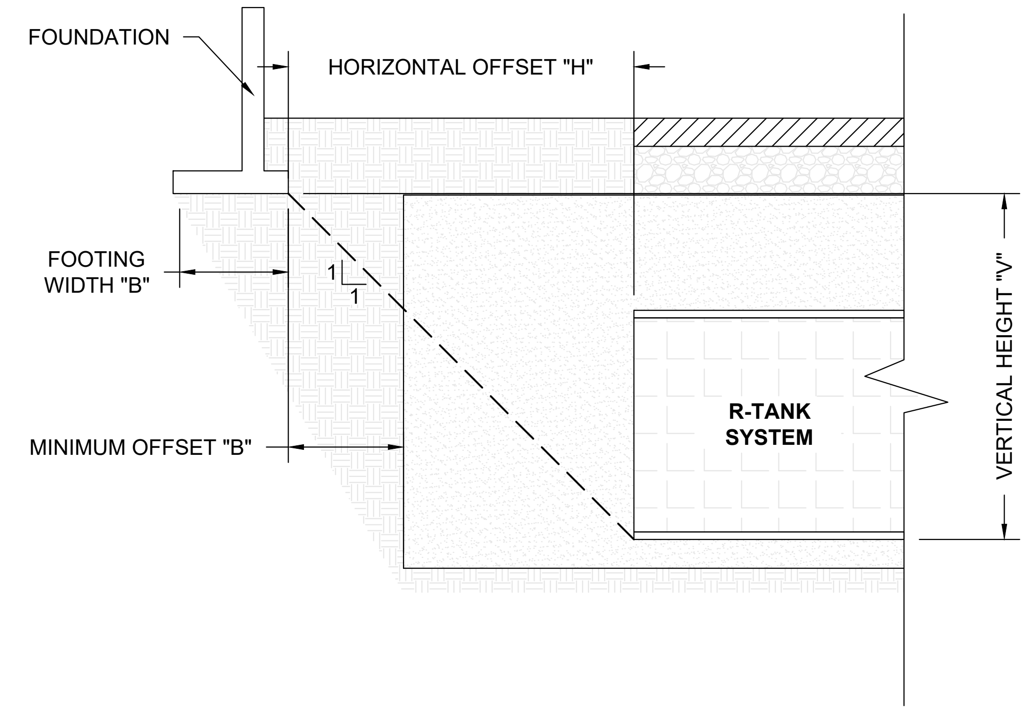


PIPE DIA. (A)	STUB LENGTH (B)	CPS DEPTH (C)	CPS HEIGHT (D)	CPS WIDTH (E)
6"	4.00"	8.50"	10.00"	10.00"
8"	6.00"	11.00"	12.25"	11.50"
12"	8.00"	16.00"	15.50"	15.50"
15"	8.00"	16.00"	21.50"	18.50"
18"	8.00"	16.00"	28.00"	21.50"

STORMRING CPS PRETREATMENT DETAIL



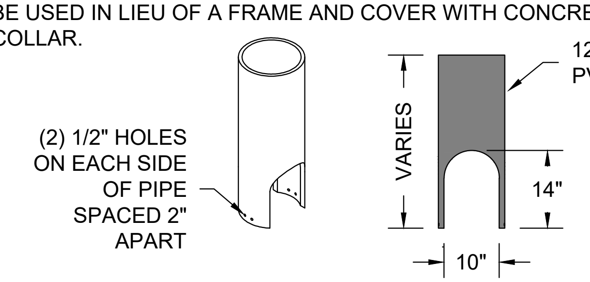
R-TANK IS NOT DESIGNED TO SUPPORT STRUCTURAL LOADS ASSOCIATED WITH BUILDINGS, RETAINING WALLS OR OTHER STRUCTURES.

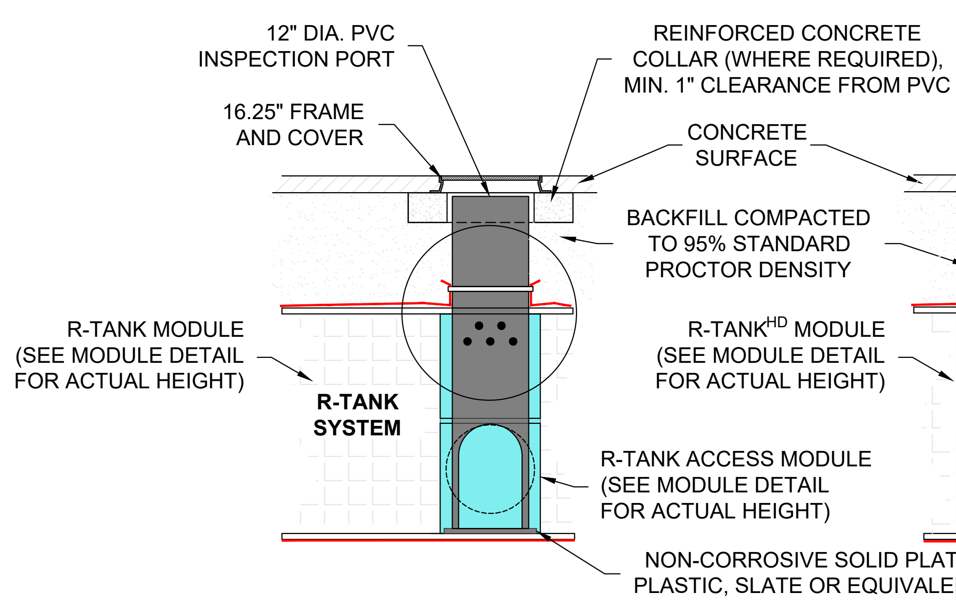
- FOUNDATIONS OF WIDTH "B" SHALL BE OFFSET A MINIMUM "B" FEET FROM THE STONE BACKFILL.
- THE CIVIL ENGINEER AND STRUCTURAL ENGINEER SHALL REVIEW ALL DOCUMENTATION AND DESIGNS TO ENSURE NO STRUCTURAL LOADS ARE APPLIED BY THE STRUCTURE ON THE R-TANK UNITS. IT IS RECOMMENDED THAT ALL R-TANK UNITS BE INSTALLED ABOVE THE ELEVATION OF ANY SUBSURFACE STRUCTURAL ELEMENTS SUCH AS FOOTERS. IF THIS CANNOT BE ACHIEVED, IT RECOMMENDED THAT A MINIMUM 1:1 ZONE OF INFLUENCE, SITE SPECIFIC SOILS MAY REQUIRE GREATER SEPARATION DISTANCE, BE MAINTAINED BETWEEN THE STRUCTURAL ELEMENT AND THE UNITS.
- THE STRUCTURAL ENGINEER AND GEOTECHNICAL ENGINEER SHALL REVIEW ALL DOCUMENTATION AND DESIGNS TO ENSURE PROPER STABILITY OF STRUCTURAL ELEMENTS DURING SATURATED SOIL CONDITIONS POTENTIALLY CAUSED BY THE PROXIMITY OF INFILTRATION FACILITIES TO THE STRUCTURAL ELEMENTS.

