LAM RESEARCH EXPANSION PROJECT
BUILDINGS T, U AND X
NWC of SW Leveton Drive & SW 108th Avenue
City of Tualatin, Washington County, Oregon

Preliminary Stormwater Report

Project No. D3822800

Prepared for:

LAM RESEARCH CORPORATION
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Prepared by:

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2020 SW 4th Ave
Portland, OR 97201

June 6th, 2024

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APPENDICES

1.0 INTRODUCTION

1.1 PROJECT DESCRIPTION AND LOCATION

The project is located at the Lam Research Industrial Campus, approximately 1,500 feet northwest of the intersection of Leveton Drive and SW 108th Avenue (see exhibit 1 for Vicinity Map).

Lam Research is proposing to construct a new lab (Building X), office (Building T), CUB (Building U) buildings and Bulk Gas Yard at their existing campus along with new parking areas in the east and west side of the campus.

Stormwater drainage design and treatment are provided in accordance with the Clean Water Services (CWS) design standards (see Reference 1).

1.2 PURPOSE OF REPORT

The purpose of this report is to analyze the effects the proposed development will have on the existing stormwater conveyance system and to design the proposed stormwater system; and present the results of the hydraulic analysis.

2.0 EXISTING CONDITIONS

The existing LAM Research property is an operational advanced technology campus comprised of multiple building structures, parking lots and landscape areas. There are several stormwater facilities located in the southern portion of the property, existing detention ponds A, B, C and D.

An analysis of the USDA Web Soil Survey shows most of the soil categorized as Hydrologic Soil Group of B (see Appendix C for Soil Data).

3.0 PROPOSED CONDITIONS

The proposed improvements include a new Lab, office building and CUB and associated parking lots. The additional impervious area will be directed to new detention ponds and one modified existing pond (Pond B).

4.0 DESIGN CRITERIA

The basis of design for Stormwater Quality and Quantity as presented in the Clean Water Services (CWS) Design and Construction Standards for Sanitary Sewer and Surface Water Management, December 2019 is as follows:

Project Classification: Per CWS 2019 Section 4.03.2, unless specifically waived in writing by the District, a Hydromodification Assessment is required of all activities. Section 4.03.3 describes the Hydromodification Assessment Methodology. Risk Level is determined using the Hydromodification Planning Tool available on CWS's GIS website and following the discharge point ¼ mile downstream. The Development Class is also determined through the CWS's GIS website and identifying the project site location. Project Size Category is determined by calculating the area of proposed new and/or modified impervious surface.

- Risk Level: Low
- Development Class: Developed Area
- Project Size: Large
- Category 2

TABLE 4-2
HYDROMODIFICATION APPROACH PROJECT CATEGORY TABLE

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

Detention: Per CWS 2019 Section 4.08.06.b, facilities requiring hydromodification approach shall be designed such that the post-development runoff rates from the site do not exceed the predevelopment runoff rates in Table 4-6 of the CWS Design Standards:

TABLE 4-6

Post-Development Peak Runoff Rate	Pre-Development Peak Runoff Rate Target
2-year, 24-hour	2-year, 24-hour
10-year, 24-hour	10-year, 24-hour
25-year, 24-hour	25-year, 24-hour

Water Quality: Per CWS 2019, Section 4.04.1, owners of new develop and other activities which create or modify 1,000 square feet or greater of impervious surface or increase the amount of stormwater runoff or pollution leaving the site, are required to implement or fund permanent water quality approaches to reduce contaminants entering the storm and surface water system.

5.0 **ANALYSIS**

Methodology

The proposed improvements will be directed to proposed detention ponds (E, F and G) and one existing Pond B that has been modified. See Appendix A for the Onsite Drainage Map exhibit. For Calculations see Appendix B. These facilities are a combination water quality and water quantity detention pond adhering to the requirements set forth in CWS 2019 Section 4.09.2.

The rainfall rates used for calculations are presented in the table below:

TABLE 4-4

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

Water Quality

The water quality requirements and calculations are based on the CWS 2019 Design Standards Manual (see Appendix B for detailed calculations).

$$Water\ Quality\ Volume\ \ (cf) = \frac{0.36\ (in)\ \times\ Area\ (sf)}{12\ (\frac{in}{ft})}$$

$$Water\ Quality\ Flow\ \ (cfs) = \frac{Water\ Quality\ Volume\ (cf)}{14{,}400\ seconds}$$

$$Water\ Quality\ Flow\ \ (cfs) = \frac{0.36\ (in) \times Area\ (sf)}{12\ \left(\frac{in}{ft}\right) \times 4hr \times 60\,\frac{min}{hr} \times 60\,\frac{sec}{min}}$$

The water quality volume is provided in the detention pond with a ditch inlet set at the bottom of the pond. Within the inlet an orifice plate over the outlet; the orifice is sized according to limit the outflow during the water quality event to the flow determined by the Water Quality Flow equation (see Appendix B for Calculations).

Water Quantity

Hydromodification will be provided using peak flow matching from the Santa Barbara Unit Hydrograph (SBUH) methos using the Autocad Civil 3D Hydraflow Hydrographs extension for modeling. Detention will be combined with treatments within the proposed detention ponds andstorage volumes will occur above the water quality elevations for each pond.

The exisitng modified detention pond B will receive less area than before since a large portion where the new Lab, office and CUB buildngs will be constructed has been removed from that pond and re-directed to the proposed detention pond F.

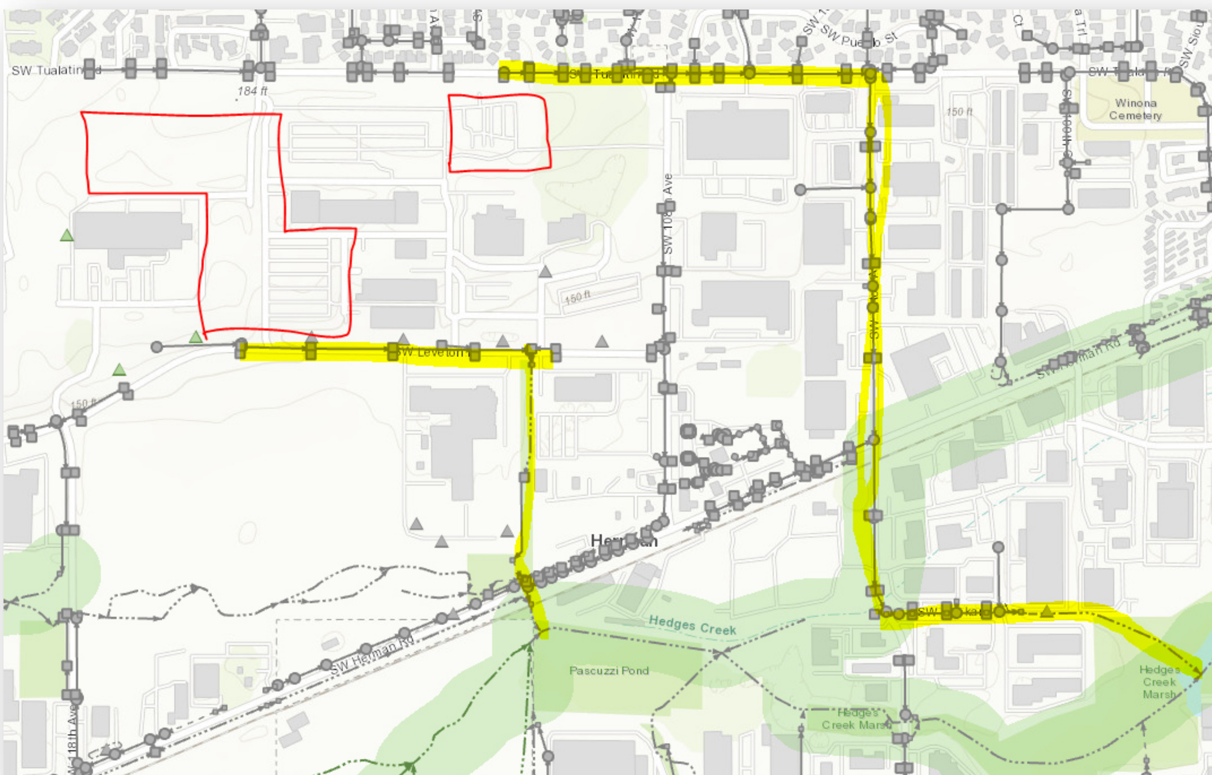
6.0 CONVEYANCE

Storm conveyance calculations for the proposed storm drain system will be sized to convey peak flows generated by the 25-year design storm event to size the pipes and the 10-year storm event for the inlets as presented in Chapter 5 of the CWS Design and Construction Standards.

7.0 DOWNSTREAM ANALYSIS

There are two existing storm drain systems along Leveton Drive and Tualatin Road both ultimately discharging into Hedges Creek classified as Low Risk Level for Hydromodifcation.

Increased runoff generated by the project will be managed by proposed detention ponds which were designed to accommodate and mitigate the post development peak flows to not exceed the existing rates. The available or existing capacity for the existing storm drain systems along Leveton Drive and Tualatin Road is currently unknown at this time and will need to be evaluated once more concrete information is obtained.



CWS Public Sanitary and Storm Sewers GIS Map

8.0 CONCLUSION

Based on compliance with the Clean Water Services (CWS) Design and Construction Standards for Sanitary Sewer and Surface Water Management, December 2019:

· Detention will mitigate the post-development peak flows to not surpass the pre-developed condition peak flows.

· Water quality provides treatment for the calculated water quality volume.

· Storm conveyance will be designed for a 25-year storm frequency using the Rational Method or the Santa Barbara Urban Hydrograph.

9.0 REFERENCES

1- Design and Construction Standards for Sanitary Sewer and Surface Water Management, Clean Water Services (CWS), December 2019.

2- Public Works Construction Code 2021, City of Tualatin, Oregon.

3- The City Charter & Municipal Code 2019, City of Tualatin, Oregon.

4- Storm Calculations – Novellus, Tualatin, Oregon, Revised March 6th, 2001.

APPENDIX A
EXHIBITS

Exhibit 1: Vicinity Map

Exhibit 2: Onsite Drainage Map

LEGEND

PROJECT SITE

N.T.S.

LAM Research Expansion Project Buildings T, U and X
NWC of SW Leveton Drive & SW 108th Avenue
Tualatin, Washington County, Oregon

EXHIBIT 1: VICINITY MAP

Jacobs

2020 S.W. 4th Avenue
Portland, Oregon 97201

03/Jun/24 11:15:48 AM

LAM RESEARCH EXPANSION PROJECT
BUILDINGS T, U and X
NWC of SW Leveton Drive & 108th Avenue
City of Tualatin, Washington County, Oregon

EXHIBIT 2: ONSITE DRAINAGE MAP

Jacobs

2020 S.W. 4th Avenue
Portland, Oregon 97201

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APPENDIX B
HYDRAULIC ANALYSIS AND RESULTS

WATER QUALITY CALCULATIONS

Drainage Area PK-1
Detention Pond-E

Impervious Area Summary

Existing Conditions

Existing Pervious Area:	7.286	Ac	*CN=69*
Existing Impervious Area:	0.560	Ac	*CN=98 & CN=75*
Total Drainage Area:	7.398	Ac	

Proposed Conditions

New Impervious Area:	6.120	Ac	*CN=98*
Modified Impervious Area:	0.154	Ac	*CN=75*
New Pervious Area:	0.082	Ac	*CN=69*
Exisiting Pervious Area:	1.042	Ac	*CN=69*
Total Drainage Area:	7.398	Ac	

Water Quality Treatment Area

Removed Impervious Area=	0.070	Ac	3,046	sf
New Impervious Area=	6.120	Ac		
Modified Impervious Area=	0.154	Ac		
Total Impervious Area=	6.274	Ac		

Area $_{WQ}$= New Impervious + 3(Modifed Impervious - Permanently Removed Imp)

Area $_{WQ}$= 6.372 Ac 277,574 sf

d. For all developments and re-development, other than single family and duplex, stormwater management approaches shall be sized based on the following:

1. Quality:

   All new impervious surfaces and three times the modified impervious surface, up to the total existing impervious surface on the site. The area requiring treatment is shown in the formula below:

   Area = New Impervious + 3(Modified Impervious)

   When modification results in the permanent removal of 1,000 square feet or greater of impervious surface, the treatment approach shall be sized for three times the replaced impervious surface, in addition to the new impervious surface. In this case, the area requiring treatment is shown in the formula below:

   Area = New Imp. + 3(Modified Imp. - Permanently Removed Imp.)

   Impervious areas shall be determined based upon building permits, construction plans, or other appropriate methods of measurement deemed reliable by District and/or City.

2. Quantity required for conveyance capacity or hydromodification: All new and modified impervious area created by the development.

Water Quality Volume

Water Quality Volume (WQV)
The WQV is the volume of water that is produced by the water quality storm. The WQV equals 0.36 inches over the impervious area that is required to be treated as shown in the formula below:

$$\text{Water Quality Volume (cu.ft.)} = \frac{0.36\ \text{(in.)}\ \times\ \text{Area (sq.ft.)}}{12\ \text{(in./ft.)}}$$

WQV= 8,327 cf

Water Quality Flow

Water Quality Flow (WQF)
The WQF is the average design flow anticipated from the water quality storm as shown in the formulas below:

$$\text{Water Quality Flow (cfs)} = \frac{\text{Water Quality Volume (cu.ft.)}}{14{,}400\ \text{seconds}}$$

$$\text{Water Quality Flow (cfs)} = \frac{0.36\ \text{(in.)}\ \times\ \text{Area (sq.ft.)}}{12\text{(in/ft)}(4\ \text{hr})(60\ \text{min/hr})(60\ \text{sec/min})}$$

WQF= 0.58 cfs

Water Quality Depth

Contour (ft)	Area (sf)	Incremental Storage (cf)	Total Storage (cf)
169	6,771	-	0
170	8,644	7,708	7,708
171	10,589	9,617	17,324
172	12,606	11,598	28,922
173	14,682	13,644	42,566
174	16,815	15,749	58,314

Storage Elevation=
170.06 ft

Water Quality Orifice Sizing

Water Quality Flow=	0.58	cfs
Water Quality Elevation=	170.06	ft
Water Quality Storage Depth=	1.06	ft
Water Quality Orifice Size=	1.12	in

$$D = 24\sqrt{\frac{Q}{C \cdot \pi \cdot \sqrt{2 \cdot g \cdot {}^{2}/_{3}\,H}}}$$

Orifice Size:
USE: D = 24 * [(Q/ (C[2gH]$^{0.5}$)) / π] $^{0.5}$
Where:
D (in) = diameter of orifice
Q(cfs) = WQV(cf) /(48*60*60)
C = 0.62
H(ft) = 2/3 x temporary detention height to centerline of orifice.