R-TANK - MINIMUM OFFSET DETAIL

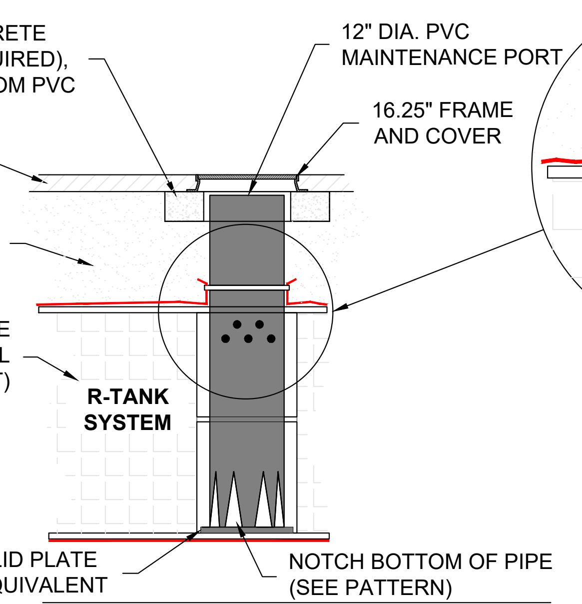
NOTES

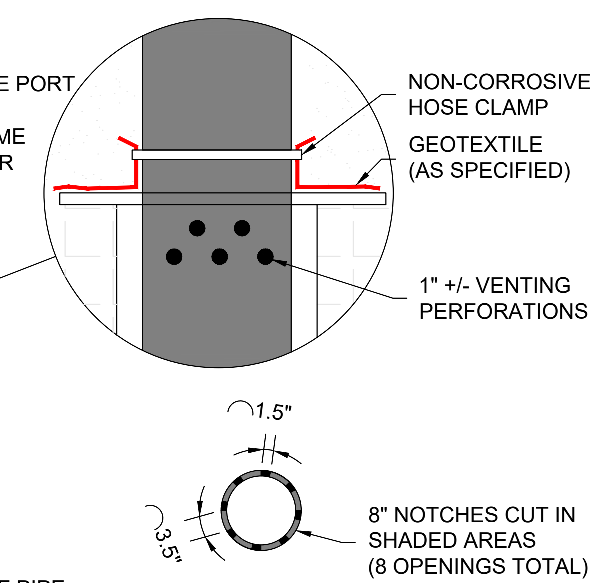
1. THE INSPECTION PORT IS USED IN THE ACCESS MODULE TO INSPECT THE LEVEL OF SEDIMENT ACCUMULATION.
2. THE MAINTENANCE PORT IS USED TO PUMP WATER INTO THE SYSTEM AND RE-SUSPEND ACCUMULATED SEDIMENT SO THAT IT MAY BE PUMPED OUT.
3. MINIMUM REQUIRED MAINTENANCE INCLUDES A QUARTERLY INSPECTION DURING THE FIRST YEAR OF OPERATION AND A YEARLY INSPECTION THEREAFTER. FLUSH AS NEEDED.
3. R-TANK^{HD}, R-TANK^{SD}, R-TANK^{UD} AND R-TANK^{XD} MAY BE USED IN TRAFFIC APPLICATIONS.
4. SEE TRAFFIC LOADING DETAIL FOR MINIMUM & MAXIMUM COVER REQUIREMENTS.
5. IF PORT IS LOCATED IN A NON-TRAFFIC AREA, A PLASTIC CAP CAN BE USED IN LIEU OF A FRAME AND COVER WITH CONCRETE COLLAR.





R-TANK TYPICAL INSPECTION PORT





PIPE NOTCHING PATTERN

R-TANK TYPICAL MAINTENANCE PORT

R-TANK HD

ENGINEER OF RECORD TO REVIEW, APPROVE AND ENDORSE FINAL SITE SPECIFIC DESIGN.

FERGUSON WATERWORKS

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R-TANK^{HD} SYSTEM DETAILS
WESTGATE AND HALL
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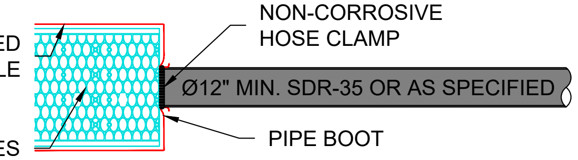
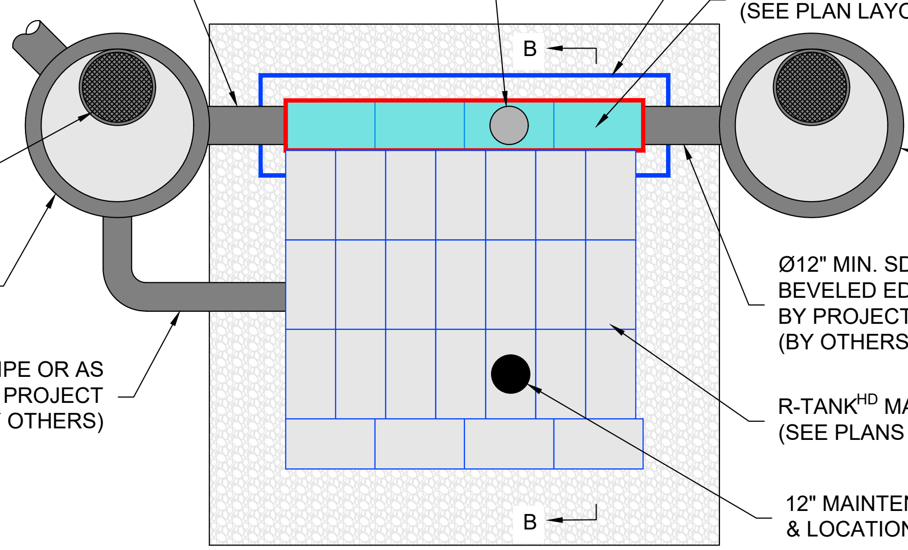
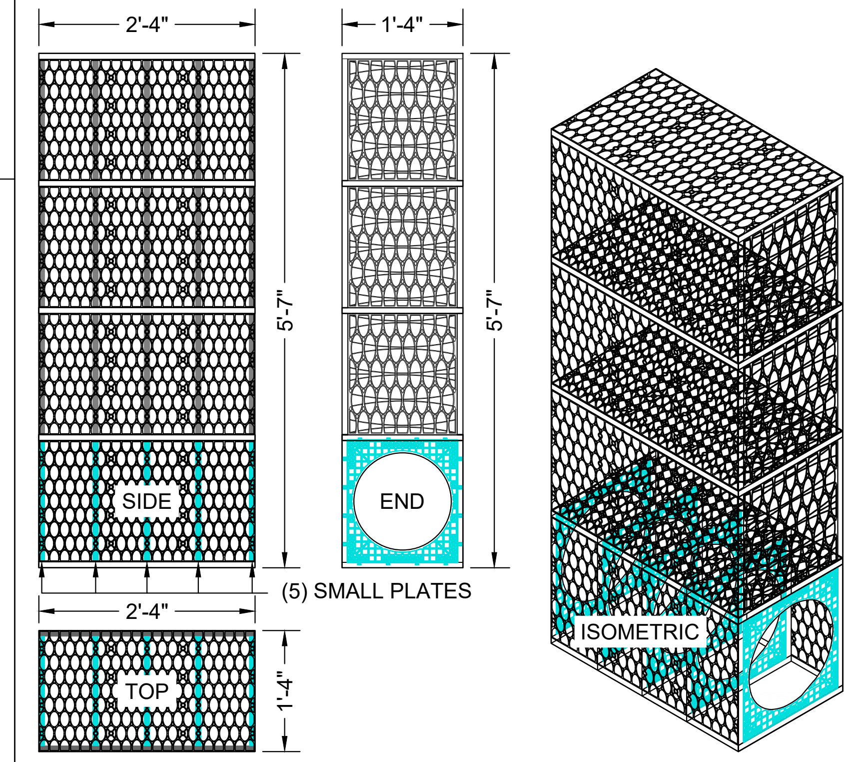


TYPICAL PIPE CONNECTION DETAIL

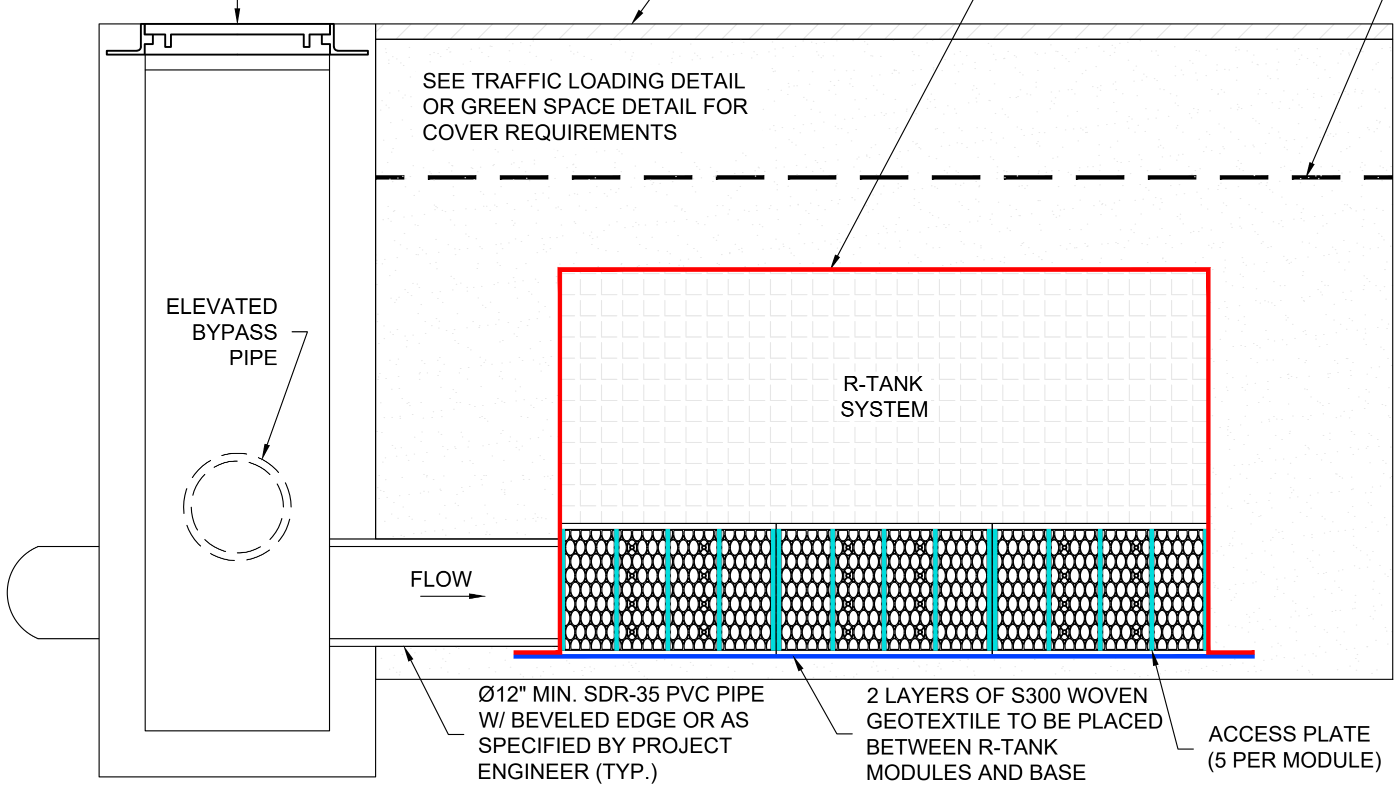
NOTE:
IF PIPE CONNECTING TO TREATMENT ROW IS LARGER THAN Ø12" SDR-35, IT SHALL BE ABUTTED FLUSH TO END PLATE AND SEALED WITH A PIPE BOOT.



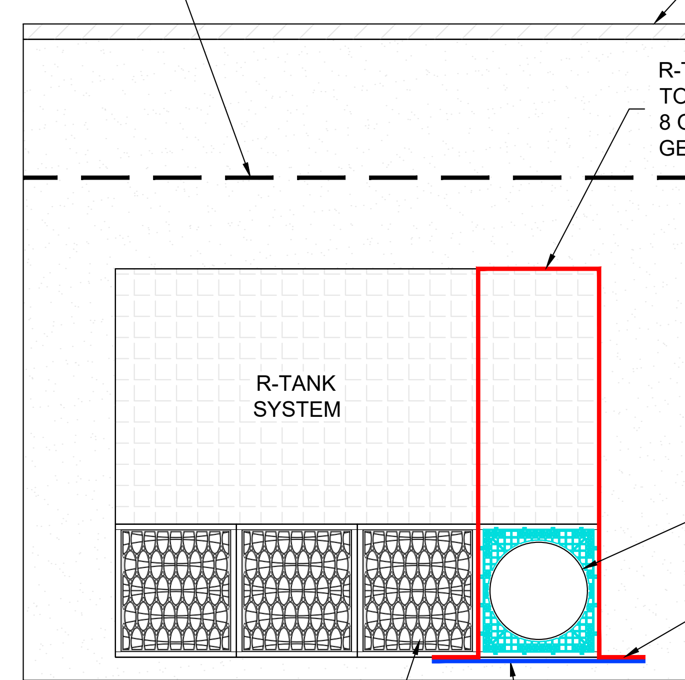
R-TANK HD TREATMENT ROW WITH PRECAST INLET/ACCESS STRUCTURE



QUAD R-TANK HD - ACCESS MODULE DETAIL
(FOR MODULE DATA, SEE STANDARD MODULE DETAIL)



R-TANK HD TREATMENT ROW SECTION A-A



R-TANK HD TREATMENT ROW SECTION B-B

ENGINEER OF RECORD TO REVIEW, APPROVE AND ENDORSE FINAL SITE SPECIFIC DESIGN.