Q (cfs) =	WQV(cf)/(48*60*60)	0.048	cfs
WQ Drawdown Time=	48	hrs	
C=	0.62		
H (ft)=	2/3 x Temp. Detention Height to C/L of Orifice	2.0	

Drainage Area PK-3
Detention Pond-F

Impervious Area Summary

Existing Conditions

Existing Pervious Area:	4.894	Ac	*CN=69*
Existing Impervious Area:	2.894	Ac	*CN=98 & CN=75*
Total Drainage Area:	7.788	Ac	

Proposed Conditions

New Impervious Area:	4.028	Ac	*CN=98*
Modified New Impervious Area:	2.804	Ac	*CN=75*
New Pervious Area:	0.088	Ac	*CN=69*
Exisiting Pervious Area:	0.868	Ac	*CN=69*
Total Drainage Area:	7.788	Ac	

Water Quality Treatment Area

Removed Impervious Area=	0.088	Ac	3,849 sf
New Impervious Area=	4.028	Ac	
Modified Impervious Area=	2.804	Ac	
Total Impervious Area=	6.832	Ac	

Area $_{WQ}$= New Impervious + 3(Modifed Impervious - Permanently Removed Imp)

Area $_{WQ}$= 12.175 Ac 530,339 sf

d. For all developments and re-development, other than single family and duplex, stormwater management approaches shall be sized based on the following:

1. Quality:

   All new impervious surfaces and three times the modified impervious surface, up to the total existing impervious surface on the site. The area requiring treatment is shown in the formula below:

   Area = New Impervious + 3(Modified Impervious)

   When modification results in the permanent removal of 1,000 square feet or greater of impervious surface, the treatment approach shall be sized for three times the replaced impervious surface, in addition to the new impervious surface. In this case, the area requiring treatment is shown in the formula below:

   Area = New Imp. + 3(Modified Imp. - Permanently Removed Imp.)

   Impervious areas shall be determined based upon building permits, construction plans, or other appropriate methods of measurement deemed reliable by District and/or City.

2. Quantity required for conveyance capacity or hydromodification: All new and modified impervious area created by the development.

Water Quality Volume

Water Quality Volume (WQV)
The WQV is the volume of water that is produced by the water quality storm. The WQV equals 0.36 inches over the impervious area that is required to be treated as shown in the formula below:

$$\text{Water Quality Volume (cu.ft.)} = \frac{0.36\text{ (in.) x Area (sq.ft.)}}{12\text{ (in./ft.)}}$$

WQV= 15,910 cf

Water Quality Flow

Water Quality Flow (WQF)
The WQF is the average design flow anticipated from the water quality storm as shown in the formulas below:

$$\text{Water Quality Flow (cfs)} = \frac{\text{Water Quality Volume (cu.ft.)}}{14{,}400\text{ seconds}}$$

$$\text{Water Quality Flow (cfs)} = \frac{0.36\text{ (in.) x Area (sq.ft.)}}{12\text{(in/ft)(4 hr)(60 min/hr)(60 sec/min)}}$$

WQF= 1.10 cfs

Water Quality Depth

Contour (ft)	Area (sf)	Incremental Storage (cf)	Total Storage (cf)
148.5	8,622	-	0
149.5	9,721	9,172	9,172
150.5	10,893	10,307	19,479
151.5	12,137	11,515	30,994
152.5	13,454	12,796	43,789
153.5	14,843	14,149	57,938

Storage Elevation=
150.15 ft

Water Quality Orifice Sizing

Water Quality Flow= 1.10 cfs
Water Quality Elevation= 150.15 ft
Water Quality Storage Depth= 1.65 ft

Water Quality Orifice Size= 1.54 in

$$D = 24\sqrt{\frac{Q}{C \cdot \pi \cdot \sqrt{2 \cdot g \cdot {}^{2}/_{3} H}}}$$

Orifice Size:
USE: D = 24 * [(Q/ (C[2gH]$^{0.5}$)) / π] $^{0.5}$
Where:
D (in) = diameter of orifice
Q(cfs) = WQV(cf) /(48*60*60)
C = 0.62
H(ft) = 2/3 x temporary detention height to centerline of orifice.

Q (cfs) = WQV(cf)/(48*60*60) 0.092 cfs
WQ Drawdown Time= 48 hrs
C= 0.62
H (ft)= 2/3 x Temp. Detention Height to C/L of Orifice 2.0

Drainage Area PK-2
Detention Pond-G

Impervious Area Summary

Existing Conditions

Existing Pervious Area:	1.271	Ac	*CN=69*
Existing Impervious Area:	1.954	Ac	*CN=98 & CN=75*
Total Drainage Area:	3.225	Ac	

Proposed Conditions

New Impervious Area:	0.849	Ac	*CN=98*
Modified Impervious Area:	1.745	Ac	*CN=75*
New Pervious Area:	0.166	Ac	*CN=69*
Exisiting Pervious Area:	0.465	Ac	*CN=69*
Total Drainage Area:	3.225	Ac	

Water Quality Treatment Area

Removed Impervious Area=	0.166	Ac	7,213 sf
New Impervious Area=	0.849	Ac	
Modified Impervious Area=	1.745	Ac	
Total Impervious Area=	2.594	Ac	

Area $_{WQ}$= New Impervious + 3(Modifed Impervious - Permanently Removed Imp)

Area $_{WQ}$= 5.587 Ac 243,380 sf

d. For all developments and re-development, other than single family and duplex, stormwater management approaches shall be sized based on the following:

1. Quality:

   All new impervious surfaces and three times the modified impervious surface, up to the total existing impervious surface on the site. The area requiring treatment is shown in the formula below:

   Area = New Impervious + 3(Modified Impervious)

   When modification results in the permanent removal of 1,000 square feet or greater of impervious surface, the treatment approach shall be sized for three times the replaced impervious surface, in addition to the new impervious surface. In this case, the area requiring treatment is shown in the formula below:

   Area = New Imp. + 3(Modified Imp. - Permanently Removed Imp.)

   Impervious areas shall be determined based upon building permits, construction plans, or other appropriate methods of measurement deemed reliable by District and/or City.

2. Quantity required for conveyance capacity or hydromodification: All new and modified impervious area created by the development.

Water Quality Volume

Water Quality Volume (WQV)
The WQV is the volume of water that is produced by the water quality storm. The WQV equals 0.36 inches over the impervious area that is required to be treated as shown in the formula below:

$$\text{Water Quality Volume (cu.ft.)} = \frac{0.36\ \text{(in.)} \times \text{Area (sq.ft.)}}{12\ \text{(in./ft.)}}$$

WQV= 7,301 cf

Water Quality Flow

Water Quality Flow (WQF)
The WQF is the average design flow anticipated from the water quality storm as shown in the formulas below:

$$\text{Water Quality Flow (cfs)} = \frac{\text{Water Quality Volume (cu.ft.)}}{14{,}400\ \text{seconds}}$$

$$\text{Water Quality Flow (cfs)} = \frac{0.36\ \text{(in.)} \times \text{Area (sq.ft.)}}{12\text{(in/ft)}(4\ \text{hr})(60\ \text{min/hr})(60\ \text{sec/min})}$$

WQF= 0.51 cfs

Water Quality Depth

Contour (ft)	Area (sf)	Incremental Storage (cf)	Total Storage (cf)
173	5,525	-	0
174	6,573	6,049	6,049
175	7,678	7,126	13,175
176	8,840	8,259	21,434
177	10,058	9,449	30,883

Storage Elevation= 174.21 ft

Water Quality Orifice Sizing

Water Quality Flow=	0.51	cfs
Water Quality Elevation=	174.21	ft
Water Quality Storage Depth=	1.21	ft
Water Quality Orifice Size=	1.05	in

$$D = 24\sqrt{\frac{Q}{C \cdot \pi \cdot \sqrt{2 \cdot g \cdot {}^{2}/_{3} H}}}$$

Orifice Size:
USE: $D = 24 * [(Q/ (C[2gH]^{0.5})) / \pi]^{0.5}$
Where:
D (in) = diameter of orifice
Q(cfs) = WQV(cf) /(48*60*60)
C = 0.62
H(ft) = 2/3 x temporary detention height to centerline of orifice.

Q (cfs) = WQV(cf)/(48*60*60) 0.042 cfs
WQ Drawdown Time= 48 hrs
C= 0.62
H (ft)= 2/3 x Temp. Detention Height to C/L of Orifice 2.0

WATER QUANTITY CALCULATIONS

DETENTION POND E

Hydrograph Report

Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2021 Friday, 06 / 28 / 2024

Hyd. No. 3

Pre_Developed_PK1

Hydrograph type	= SBUH Runoff	Peak discharge	= 0.209 cfs
Storm frequency	= 2 yrs	Time to peak	= 8.80 hrs
Time interval	= 2 min	Hyd. volume	= 11,289 cuft
Drainage area	= 7.390 ac	Curve number	= 69*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 7.00 min
Total precip.	= 2.50 in	Distribution	= Type IA
Storm duration	= 24 hrs	Shape factor	= n/a

* Composite (Area/CN) = [(0.080 x 98) + (0.030 x 75) + (7.285 x 69)] / 7.390

Hydrograph Report

Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2021 Friday, 06 / 28 / 2024

Hyd. No. 3

Pre_Developed_PK1

Hydrograph type	= SBUH Runoff	Peak discharge	= 1.046 cfs
Storm frequency	= 10 yrs	Time to peak	= 8.00 hrs
Time interval	= 2 min	Hyd. volume	= 24,791 cuft
Drainage area	= 7.390 ac	Curve number	= 69*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 7.00 min
Total precip.	= 3.45 in	Distribution	= Type IA
Storm duration	= 24 hrs	Shape factor	= n/a

* Composite (Area/CN) = [(0.080 x 98) + (0.030 x 75) + (7.285 x 69)] / 7.390

Hydrograph Report

Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2021 Friday, 06 / 28 / 2024

Hyd. No. 3

Pre_Developed_PK1

Hydrograph type	= SBUH Runoff	Peak discharge	= 1.553 cfs
Storm frequency	= 25 yrs	Time to peak	= 8.00 hrs
Time interval	= 2 min	Hyd. volume	= 32,247 cuft
Drainage area	= 7.390 ac	Curve number	= 69*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 7.00 min
Total precip.	= 3.90 in	Distribution	= Type IA
Storm duration	= 24 hrs	Shape factor	= n/a

* Composite (Area/CN) = [(0.080 x 98) + (0.030 x 75) + (7.285 x 69)] / 7.390

Hydrograph Report

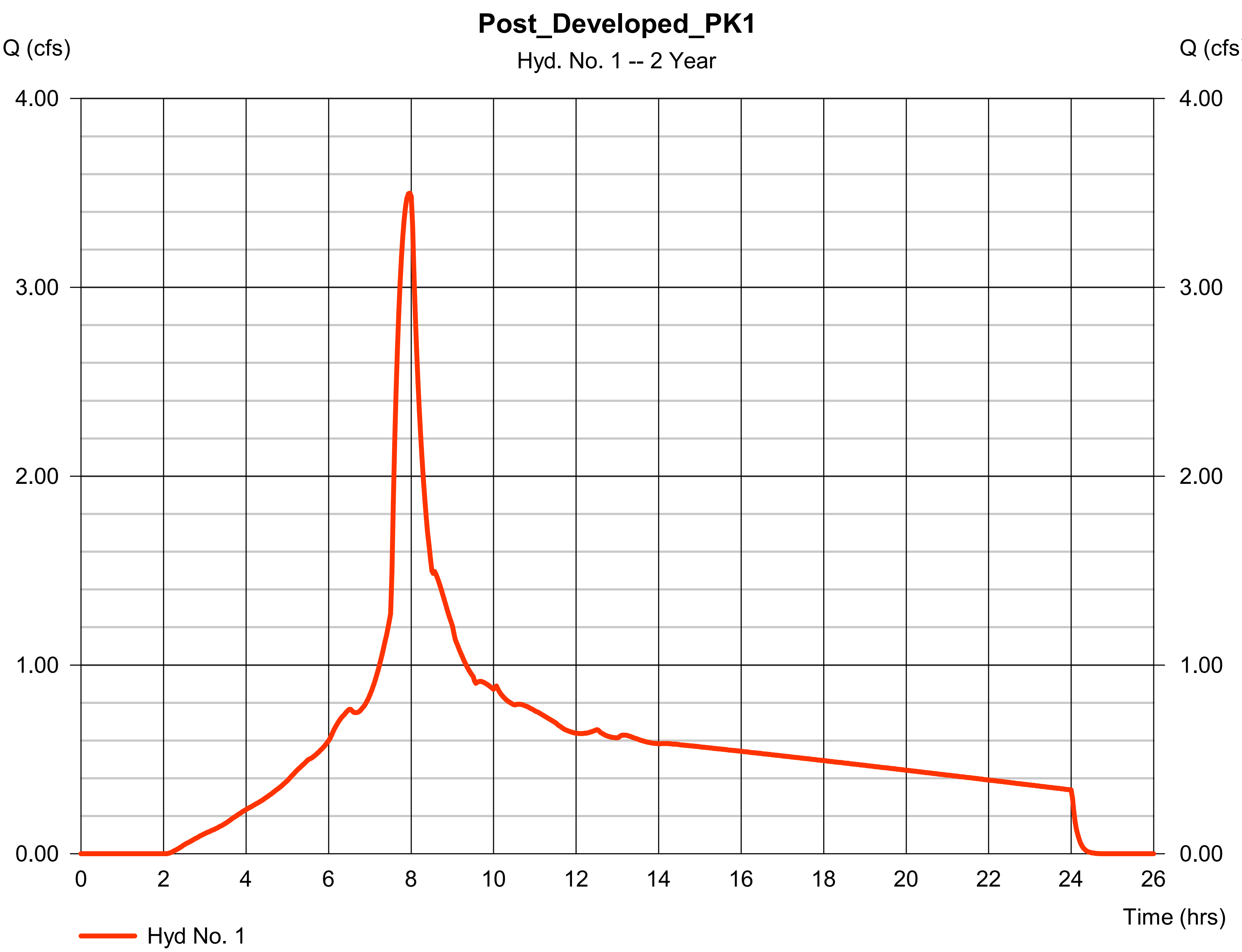
Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2021 Friday, 06 / 28 / 2024

Hyd. No. 1

Post_Developed_PK1

Hydrograph type	= SBUH Runoff	Peak discharge	= 3.499 cfs
Storm frequency	= 2 yrs	Time to peak	= 7.97 hrs
Time interval	= 2 min	Hyd. volume	= 50,147 cuft
Drainage area	= 7.390 ac	Curve number	= 94*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 7.00 min
Total precip.	= 2.50 in	Distribution	= Type IA
Storm duration	= 24 hrs	Shape factor	= n/a