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R-TANK HD TREATMENT ROW DETAILS
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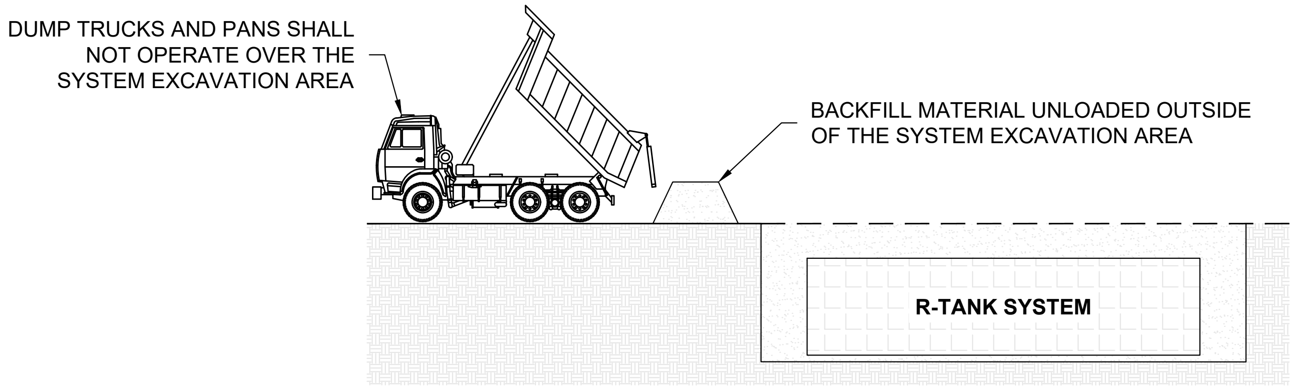
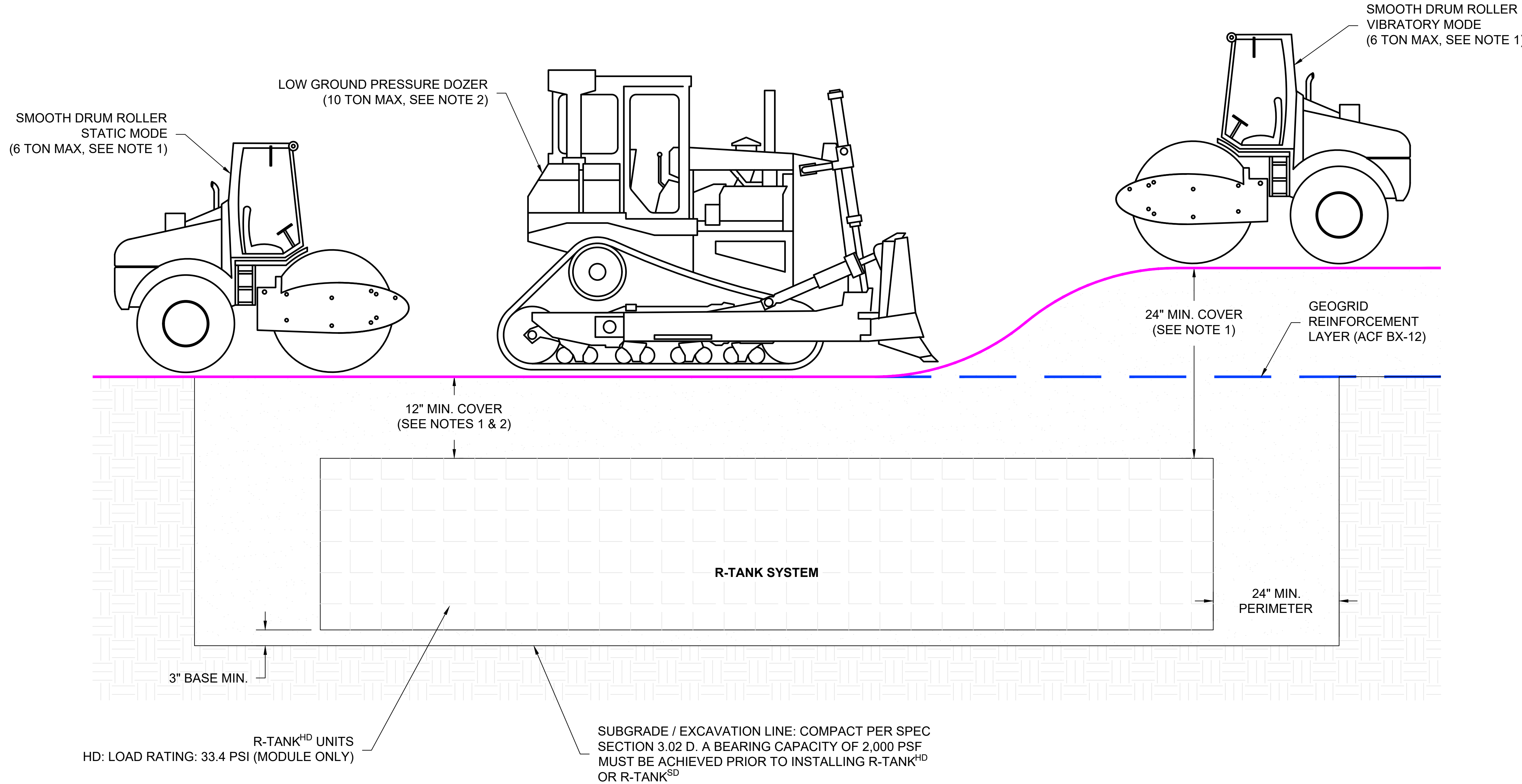


DUMP TRUCK DETAIL (SEE NOTE 3)

NOTES:

1. FOLLOWING PLACEMENT OF SIDE BACKFILL, A UNIFORM 12" LIFT OF THE FREELY DRAINING MATERIAL (SPEC SECTION 2.03 B2) SHALL BE PLACED OVER THE R-TANK AND LIGHTLY COMPACTED USING A WALK-BEHIND TRENCH ROLLER. ALTERNATELY, A ROLLER (MAXIMUM GROSS VEHICLE WEIGHT OF 6 TONS) MAY BE USED. ROLLER MUST REMAIN IN STATIC MODE UNTIL A MINIMUM OF 24" OF COVER HAS BEEN PLACED OVER THE MODULES. SHEEP FOOT ROLLERS SHOULD NOT BE USED. **SPEC SECTION 3.05 A5**
2. ONLY LOW PRESSURE TIRE OR TRACK VEHICLES (LESS THAN 7 PSI AND OPERATING WEIGHT OF LESS THAN 20,000 LBS) SHALL BE OPERATED OVER THE R-TANK SYSTEM DURING CONSTRUCTION. **SPEC SECTION 3.05 A5**
3. DUMP TRUCKS AND PANS SHALL NOT BE OPERATED WITHIN THE R-TANK SYSTEM AT ANY TIME. WHERE NECESSARY, THE HEAVY EQUIPMENT SHOULD UNLOAD IN AN AREA ADJACENT TO THE R-TANK SYSTEM AND THE MATERIAL SHOULD BE MOVED OVER THE SYSTEM WITH TRACKED EQUIPMENT. **SPEC SECTION 3.05 A5**
4. ENSURE THAT ALL UNRELATED CONSTRUCTION TRAFFIC IS KEPT AWAY FROM THE LIMITS OF EXCAVATION UNTIL THE PROJECT IS COMPLETE AND FINAL SURFACE MATERIALS ARE IN PLACE. NO NON-INSTALLATION RELATED LOADING SHOULD BE ALLOWED OVER THE R-TANK SYSTEM UNTIL THE FINAL DESIGN SECTION HAS BEEN CONSTRUCTED (INCLUDING PAVEMENT). **SPEC SECTION 3.05 B**
5. SEE R-TANK INSTALLATION GUIDE OR CONTACT YOUR LOCAL FERGUSON REPRESENTATIVE FOR ADDITIONAL INFORMATION.



CONSTRUCTION EQUIPMENT COVER DETAIL - VEHICULAR TRAFFIC

R-TANK HD

ENGINEER OF RECORD TO REVIEW, APPROVE AND ENDORSE FINAL SITE SPECIFIC DESIGN.

FERGUSON WATERWORKS

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R-TANK^{HD} CONSTRUCTION EQUIPMENT COVER DETAIL
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R-TANK SPECIFICATION

PART 1 - GENERAL

1.01 RELATED DOCUMENTS

A. Drawings, technical specification and general provisions of the Contract as modified herein apply to this section.

1.02 DESCRIPTION OF WORK INCLUDED

A. Provide excavation and base preparation per geotechnical engineer's recommendations and/or as shown on the design drawings, to provide adequate support for project design loads and safety from excavation sidewall collapse. Excavations shall be in accordance with the owner's and OSHA requirements.

B. Provide and install R-TankLD/, R-TankHD/, R-TankSD/, or R-TankU/D/ system (hereafter called R-Tank) and all related products including fill materials, geotextiles, geogrids, inlet and outlet pipe with connections per the manufacturer's installation guidelines provided in this section.

C. Provide and construct the cover of the R-Tank system including; stone backfill, structural fill cover, and pavement section as specified.

D. Protect R-Tank system from construction traffic after installation until completion of all construction activity in the installation area.

1.03 QUALITY CONTROL

A. All materials shall be manufactured in ISO certified facilities.

B. Installation Contractor shall demonstrate the following experience:

1. A minimum of three R-Tank or equivalent projects completed within 2 years; and,
2. A minimum of 25,000 cubic feet of storage volume completed within 2 years.
3. Contractor experience requirement may be waived if the manufacturer's representative provides on-site training and review during construction.

C. Installation Personnel: Performed only by skilled workers with satisfactory record of performance on bulk earthworks, pipe, chamber, or pond/landfill construction projects of comparable size and quality.

D. Contractor must have manufacturer's representative available for site review if requested by Owner.

1.04 SUBMITTALS

A. Submit proposed R-Tank layout drawings. Drawings shall include typical section details as well as the required base elevation of stone and tanks, minimum cover requirements and tank configuration.

B. Submit manufacturer's product data, including compressive strength and unit weight.

C. Submit manufacturer's installation instructions.

D. Submit R-Tank sample for review. Reviewed and accepted samples will be returned to the Contractor.

E. Submit material certificates for geotextile, geogrid, base course and backfill materials.

F. Submit required experience and personnel requirements as specified in Section 1.03.

G. Any proposed equal alternative product substitution to this specification must be submitted for review and approved prior to bid opening. Review package should include third party reviewed performance data that meets or exceeds criteria in Table 2.01 B.

1.05 DELIVERY, STORAGE, AND HANDLING

A. Protect R-Tank and other materials from damage during delivery, and store UV sensitive materials under tarp to protect from sunlight when time from delivery to installation exceeds two weeks. Storage of materials should be on smooth surfaces, free from dirt, mud and debris.

B. Handling is to be performed with equipment appropriate to the materials and site conditions, and may include hand, handcart, forklifts, extension lifts, etc.

C. Cold weather:

1. Care must be taken when handling plastics when air temperature is 40 degrees or below as plastic becomes brittle.
2. Do not use frozen materials or materials mixed or coated with ice or frost.
3. Do not build on frozen ground or wet, saturated or muddy subgrade.

1.06 PREINSTALLATION CONFERENCE.

A. Prior to the start of the installation, a preinstallation conference shall occur with the representatives from the design team, the general contractor, the excavation contractor, the R-Tank installation contractor, and the manufacturer's representative.

1.07 PROJECT CONDITIONS

A. Coordinate installation for the R-Tank system with other on-site activities to eliminate all non-installation related construction traffic over the completed R-Tank system. No loads heavier than the design loads shall be allowed over the system, and in no case shall loads higher than a standard AASHTO HS20 (or HS25, depending on design criteria) load be allowed on the system at any time.

B. Protect adjacent work from damage during R-Tank system installation.

C. All pre-treatment systems to remove debris and heavy sediments must be in place and functional prior to operation of the R-Tank system. Additional pretreatment measures may be needed if unit is operational during construction due to increased sediment loads.

D. Contractor is responsible for any damage to the system during construction.

PART 2 - PRODUCTS

2.01 R-TANK UNITS

A. R-Tank - Injection molded plastic tank plates assembled to form a 95% void modular structure of predesigned height (custom for each project).