* Composite (Area/CN) = [(6.270 x 98) + (1.120 x 69)] / 7.390



Hydrograph Report

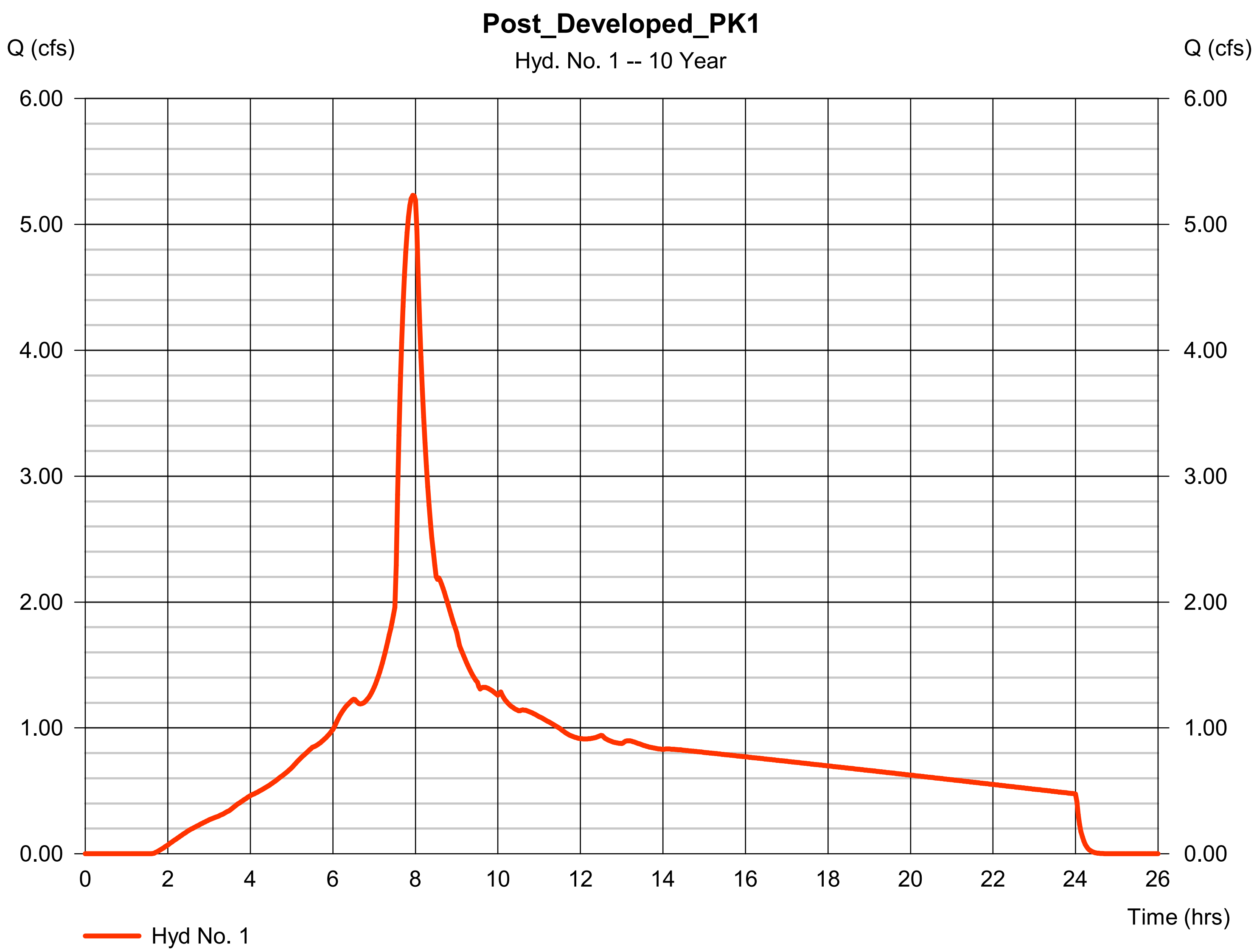
Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2021 Friday, 06 / 28 / 2024

Hyd. No. 1

Post_Developed_PK1

Hydrograph type	= SBUH Runoff	Peak discharge	= 5.231 cfs
Storm frequency	= 10 yrs	Time to peak	= 7.93 hrs
Time interval	= 2 min	Hyd. volume	= 74,761 cuft
Drainage area	= 7.390 ac	Curve number	= 94*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 7.00 min
Total precip.	= 3.45 in	Distribution	= Type IA
Storm duration	= 24 hrs	Shape factor	= n/a

* Composite (Area/CN) = [(6.270 x 98) + (1.120 x 69)] / 7.390



Hydrograph Report

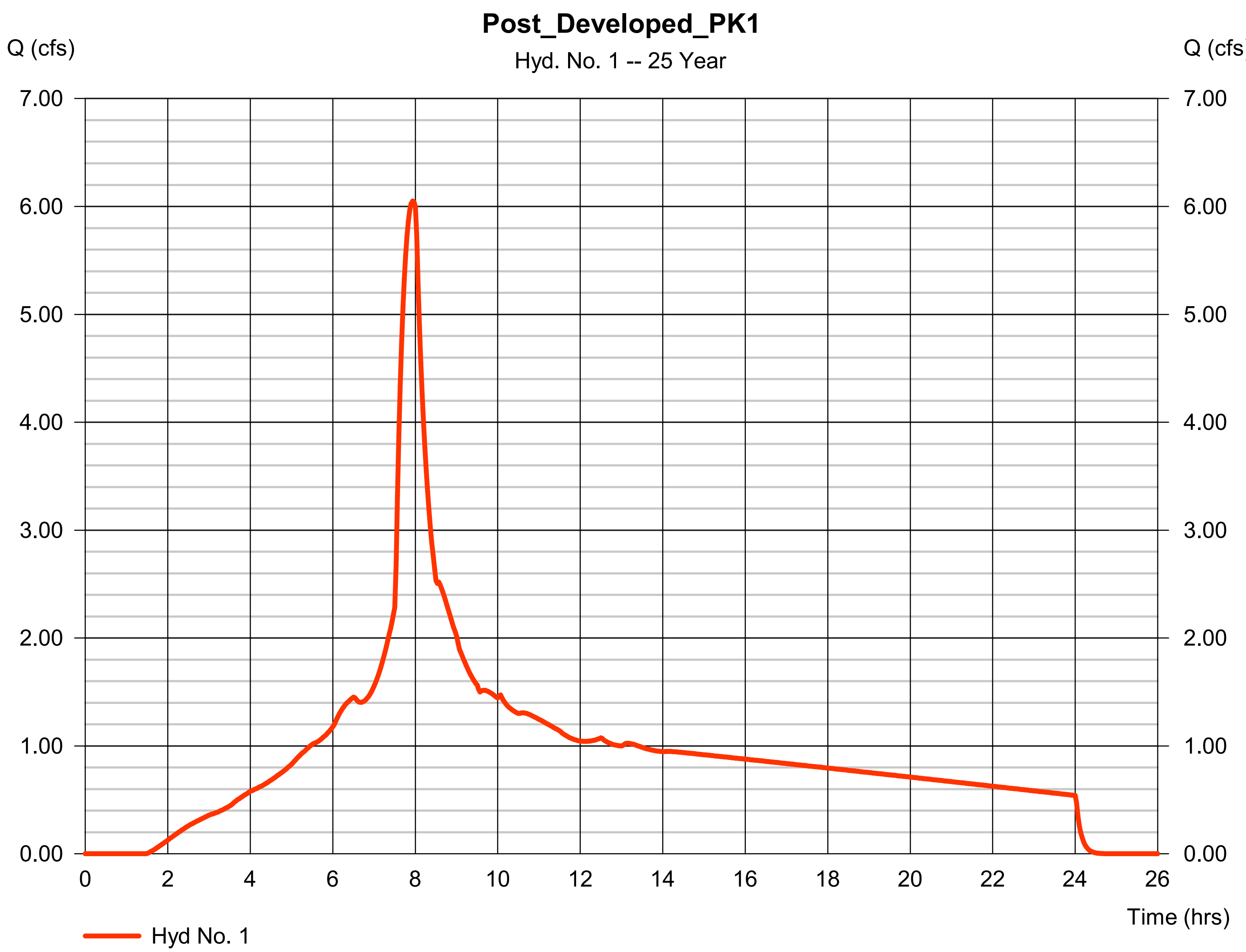
Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2021 Friday, 06 / 28 / 2024

Hyd. No. 1

Post_Developed_PK1

Hydrograph type	= SBUH Runoff	Peak discharge	= 6.050 cfs
Storm frequency	= 25 yrs	Time to peak	= 7.93 hrs
Time interval	= 2 min	Hyd. volume	= 86,551 cuft
Drainage area	= 7.390 ac	Curve number	= 94*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 7.00 min
Total precip.	= 3.90 in	Distribution	= Type IA
Storm duration	= 24 hrs	Shape factor	= n/a

* Composite (Area/CN) = [(6.270 x 98) + (1.120 x 69)] / 7.390



Hydrograph Report

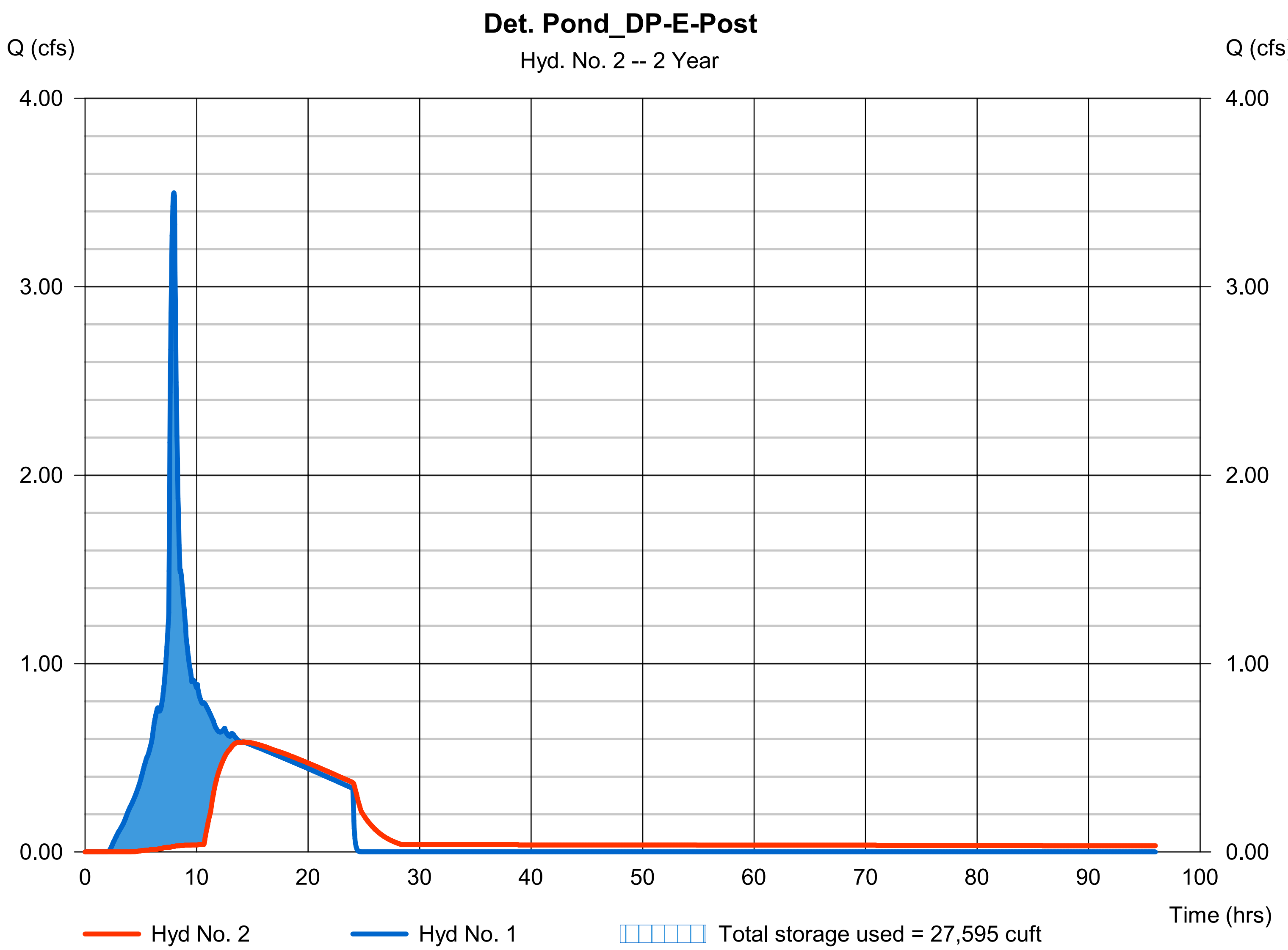
Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2021 Friday, 06 / 28 / 2024

Hyd. No. 2

Det. Pond_DP-E-Post

Hydrograph type	= Reservoir	Peak discharge	= 0.583 cfs
Storm frequency	= 2 yrs	Time to peak	= 14.23 hrs
Time interval	= 2 min	Hyd. volume	= 34,243 cuft
Inflow hyd. No.	= 1 - Post_Developed_PK1	Max. Elevation	= 172.71 ft
Reservoir name	= Detention Pond-E	Max. Storage	= 27,595 cuft

Storage Indication method used.