B. R-Tank units shall meet the following Physical & Chemical Characteristics:

PROPERTY	DESCRIPTION	R-Tank^{LD} VALUE	R-Tank^{HD} VALUE	R-Tank^{SD} VALUE	R-Tank^{UD} VALUE
Void Area	Volume available for water storage	95%	95%	95%	95%
Surface Void Area	Percentage of exterior available for infiltration	90%	90%	90%	90%
Vertical Compressive Strength	ASTM D 2412 / ASTM F 2418	30.0 psi	33.4 psi	42.9 psi	134.2 psi
Lateral Compressive Strength	ASTM D 2412 / ASTM F 2418	20.0 psi	22.4 psi	28.9 psi	N/A
HS-20 Minimum Cover	Cover required to support HS-20 loads	N/A	20"	18"	12" (STONE BACKFILL)
HS-25 Minimum Cover	Cover required to support HS-25 loads	N/A	24"	19"	15" (STONE BACKFILL)
Maximum Cover	Maximum allowable cover depth	3 feet	< 7 feet	< 10 feet	5 feet
Unit Weight	Weight of plastic per cubic foot of tank	3.29 lbs / cf	3.62 lbs/cf	3.96 lbs / cf	4.33 lbs / cf
Rib Thickness	Thickness of load-bearing members	0.18 inches	0.18 inches	0.18 inches	N/A
Service Temperature	Safe temperature range for use	-14 – 167° F	-14 – 167° F	-14 – 167° F	-14 – 167° F

C. Supplier: Ferguson Waterworks 2831 Cardwell Road Richmond, VA 23234 (T): 800-448-3636; (F): 804-743-7779 www.ferguson.com

2.02 GEOSYNTHETICS

A. Geotextile. A geotextile envelope is required to prevent backfill material from entering the R-Tank modules.

1. Standard Application: The standard geotextile shall be an 8 oz per square yard nonwoven geotextile (ACF N080 or equivalent).
2. Infiltration Applications: When water must infiltrate/exfiltrate through the geotextile as a function of the system design, a woven monofilament (ACF M200 or equivalent) shall be used.

B. Geogrid. For installations subject to traffic loads and/or when required by project plans, install geogrid (ACF BX12 or equivalent) to reinforce backfill above the R-Tank system. Geogrid is not always required for R-TankUD/ installations, and is often not required for non-traffic load applications.

2.03 BACKFILL & COVER MATERIALS

A. Bedding Materials: Stone (angular and smaller than 1.5" in diameter) or soil (GW, GP, SW, or SP as classified by the Unified Soil Classification System) shall be used below the R-Tank system (3" minimum). Material must be free from lumps, debris, and any sharp objects that could cut the geotextile. Material shall be within 3 percent of the optimum moisture content as determined by ASTM D698 at the time of installation. For infiltration applications bedding material shall be free draining.

B. Side and Top Backfill: Material must be free from lumps, debris and any sharp objects that could cut the geotextile. Material shall be within 3 percent of the optimum moisture content as determined by ASTM D698 at the time of installation.

1. Traffic Applications - Free draining material shall be used adjacent to (24" minimum) and above (for the first 12") the R-Tank system.

a. For HD, and SD modules, backfill materials shall be free draining stone (angular and smaller than 1.5" in diameter) or soil (GW, GP, SW, or SP as classified by the Unified Soil Classification System).

b. For UD modules with less than 14" of top cover, backfill materials shall be free draining stone (angular and smaller than 1.5" in diameter). The use of soil backfill on the sides and top of the UD module is not permitted unless the modules are installed outside of traffic areas or with cover depths of 14" or more. Top backfill material (from top of module to bottom of pavement base or 12" maximum) must be consistent with side backfill.

2. Non-Traffic / Green Space Applications - For all R-Tank modules installed in green spaces and not subjected to vehicular loads, backfill materials may either follow the guidelines for Traffic Applications above, or the top backfill layer (12" minimum) may consist of AASHTO #57 stone blended with 30-40% (by volume) topsoil to aid in establishing vegetation.

C. Additional Cover Materials: Structural Fill shall consist of granular materials meeting the gradational requirements of SM, SP, SW, GM, GP or GW as classified by the Unified Soil Classification System. Structural fill shall have a maximum of 25 percent passing the No. 200 sieve, shall have a maximum clay content of 10 percent and a maximum Plasticity Index of 4. Material shall be within 3 percent of the optimum moisture content as determined by ASTM D698 at the time of installation.

2.04 OTHER MATERIALS

A. Utility Marker: Install metallic tape at corners of R-Tank system to mark the area for future utility detection.

PART 3 - EXECUTION

3.01 ASSEMBLY OF R-TANK UNITS

A. Assembly of modules shall be performed in accordance with the R-Tank Installation Manual, Section 2.

3.02 LAYOUT AND EXCAVATION

A. Installer shall stake out, excavate, and prepare the subgrade area to the required plan grades and dimensions, ensuring that the excavation is at least 2 feet greater than R-Tank dimensions in each direction allowing for installation of geotextile filter fabric, R-Tank modules, and free draining backfill materials.

B. All excavations must be prepared with OSHA approved excavated sides and sufficient working space.

C. Protect partially completed installation against damage from other construction traffic by establishing a perimeter with high visibility construction tape, fencing, barricades, or other means until construction is complete.

D. Base of the excavation shall be uniform, level, and free of lumps or debris and soft or yielding subgrade areas. A minimum 2,000 pounds per square foot bearing capacity is required.

1. Standard Applications: Compact subgrade to a minimum of 95% of Standard Proctor (ASTM D698) density or as required by the Owner's engineer.
2. Infiltration Applications: Subgrade shall be prepared in accordance with the contract documents. Compaction of subgrade should not be performed in infiltration applications.

E. Unsuitable Soils or Conditions: All questions about the base of the excavation shall be directed to the owner's engineer, who will approve the subgrade conditions prior to placement of stone. The owner's engineer shall determine the required bearing capacity of the R-Tank subgrade; however in no case shall a bearing capacity of less than 2,000 pounds per square foot be provided.

1. If unsuitable soils are encountered at the subgrade, or if the subgrade is pumping or appears excessively soft, repair the area in accordance with contract documents and/or as directed by the owner's engineer.
2. If indications of the water table are observed during excavation, the engineer shall be contacted to provide recommendations.
3. Do not start installation of the R-Tank system until unsatisfactory subgrade conditions are corrected and the subgrade conditions are accepted by the owner's engineer.

3.03 PREPARATION OF BASE

A. Place a thin layer (3" unless otherwise specified) of bedding material (Section 2.03 A), over the subgrade to establish a level working platform for the R-Tank modules. Level to within ½" (+/- ¼") or as shown on the plans. Native subgrade soils or other materials may be used if determined to meet the requirements of 2.03 A and are accepted by the owner's engineer.