Hydrograph Report

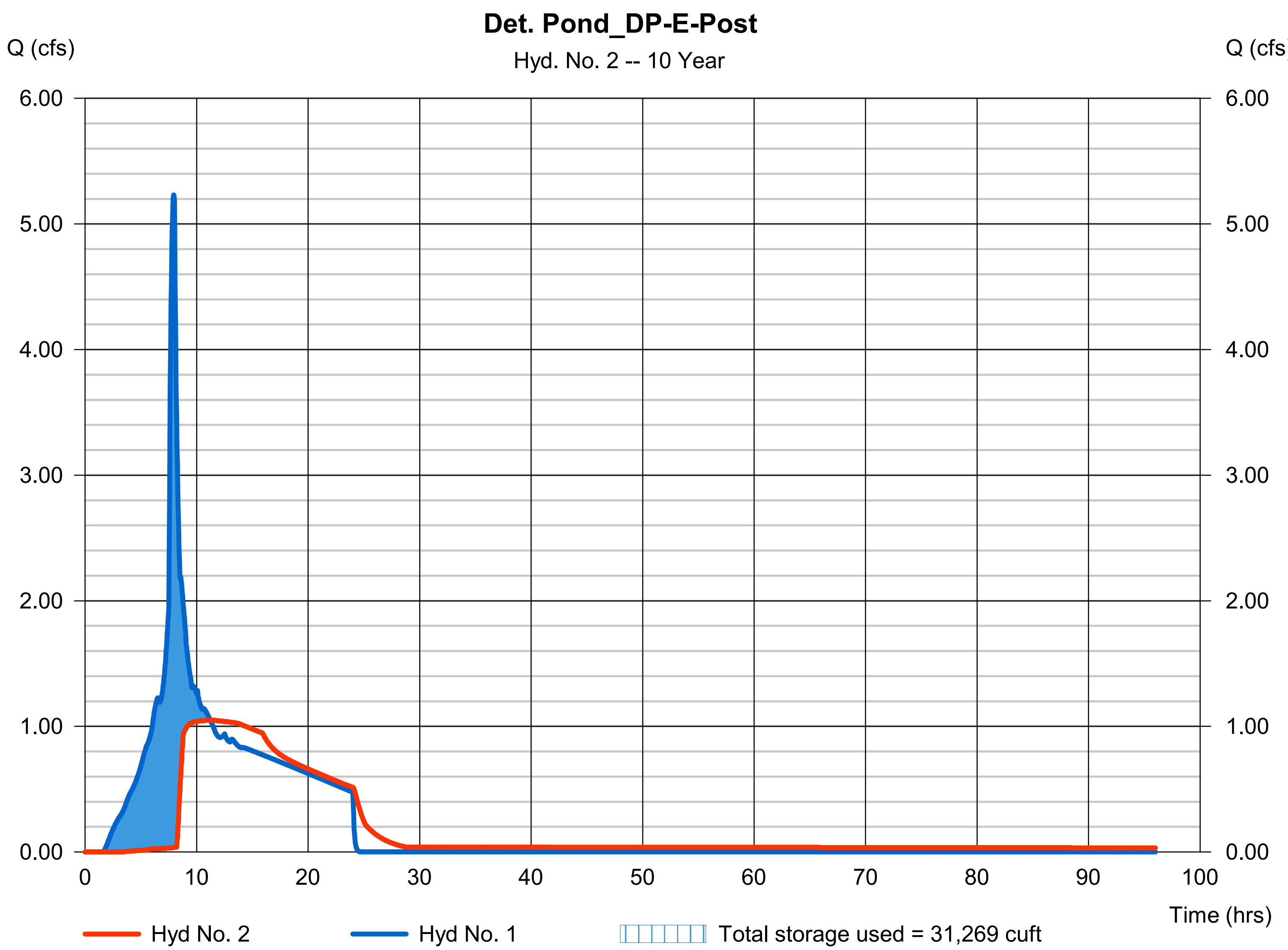
Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2021 Friday, 06 / 28 / 2024

Hyd. No. 2

Det. Pond_DP-E-Post

Hydrograph type	= Reservoir	Peak discharge	= 1.049 cfs
Storm frequency	= 10 yrs	Time to peak	= 11.23 hrs
Time interval	= 2 min	Hyd. volume	= 58,810 cuft
Inflow hyd. No.	= 1 - Post_Developed_PK1	Max. Elevation	= 172.98 ft
Reservoir name	= Detention Pond-E	Max. Storage	= 31,269 cuft

Storage Indication method used.



Hydrograph Report

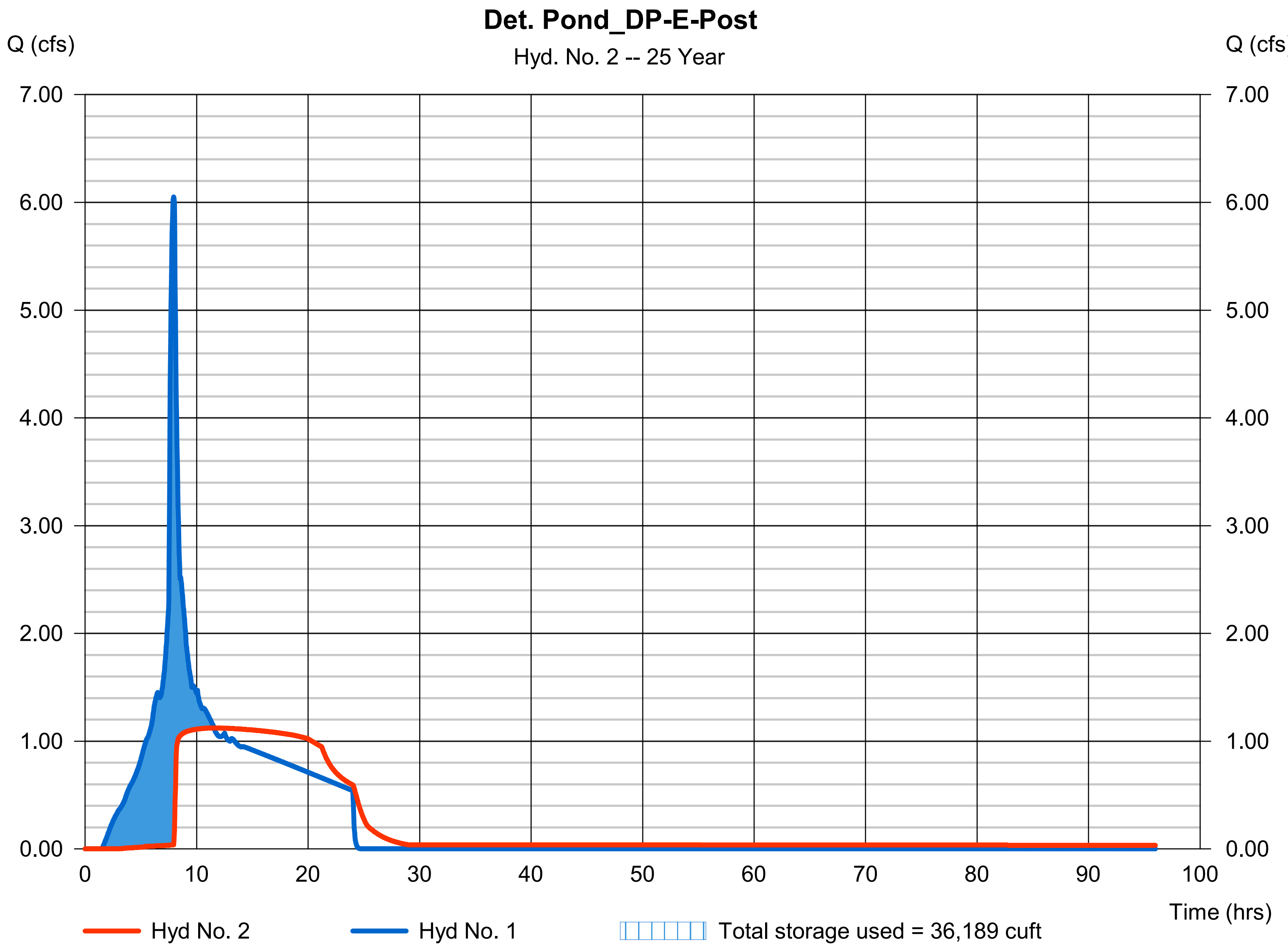
Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2021 Friday, 06 / 28 / 2024

Hyd. No. 2

Det. Pond_DP-E-Post

Hydrograph type	= Reservoir	Peak discharge	= 1.123 cfs
Storm frequency	= 25 yrs	Time to peak	= 11.57 hrs
Time interval	= 2 min	Hyd. volume	= 70,583 cuft
Inflow hyd. No.	= 1 - Post_Developed_PK1	Max. Elevation	= 173.30 ft
Reservoir name	= Detention Pond-E	Max. Storage	= 36,189 cuft

Storage Indication method used.



DETENTION POND F

Hydrograph Report

Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2021 | Wednesday, 06 / 5 / 2024

Hyd. No. 3

Pre_Developed_PK3

Hydrograph type	= SBUH Runoff	Peak discharge	= 0.446 cfs
Storm frequency	= 2 yrs	Time to peak	= 8.03 hrs
Time interval	= 2 min	Hyd. volume	= 14,948 cuft
Drainage area	= 7.790 ac	Curve number	= 72*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 7.50 min
Total precip.	= 2.50 in	Distribution	= Type IA
Storm duration	= 24 hrs	Shape factor	= n/a

* Composite (Area/CN) = [(0.090 x 98) + (2.804 x 75) + (4.894 x 69)] / 7.790

Hydrograph Report

Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2021 Wednesday, 06 / 5 / 2024

Hyd. No. 3

Pre_Developed_PK3

Hydrograph type	= SBUH Runoff	Peak discharge	= 1.495 cfs
Storm frequency	= 10 yrs	Time to peak	= 8.00 hrs
Time interval	= 2 min	Hyd. volume	= 30,776 cuft
Drainage area	= 7.790 ac	Curve number	= 72*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 7.50 min
Total precip.	= 3.45 in	Distribution	= Type IA
Storm duration	= 24 hrs	Shape factor	= n/a

* Composite (Area/CN) = [(0.090 x 98) + (2.804 x 75) + (4.894 x 69)] / 7.790

Hydrograph Report

Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2021 — Wednesday, 06 / 5 / 2024

Hyd. No. 3

Pre_Developed_PK3

Hydrograph type	= SBUH Runoff	Peak discharge	= 2.085 cfs
Storm frequency	= 25 yrs	Time to peak	= 8.00 hrs
Time interval	= 2 min	Hyd. volume	= 39,317 cuft
Drainage area	= 7.790 ac	Curve number	= 72*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 7.50 min
Total precip.	= 3.90 in	Distribution	= Type IA
Storm duration	= 24 hrs	Shape factor	= n/a

* Composite (Area/CN) = [(0.090 x 98) + (2.804 x 75) + (4.894 x 69)] / 7.790

Hydrograph Report

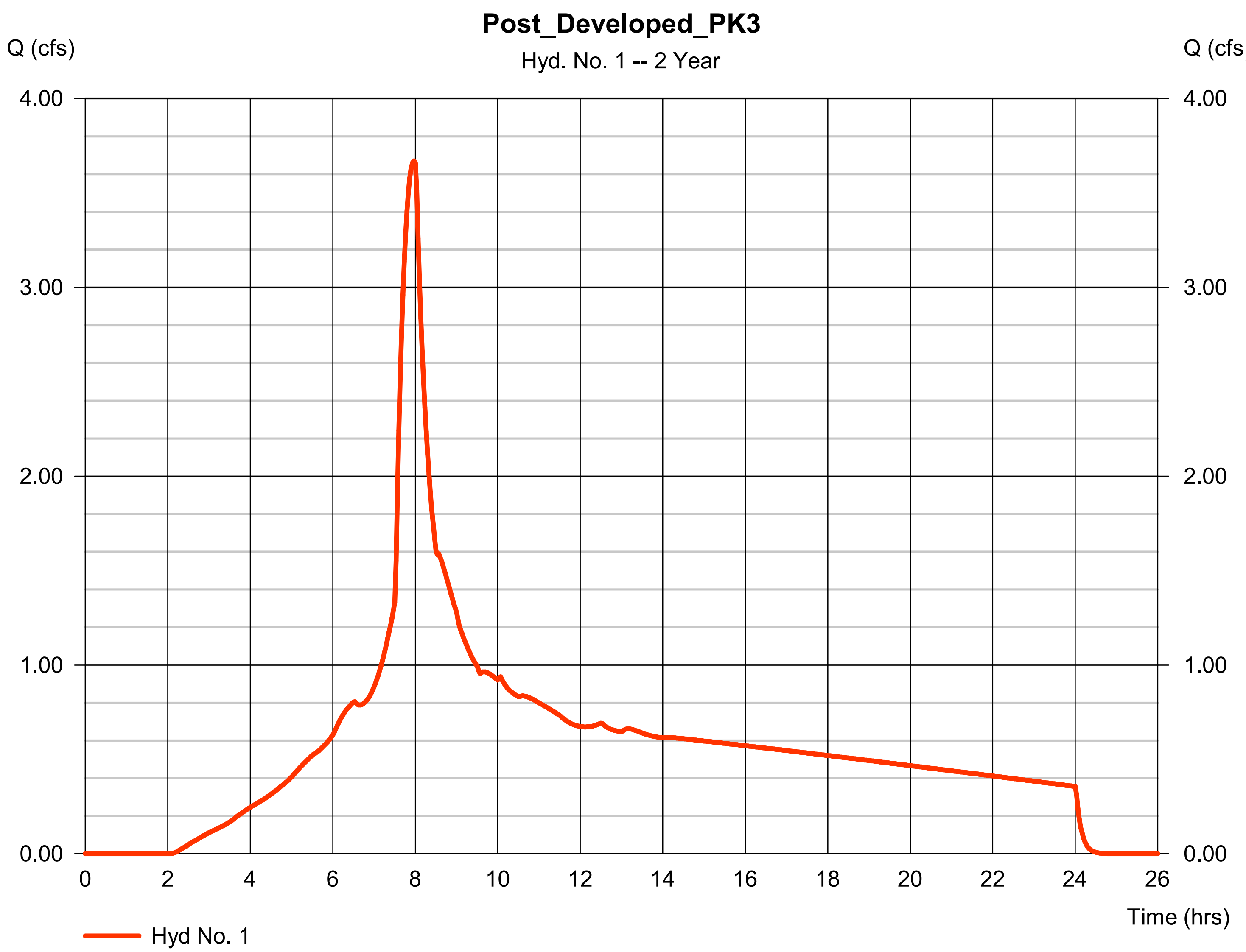
Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2021 Wednesday, 06 / 5 / 2024

Hyd. No. 1

Post_Developed_PK3

Hydrograph type	= SBUH Runoff	Peak discharge	= 3.670 cfs
Storm frequency	= 2 yrs	Time to peak	= 7.97 hrs
Time interval	= 2 min	Hyd. volume	= 52,861 cuft
Drainage area	= 7.790 ac	Curve number	= 94*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 7.50 min
Total precip.	= 2.50 in	Distribution	= Type IA
Storm duration	= 24 hrs	Shape factor	= n/a

* Composite (Area/CN) = [(6.832 x 98) + (0.956 x 69)] / 7.790



Hydrograph Report

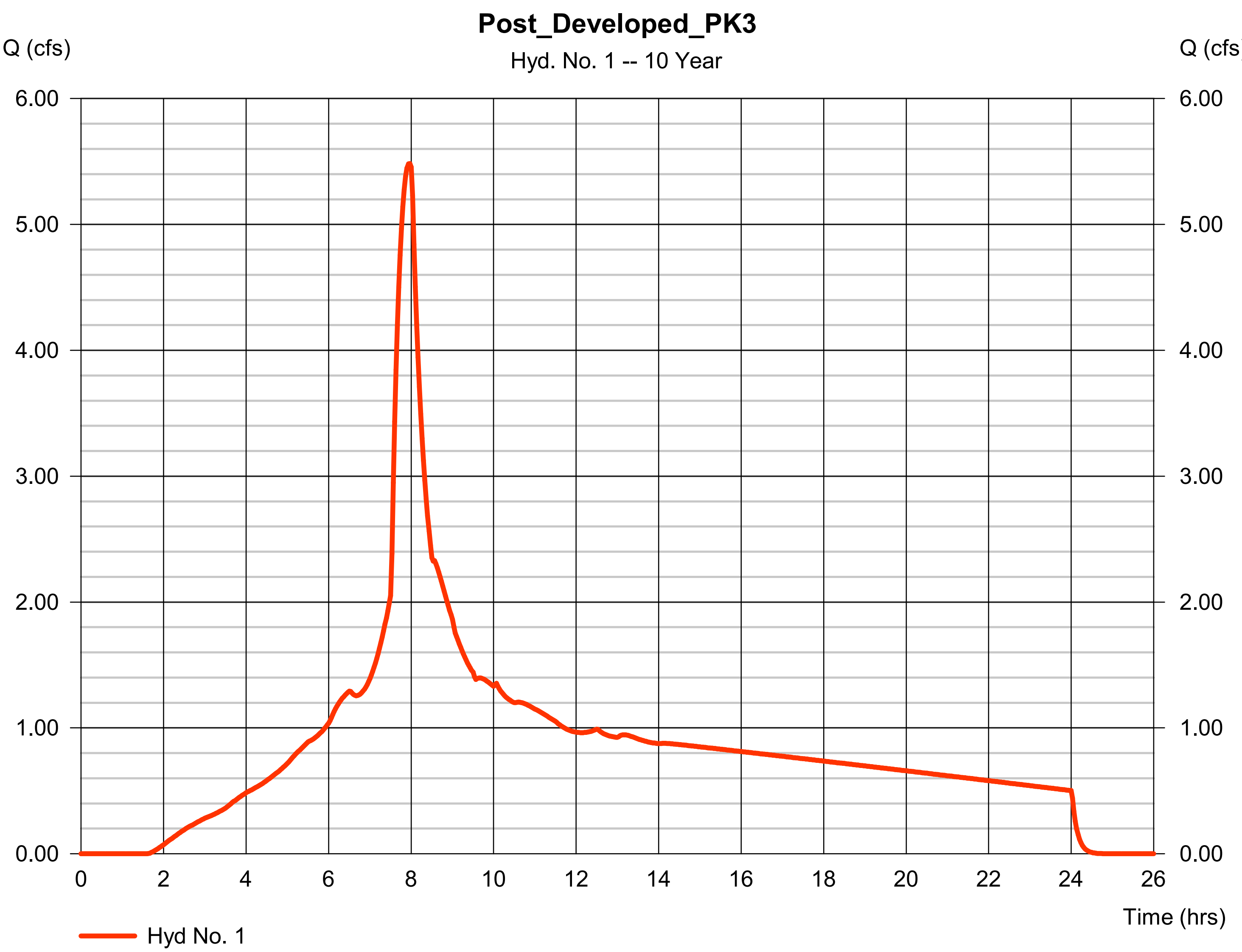
Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2021 — Wednesday, 06 / 5 / 2024

Hyd. No. 1

Post_Developed_PK3

Hydrograph type	= SBUH Runoff	Peak discharge	= 5.484 cfs
Storm frequency	= 10 yrs	Time to peak	= 7.97 hrs
Time interval	= 2 min	Hyd. volume	= 78,807 cuft
Drainage area	= 7.790 ac	Curve number	= 94*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 7.50 min
Total precip.	= 3.45 in	Distribution	= Type IA
Storm duration	= 24 hrs	Shape factor	= n/a

* Composite (Area/CN) = [(6.832 x 98) + (0.956 x 69)] / 7.790



Hydrograph Report

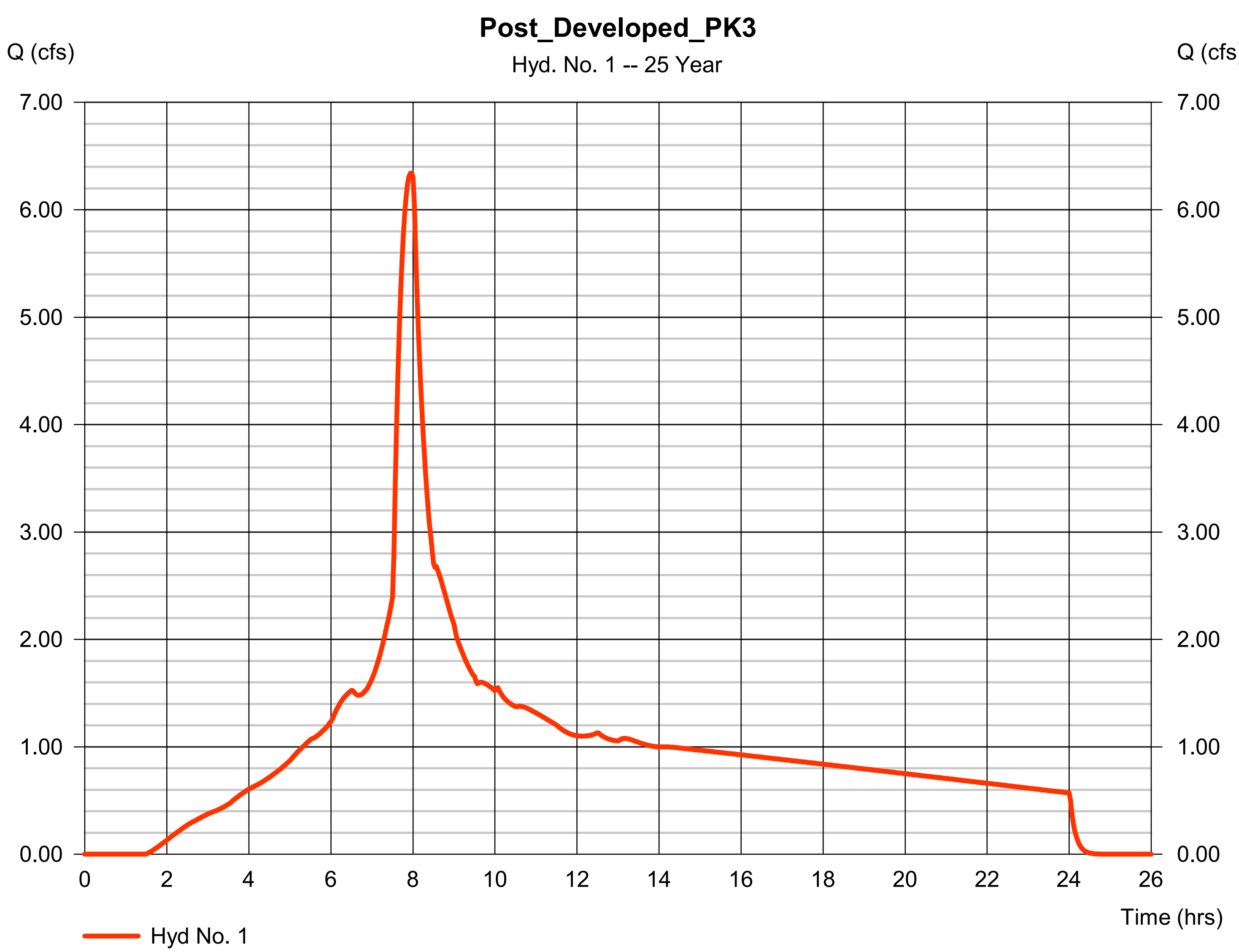
Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2021 Wednesday, 06 / 5 / 2024

Hyd. No. 1

Post_Developed_PK3

Hydrograph type	= SBUH Runoff	Peak discharge	= 6.340 cfs
Storm frequency	= 25 yrs	Time to peak	= 7.93 hrs
Time interval	= 2 min	Hyd. volume	= 91,236 cuft
Drainage area	= 7.790 ac	Curve number	= 94*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 7.50 min
Total precip.	= 3.90 in	Distribution	= Type IA
Storm duration	= 24 hrs	Shape factor	= n/a

* Composite (Area/CN) = [(6.832 x 98) + (0.956 x 69)] / 7.790



Hydrograph Report

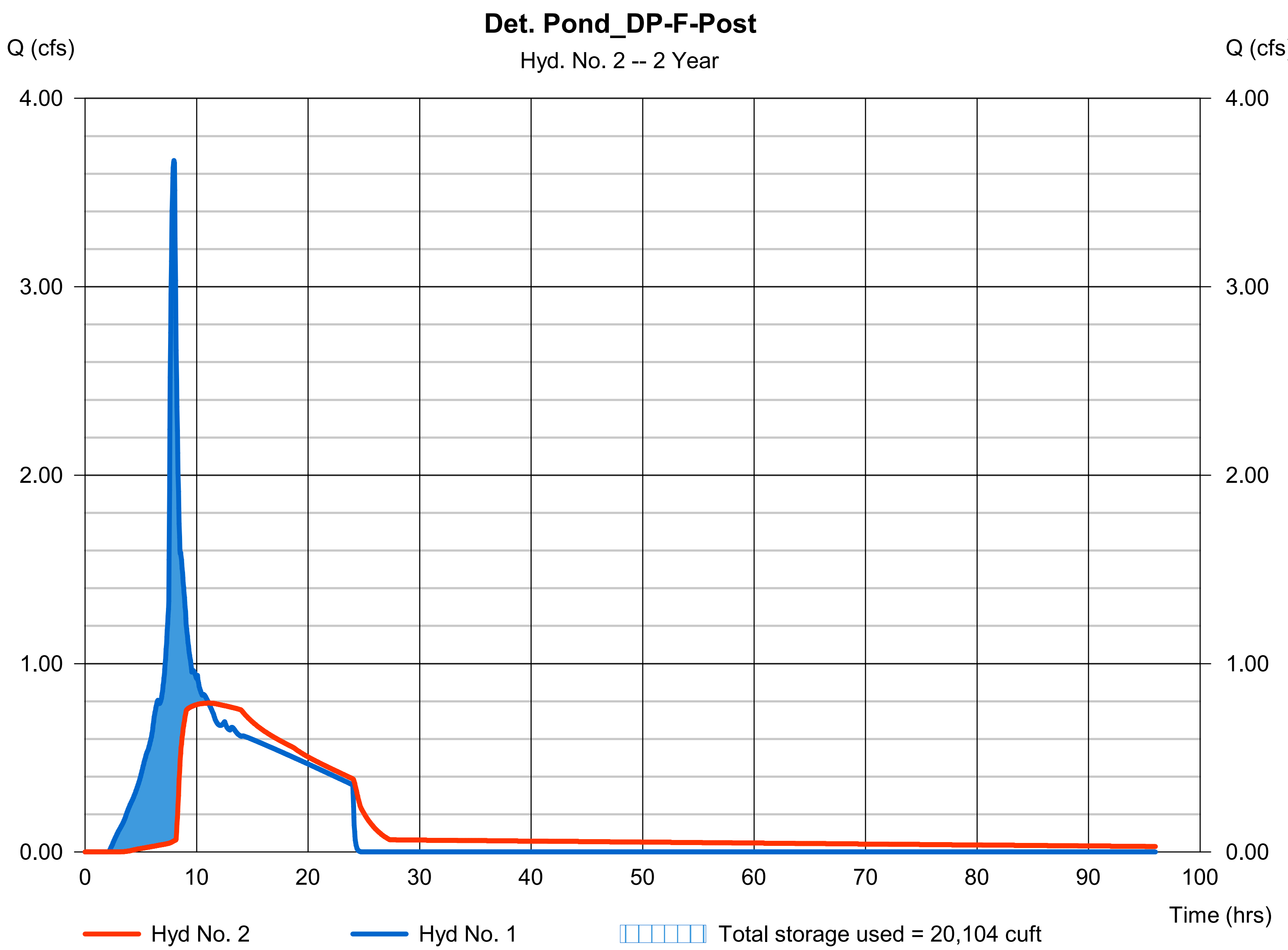
Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2021 Wednesday, 06 / 5 / 2024

Hyd. No. 2

Det. Pond_DP-F-Post

Hydrograph type	= Reservoir	Peak discharge	= 0.790 cfs
Storm frequency	= 2 yrs	Time to peak	= 11.07 hrs
Time interval	= 2 min	Hyd. volume	= 49,156 cuft
Inflow hyd. No.	= 1 - Post_Developed_PK3	Max. Elevation	= 151.88 ft
Reservoir name	= Detention Pond-F	Max. Storage	= 20,104 cuft

Storage Indication method used.



Hydrograph Report

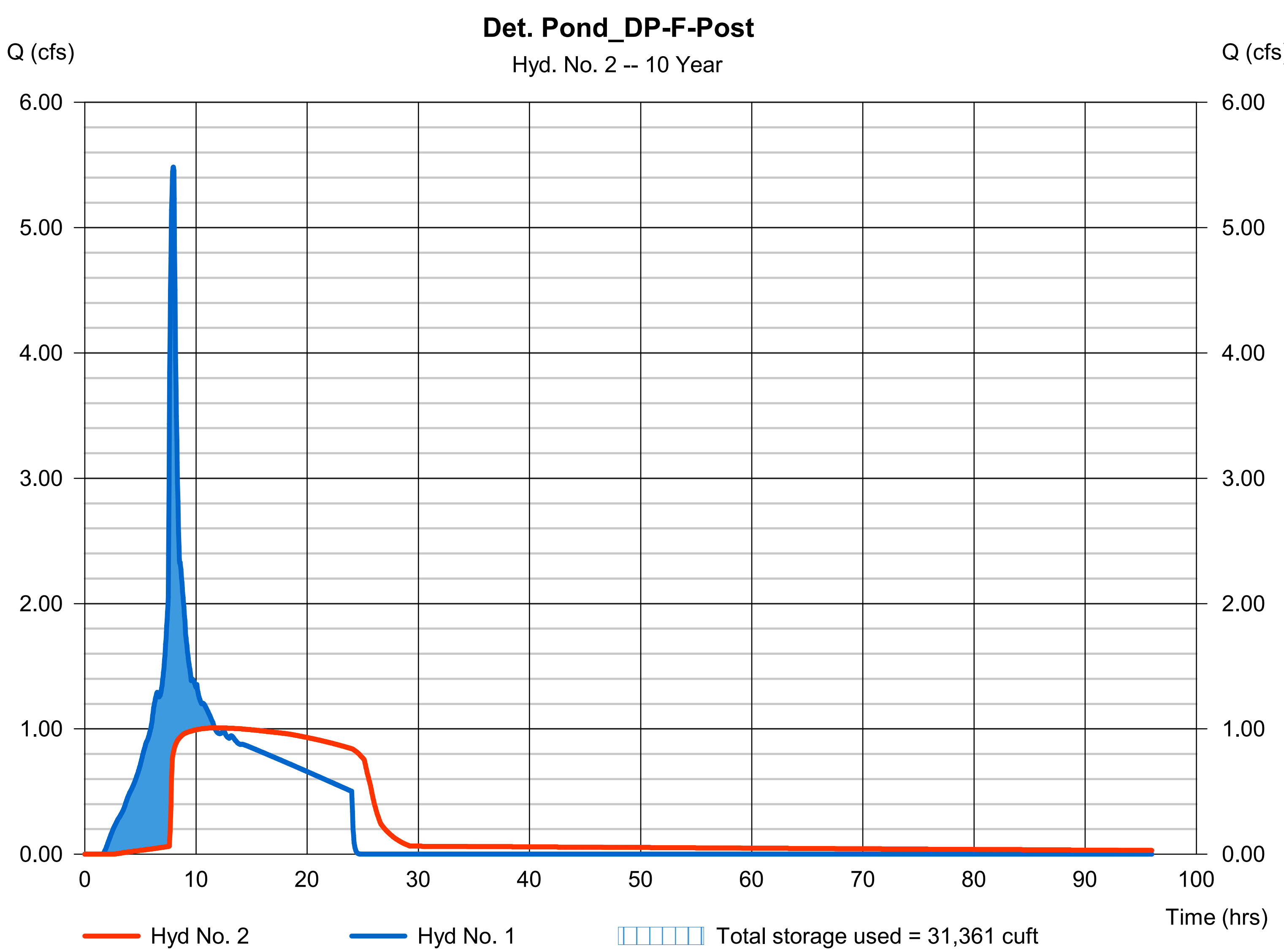
Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2021 — Wednesday, 06 / 5 / 2024

Hyd. No. 2

Det. Pond_DP-F-Post

Hydrograph type	= Reservoir	Peak discharge	= 1.008 cfs
Storm frequency	= 10 yrs	Time to peak	= 11.67 hrs
Time interval	= 2 min	Hyd. volume	= 74,893 cuft
Inflow hyd. No.	= 1 - Post_Developed_PK3	Max. Elevation	= 152.74 ft
Reservoir name	= Detention Pond-F	Max. Storage	= 31,361 cuft

Storage Indication method used.