1. Standard Applications: Static roll or otherwise compact bedding materials until they are firm and unyielding.
2. Infiltration Applications: Bedding materials shall be prepared in accordance with the contract documents.

B. Outline the footprint of the R-Tank system on the excavation floor using spray paint or chalk line to ensure a 2' perimeter is available around the R-Tank system for proper installation and compaction of backfill.

3.04 INSTALLATION OF THE R-TANKS

A. Where a geotextile wrap is specified on the stone base, cut strips to length and install in excavation, removing wrinkles so material lays flat. Overlap geotextile a minimum 12" or as recommended by manufacturer. Use tape, special adhesives, sandbags or other ballast to secure overlaps. As geotextiles can be damaged by extreme heat, smoking is not permissible on/near the geotextile, and tools using a flame to tack the overlaps, such as propane torches, are prohibited.

B. Where an impervious liner (for containment) is specified, install the liner per manufacturer's recommendations and the contract documents. The R-Tank units shall be separated from impervious liner by a non-woven geotextile fabric installed accordance with Section 3.04A.

C. Install R-Tank modules by placing side by side, in accordance with the design drawings. No lateral connections are required. It is advisable to use a string line to form square corners and straight edges along the perimeter of the R-Tank system. The modules are to be oriented as per the design drawing with required depth as shown on plans.

1. For LD, HD, and SD installations, the large side plate of the tank should be placed on the perimeter of the system. This will typically require that the two ends of the tank area will have a row of tanks placed perpendicular to all other tanks. If this is not shown in the construction drawings, it is a simple field adjustment that will have minimal effect on the overall system footprint. Refer to R-Tank Installation Guide for more details
2. For UD installations, there is no perpendicular end row required.

D. Wrap the R-Tank top and sides in specified geotextile. Cut strips of geotextile so that it will cover the sides and top, encapsulating the entire system to prevent backfill entry into the system. Overlap geotextile 12" or as recommended by manufacturer. Take great care to avoid damage to geotextile (and, if specified, impervious liner) during placement.

E. Identify locations of inlet, outlet and any other penetrations of the geotextile (and optional liner). These connections should be installed flush (butted up to the R-Tank) and the geotextile fabric shall be cut to enable hydraulic continuity between the connections and the R-Tank units. These connections shall be secured using pipe boots with stainless steel pipe clamps. Support pipe in trenches during backfill operations to prevent pipe from settling and damaging the geotextile, impervious liner (if specified) or pipe. Connecting pipes at 90 degree angles facilitates construction, unless otherwise specified. Ensure end of pipe is installed snug against R-Tank system.

F. Install Inspection and Maintenance Ports in locations noted on plans. At a minimum one maintenance port shall be installed within 10' of each inlet & outlet connection, and with a maximum spacing of one maintenance port for every 2,500 square feet. Install all ports as noted in the R-Tank Installation Guide.

G. If required, install ventilation pipes and vents as specified on drawings to provide ventilation for proper hydraulic performance. The number of pipes and vents will depend on the size of the system. Vents are often installed using a 90 degree elbow with PVC pipe into a landscaped area with 'U" bend or venting bollard to inhibit the ingress of debris. A ground level concrete or steel cover can be used.

3.05 BACKFILLING OF THE R-TANK UNITS

A. Backfill and fill with recommended materials as follows:

1. Place freely draining backfill materials (Section 2.03 B) around the perimeter in lifts with a maximum thickness of 12". Each lift shall be placed around the entire perimeter such that each lift is no more than 24" higher than the side backfill along any other location on the perimeter of the R-Tank system. No fill shall be placed over top of tanks until the side backfill has been completed.
2. Each lift shall be compacted at the specified moisture content to a minimum of 95% of the Standard Proctor Density until no further densification is observed (for self-compacting stone materials). The side lifts must be compacted with walk behind compaction equipment. Even when "self-compacting" backfill materials are selected, a walk behind vibratory compactor must be used.
3. Take care to ensure that the compaction process does not allow the machinery to come into contact with the modules due to the potential for damage to the geotextile and R-Tank units.
4. No compaction equipment is permissible to operate directly on the R-Tank modules.
5. Top Backfill: Only low pressure track vehicles shall be operated over the R-Tank system during construction. Dump Trucks and Pans shall not be operated within the R-Tank system footprint at any time. Heavy equipment should unload in an area adjacent to the R-Tank system and the material should be moved over the system using tracked equipment with an operating weight of less than 10 tons.
   a. Typical Applications: Install a 12" (or as shown on plans) lift of freely draining material (Section 2.03 B) over the R-Tank Units, maintaining 12" between equipment tracks and R-Tank System. Lightly compacted using a walk-behind trench roller. Alternately, a roller (maximum gross vehicle weight of 6 tons) may be used. Roller must remain in static mode until a minimum of 24" of cover has been placed over the modules. Sheep foot rollers should not be used.
   b. Shallow Applications (< 18" total cover): Install top backfill in accordance with plans.
6. If required, install a geogrid as shown on plans. Geogrid shall extend a minimum of 3 feet beyond the limits of the excavation wall.
7. Following placement and compaction of the initial cover, subsequent lifts of structural fill (Section 2.03 C) shall be placed at the specified moisture content and compacted to a minimum of 95% of the Standard Proctor Density and shall cover the entire footprint of the R-Tank system. During placement of fill above the system, unless otherwise specified, a uniform elevation of fill shall be maintained to within 12" across the footprint of the R-Tank system. Do not exceed maximum cover depths listed in Table 2.01 B.
8. Place additional layers of geotextile and/or geogrid at elevations as specified in the design details. Each layer of geosynthetic reinforcement placed above the R-Tank system shall extend a minimum of 3 feet beyond the limits of the excavation wall.

B. Ensure that all unrelated construction traffic is kept away from the limits of excavation until the project is complete and final surface materials are in place. No non-installation related loading should be allowed over the R-Tank system until the final design section has been constructed (including pavement).

C. Place surfacing materials, such as groundcovers (no large trees), or paving materials over the structure with care to avoid displacement of cover fill and damage to surrounding areas.

D. Backfill depth over R-Tank system must be within the limitations shown in the table in Section 2.01 B. If the total backfill depth does not comply with this table, contact engineer or manufacturer's representative for assistance.

3.06 MAINTENANCE REQUIREMENTS

A. A routine maintenance effort is required to ensure proper performance of the R-Tank system. The Maintenance program should be focused on pretreatment systems. Ensuring these structures are clean and functioning properly will reduce the risk of contamination of the R-Tank system and stormwater released from the site. Pre-treatment systems shall be inspected yearly, or as directed by the regulatory agency and by the manufacturer (for proprietary systems). Maintain as needed using acceptable practices or following manufacturer's guidelines (for proprietary systems).