Hydrograph Report

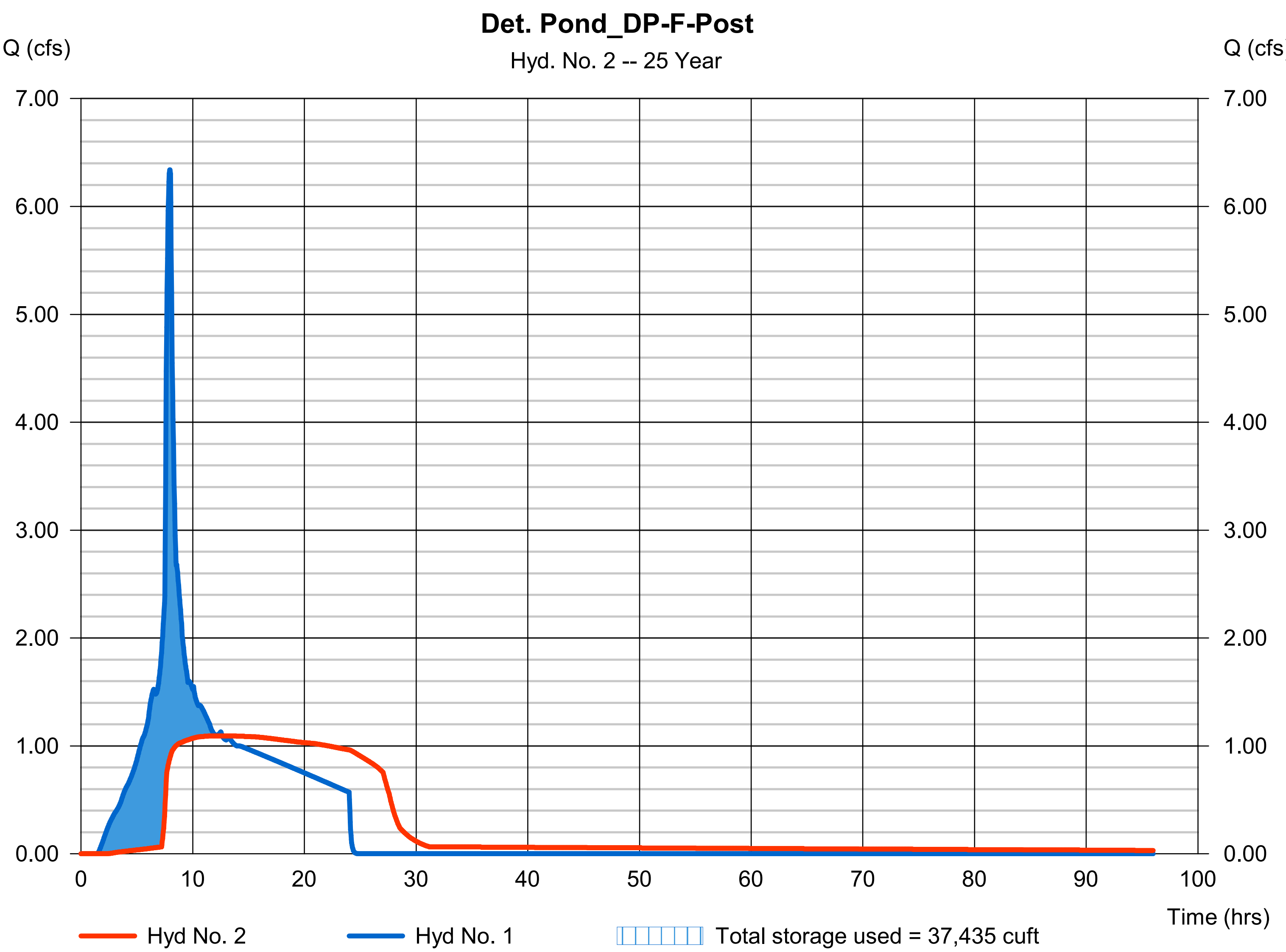
Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2021 Wednesday, 06 / 5 / 2024

Hyd. No. 2

Det. Pond_DP-F-Post

Hydrograph type	= Reservoir	Peak discharge	= 1.093 cfs
Storm frequency	= 25 yrs	Time to peak	= 12.63 hrs
Time interval	= 2 min	Hyd. volume	= 87,111 cuft
Inflow hyd. No.	= 1 - Post_Developed_PK3	Max. Elevation	= 153.16 ft
Reservoir name	= Detention Pond-F	Max. Storage	= 37,435 cuft

Storage Indication method used.



DETENTION POND G

Hydrograph Report

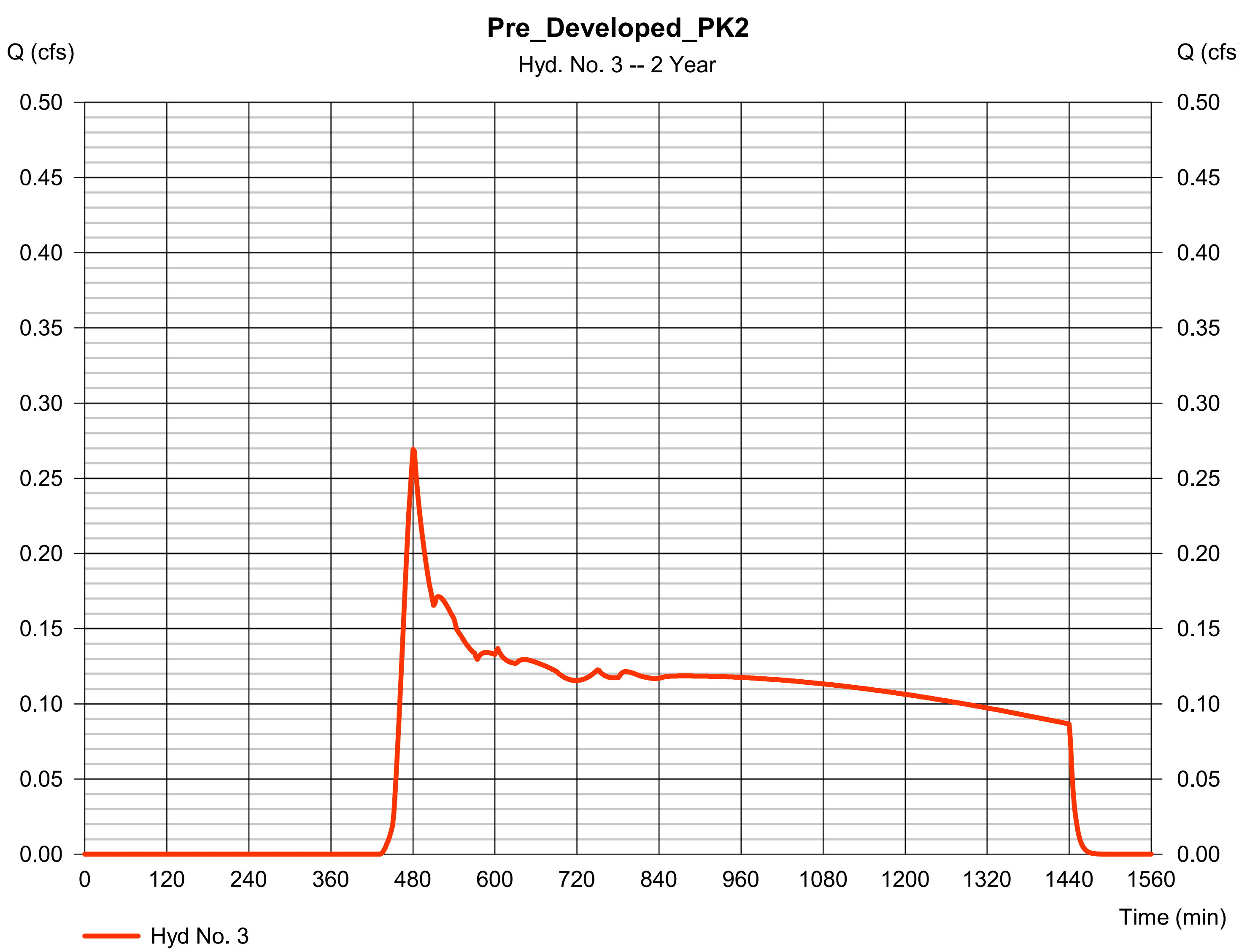
Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2021 Friday, 06 / 28 / 2024

Hyd. No. 3

Pre_Developed_PK2

Hydrograph type	= SBUH Runoff	Peak discharge	= 0.269 cfs
Storm frequency	= 2 yrs	Time to peak	= 480 min
Time interval	= 2 min	Hyd. volume	= 7,110 cuft
Drainage area	= 3.220 ac	Curve number	= 74*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 6.70 min
Total precip.	= 2.50 in	Distribution	= Type IA
Storm duration	= 24 hrs	Shape factor	= n/a

* Composite (Area/CN) = [(0.209 x 98) + (1.745 x 75) + (1.271 x 69)] / 3.220



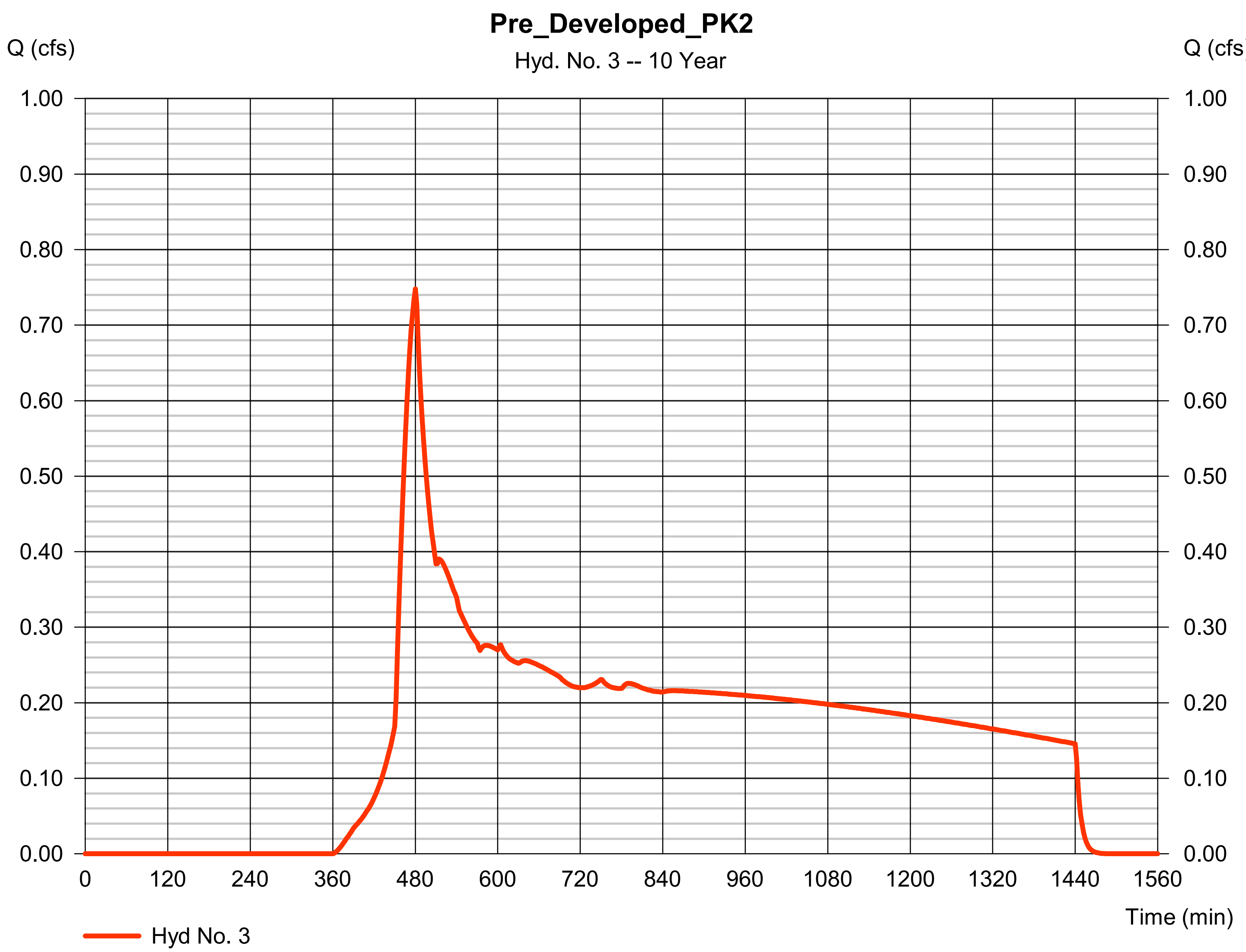
Hydrograph Report

Hyd. No. 3

Pre_Developed_PK2

Hydrograph type	= SBUH Runoff	Peak discharge	= 0.748 cfs
Storm frequency	= 10 yrs	Time to peak	= 480 min
Time interval	= 2 min	Hyd. volume	= 14,091 cuft
Drainage area	= 3.220 ac	Curve number	= 74*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 6.70 min
Total precip.	= 3.45 in	Distribution	= Type IA
Storm duration	= 24 hrs	Shape factor	= n/a

* Composite (Area/CN) = [(0.209 x 98) + (1.745 x 75) + (1.271 x 69)] / 3.220



Hydrograph Report

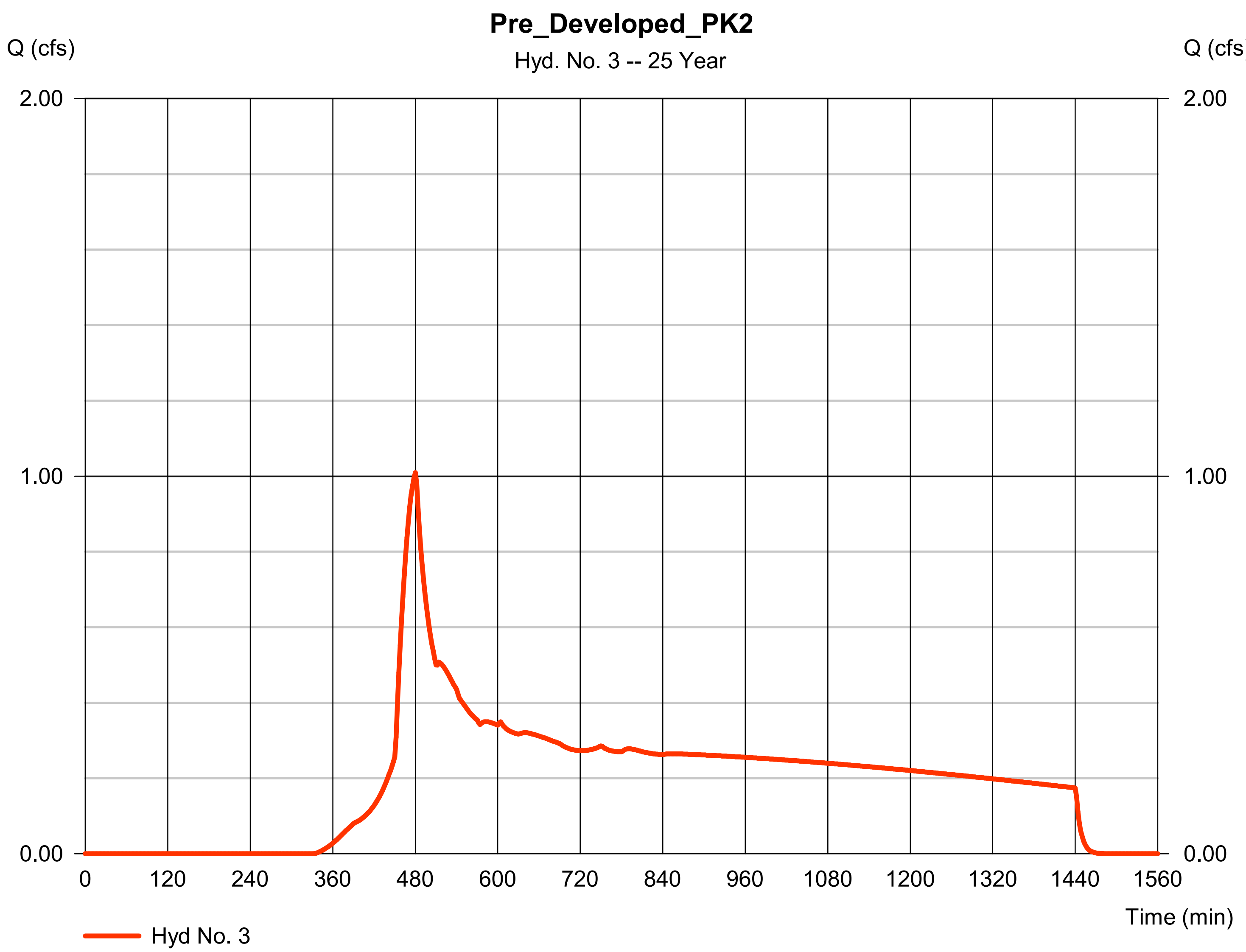
Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2021 Friday, 06 / 28 / 2024

Hyd. No. 3

Pre_Developed_PK2

Hydrograph type	= SBUH Runoff	Peak discharge	= 1.010 cfs
Storm frequency	= 25 yrs	Time to peak	= 480 min
Time interval	= 2 min	Hyd. volume	= 17,805 cuft
Drainage area	= 3.220 ac	Curve number	= 74*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 6.70 min
Total precip.	= 3.90 in	Distribution	= Type IA
Storm duration	= 24 hrs	Shape factor	= n/a

* Composite (Area/CN) = [(0.209 x 98) + (1.745 x 75) + (1.271 x 69)] / 3.220



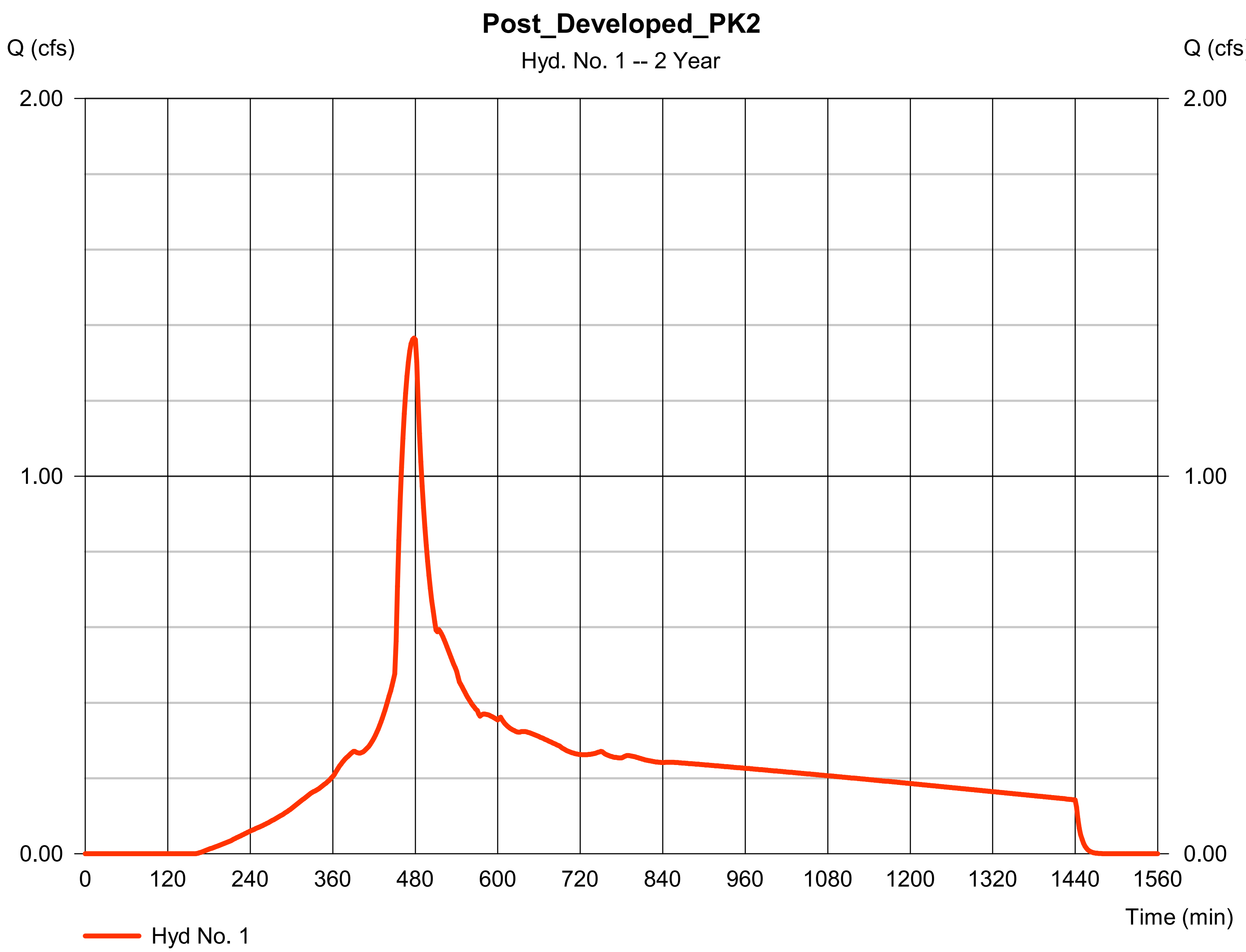
Hydrograph Report

Hyd. No. 1

Post_Developed_PK2

Hydrograph type	= SBUH Runoff	Peak discharge	= 1.366 cfs
Storm frequency	= 2 yrs	Time to peak	= 478 min
Time interval	= 2 min	Hyd. volume	= 19,790 cuft
Drainage area	= 3.220 ac	Curve number	= 92*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 6.70 min
Total precip.	= 2.50 in	Distribution	= Type IA
Storm duration	= 24 hrs	Shape factor	= n/a

* Composite (Area/CN) = [(2.590 x 98) + (0.630 x 69)] / 3.220



Hydrograph Report

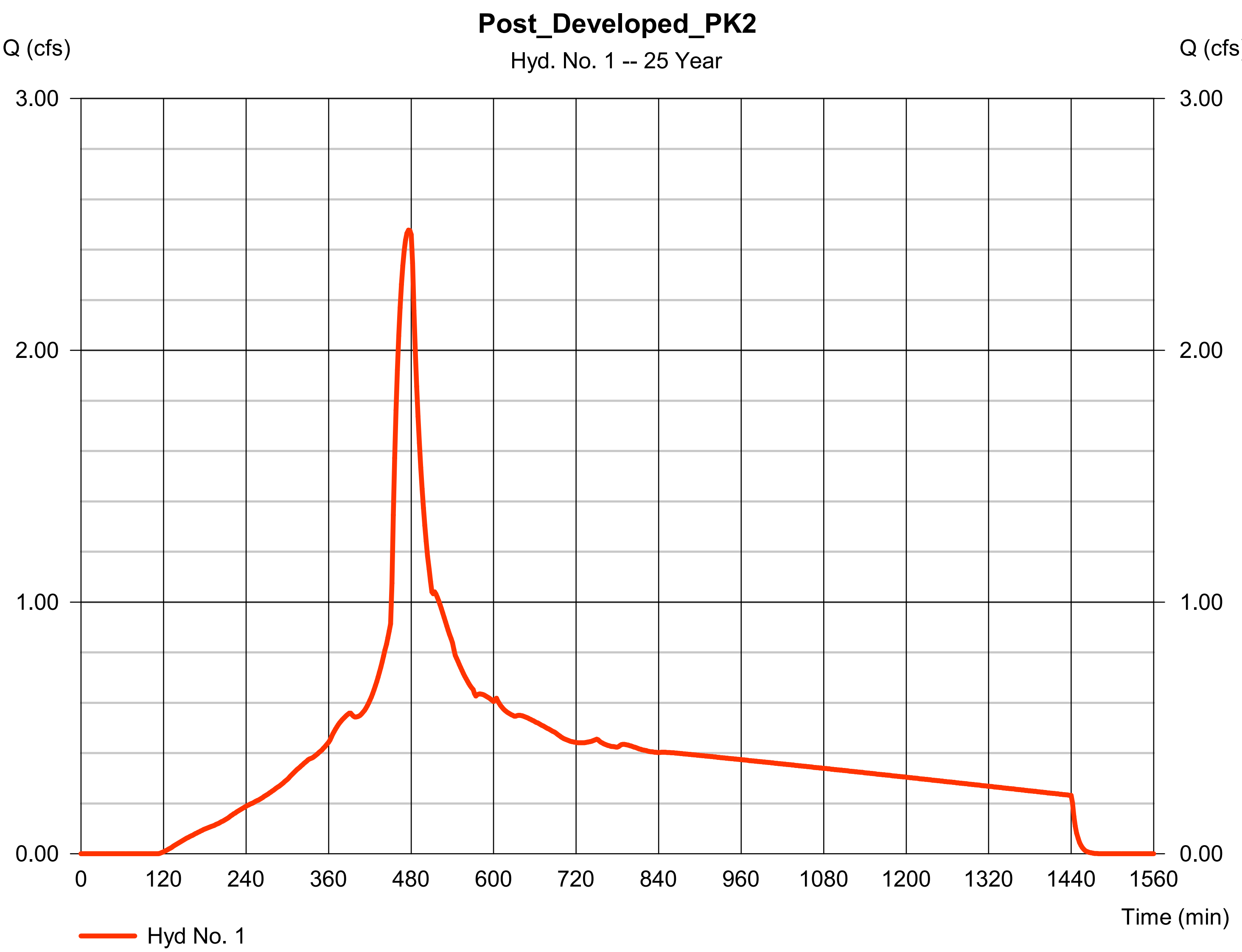
Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2021 Friday, 06 / 28 / 2024

Hyd. No. 1

Post_Developed_PK2

Hydrograph type	= SBUH Runoff	Peak discharge	= 2.477 cfs
Storm frequency	= 25 yrs	Time to peak	= 476 min
Time interval	= 2 min	Hyd. volume	= 35,312 cuft
Drainage area	= 3.220 ac	Curve number	= 92*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 6.70 min
Total precip.	= 3.90 in	Distribution	= Type IA
Storm duration	= 24 hrs	Shape factor	= n/a

* Composite (Area/CN) = [(2.590 x 98) + (0.630 x 69)] / 3.220



Hydrograph Report

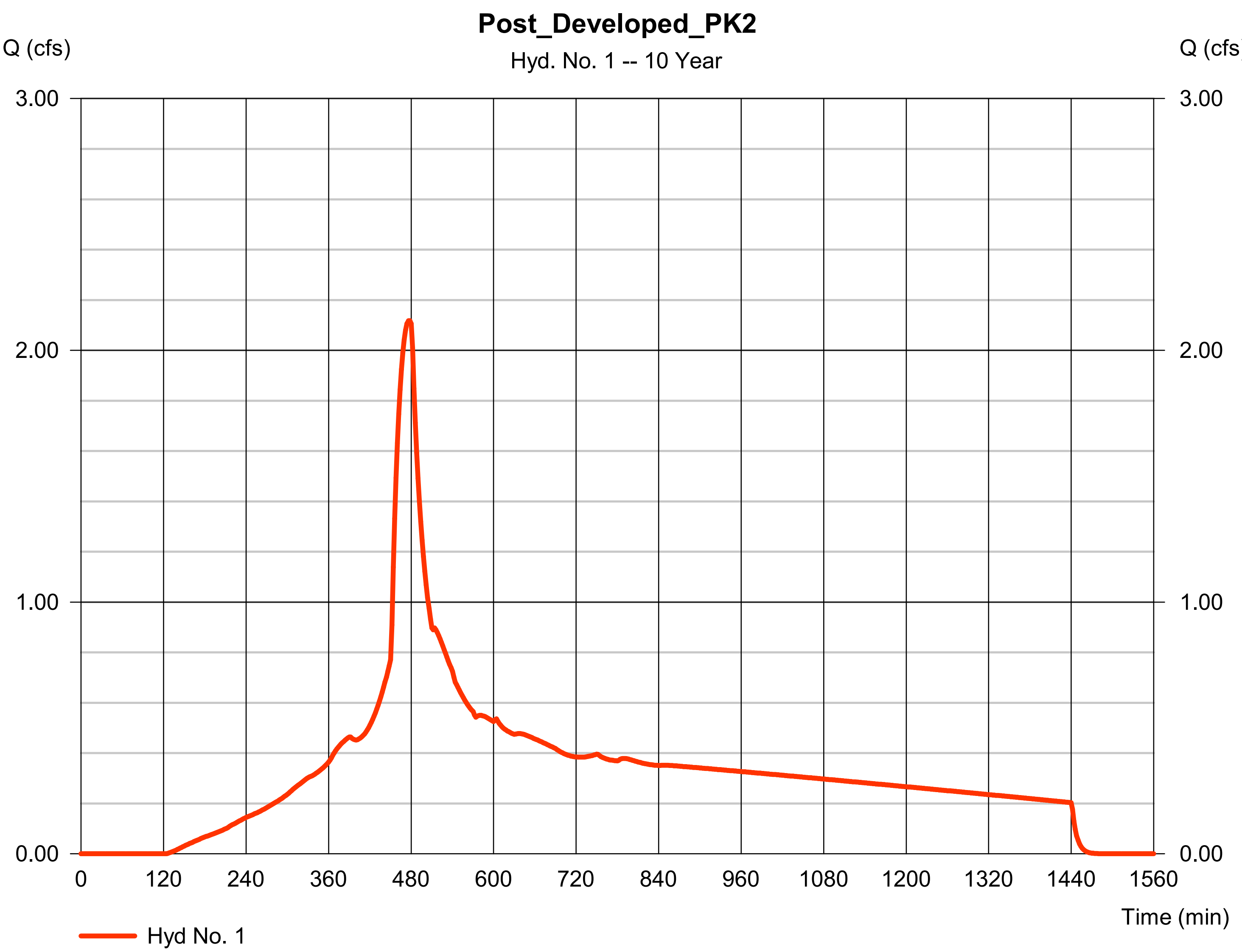
Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2021 Friday, 06 / 28 / 2024