B. All inlet pipes and Inspection and/or Maintenance Ports in the R-Tank system will need to be inspected for accumulation of sediments at least quarterly through the first year of operation and at least yearly thereafter.

C. If sediment has accumulated to the level noted in the R-Tank Maintenance Guide or beyond a level acceptable to the Owner's engineer, the R-Tank system should be flushed.

D. All inspection and maintenance activities should be performed in accordance with the R-Tank Operation, Inspection & Maintenance Manual.

ENGINEER OF RECORD TO REVIEW, APPROVE AND ENDORSE FINAL SITE SPECIFIC DESIGN.

FOR ADDITIONAL INFORMATION PLEASE CONTACT: FERGUSON WATERWORKS, 1-800-448-3636, www.ferguson.com

R-TANK^{HD} SPECIFICATION
WESTGATE AND HALL
BEAVERTON, OR

SCALE
NTS

DRAWN BY
JKB

DATE
04/07/2023

SHEET NO.
7 of 7

R-TANK® OPERATION, INSPECTION AND MAINTENANCE

Operation

Your R-Tank System has been designed to function in conjunction with the engineered drainage system on your site, the existing municipal infrastructure, and/or the existing soils and geography of the receiving watershed. Unless your site included certain unique and rare features, the operation of your R-Tank System will be driven by naturally occurring systems and will function autonomously. However, upholding a proper schedule of Inspection & Maintenance is critical to ensuring continued functionality and optimum performance of the system.

Inspection

Both the R-Tank and all stormwater pre-treatment features incorporated into your site must be inspected regularly. Inspections should be done every six months for the first year of operation, and at least yearly thereafter. Inspections may be required more frequently for pre-treatment systems. You should refer to the manufacturer requirements for the proper inspection schedule.

With the right equipment most inspections and measurements can be accomplished from the surface without physically entering any confined spaces. If your inspection does require confined space entry, you must follow all local, regional, and OSHA requirements.

All maintenance features of your system can be accessed through a covering at the surface. With the lid removed, you can visually inspect each component to identify sediment, trash, and other contaminants within the structure. Check you construction plans to identify the maintenance features engineered into your R-Tank system, which may include:

Upstream Pipes, Inlets, and Manholes

- Working from the structures adjacent the R-Tank toward those farther away, check for debris and sediment in both the structures and the pipes. Be sure to Include all structures that contain pre- treatment systems. Some structures may include a sump.

Maintenance Ports

- Located near the inlet and outlet connections and throughout the system, check sediment depth at each port.

Inspection Ports

- Less common, inspection ports are primarily located within the Treatment Row of an R-Tank System. These should be used to check for sediment deposits but are typically too small to access for backflushing.

Treatment Row

- On installations in 2018 or later, inlet pipes may connect to a row of modules with 12” diameter access holes running horizontally through the module that can be jet vacuumed. Check these rows for accumulation of sediment and debris.

All observations and measurements should be recorded on an Inspection Log kept on file. We’ve included a form you can use at the end of this guide.

Maintenance

For modules taller than 40” the R-Tank System should be back-flushed once sediment accumulation has reached 6”. For modules less than 40” tall, perform maintenance when sediment depths are greater than 15% of the total system height.

If your system includes a Treatment Row with linear access through the modules from the inlet pipe, backflush this area when sediment depths reach 6”.

BEFORE ANY MAINTENANCE IS PERFORMED ON YOUR SYSTEM -
PLUG THE OUTLET PIPE TO PREVENT CONTAMINATION OF THE DOWNSTREAM SYSTEMS.

Begin by cleaning all upstream structures, pipes, and pre-treatment systems containing sediment and/or debris. If your system includes a Treatment Row, this portion of the system should be cleaned with traditional jet-vac equipment. Add a centralizer to the jet for easiest access through the modules.

To back-flush the R-Tank, water is pumped into the system through the Maintenance Ports as rapidly as possible. The turbulent action of the water moving through the R-Tank will suspend sediments which may then be pumped out. If your system includes an Outlet Structure, this will be the ideal location to pump contaminated water out of the system. However, removal of back-flush water may be accomplished through the Maintenance Ports, as well.

For systems with large footprints that would require extensive volumes of water to properly flush the system, you should consider performing your maintenance within 24 hours of a rain event. Stormwater entering the system will aid in the suspension of sediments and reduce the volume of water required to properly flush the system.

STEP BY STEP INSTRUCTIONS FOR INSPECTION AND MAINTENANCE CAN BE FOUND ON THE NEXT PAGE, WITH A MAINTENANCE LOG ON THE LAST PAGE.

INSPECTION

1. Upstream Structures
    a. Remove cover
    b. Use flashlight to detect sediment deposits If present, measure sediment depth
    c. Inspect pipes connecting to R-Tank
        i. If inlet pipes connect to Treatment Row, check sediment depth within these modules
        ii. If access for measurement inside the Treatment Row is difficult, sediment depth can be estimated based on the coverage of the round, 12” opening of the module
    d. Inspect pre-treatment systems (if present)
    e. Record results on Maintenance Log
    f. Replace cover
    g. Repeat for ALL Manholes upstream of R-Tank until no sedimentation is observed and all pre- treatment systems have been checked
2. Maintenance Ports
    a. Remove cap
    b. Use flashlight to detect sediment deposits
    c. If present, measure sediment depth with stadia rod
    d. Record results on maintenance log
    e. Replace cap
    f. Repeat for ALL Maintenance Ports
3. Inspection Port
    a. Remove cap
    b. Use flashlight to detect sediment deposits
    c. If present, measure sediment depth with stadia rod
    d. Record results on Maintenance Log
    e. Replace cap

MAINTENANCE

1. Plug system outlet to prevent discharge of back-flush water
2. Vacuum all upstream structures, inlet pipes, and stormwater pre-treatment systems
3. If a Treatment Row is present, vacuum this row of modules
4. Determine best location to pump out back-flush water. Typically, the outlet structure will work best, but sometimes the Maintenance Ports must be used.
5. Remove cap from Maintenance Port and pump water as rapidly as possible into system through port to suspend sediments, pumping dirty water out of the system from the outlet or nearby Maintenance Port
6. Repeat at all Maintenance Ports until sediment levels are reduced to a satisfactory level
7. Sediment-laden water shall be disposed of per local regulations
8. Replace any remaining caps or covers and remove outlet plug
9. Record the back-flushing event in your Maintenance Log with any relevant specifics

FERGUSON WATERWORKS

R-Tank® Maintenance Log

Site Name:		Company:	
Location:		Contact:	
City and State:		Phone:	
System Owner:		Email:	

Date	Location	Sediment Depth	Observations / Notes	Initials