Hyd. No. 1

Post_Developed_PK2

Hydrograph type	= SBUH Runoff	Peak discharge	= 2.118 cfs
Storm frequency	= 10 yrs	Time to peak	= 476 min
Time interval	= 2 min	Hyd. volume	= 30,261 cuft
Drainage area	= 3.220 ac	Curve number	= 92*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 6.70 min
Total precip.	= 3.45 in	Distribution	= Type IA
Storm duration	= 24 hrs	Shape factor	= n/a

* Composite (Area/CN) = [(2.590 x 98) + (0.630 x 69)] / 3.220



Hydrograph Report

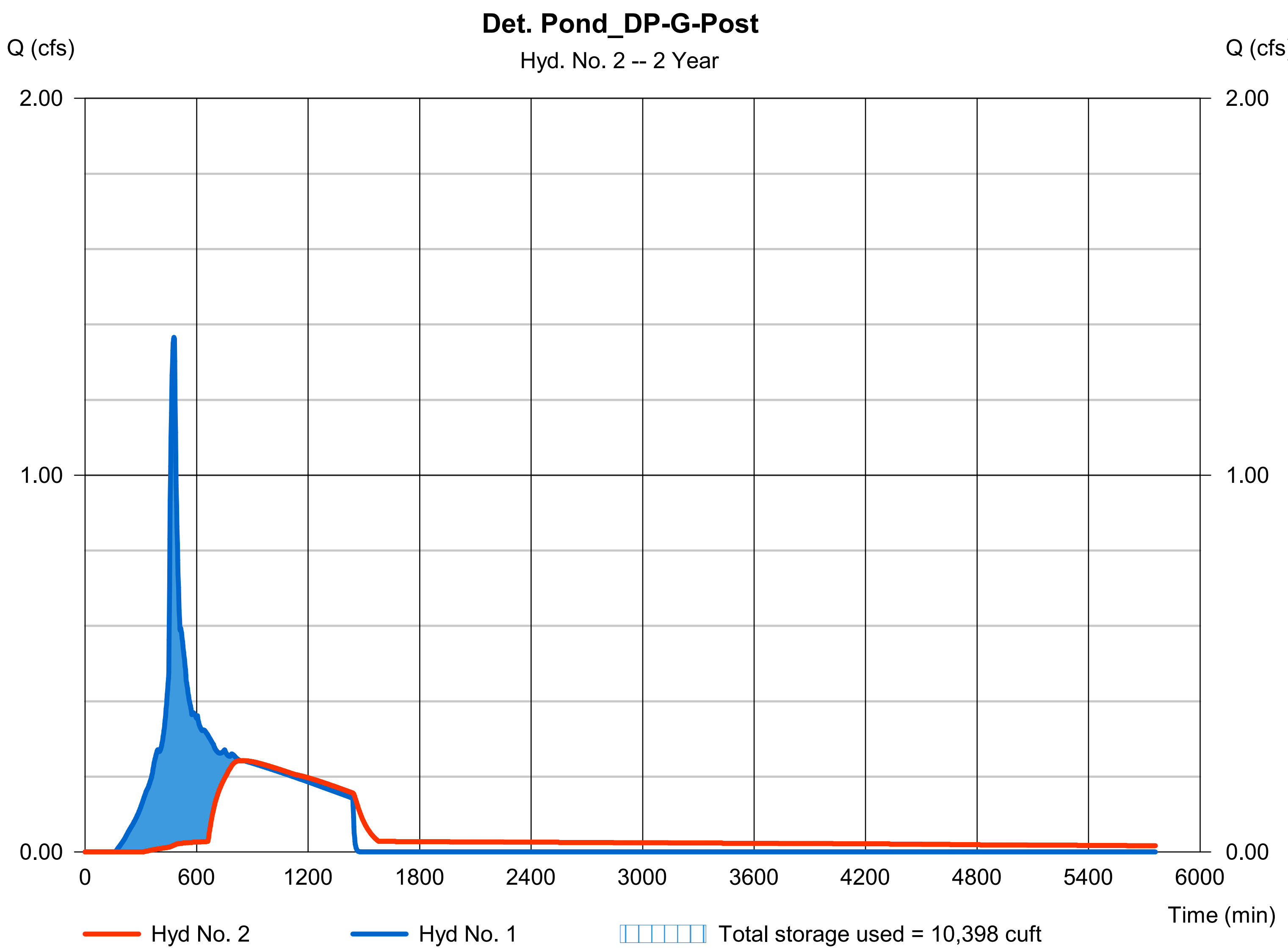
Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2021 Friday, 06 / 28 / 2024

Hyd. No. 2

Det. Pond_DP-G-Post

Hydrograph type	= Reservoir	Peak discharge	= 0.242 cfs
Storm frequency	= 2 yrs	Time to peak	= 854 min
Time interval	= 2 min	Hyd. volume	= 15,984 cuft
Inflow hyd. No.	= 1 - Post_Developed_PK2	Max. Elevation	= 175.61 ft
Reservoir name	= Detention Pond-G	Max. Storage	= 10,398 cuft

Storage Indication method used.



Hydrograph Report

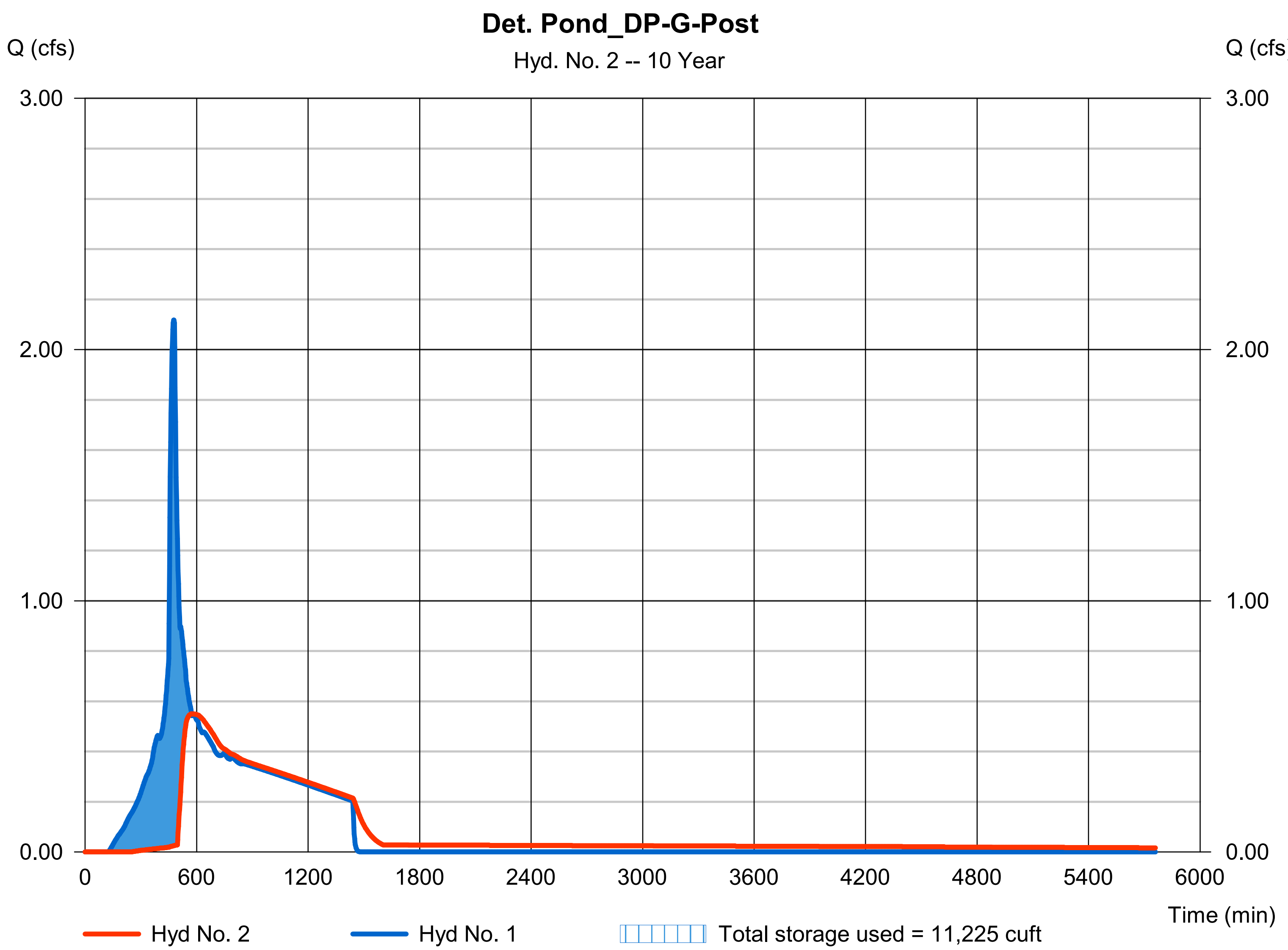
Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2021 Friday, 06 / 28 / 2024

Hyd. No. 2

Det. Pond_DP-G-Post

Hydrograph type	= Reservoir	Peak discharge	= 0.550 cfs
Storm frequency	= 10 yrs	Time to peak	= 572 min
Time interval	= 2 min	Hyd. volume	= 26,431 cuft
Inflow hyd. No.	= 1 - Post_Developed_PK2	Max. Elevation	= 175.71 ft
Reservoir name	= Detention Pond-G	Max. Storage	= 11,225 cuft

Storage Indication method used.



Hydrograph Report

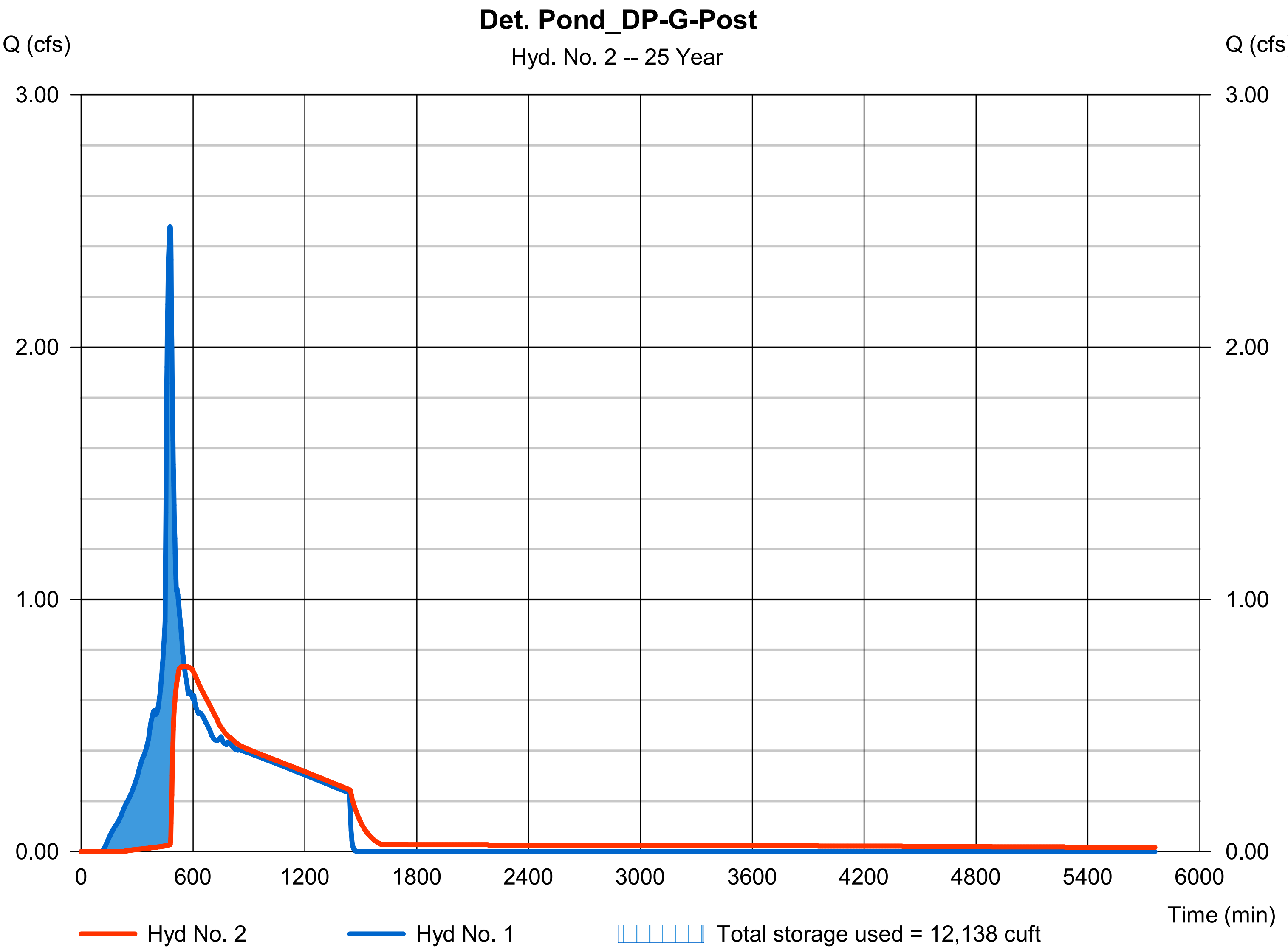
Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2021 Friday, 06 / 28 / 2024

Hyd. No. 2

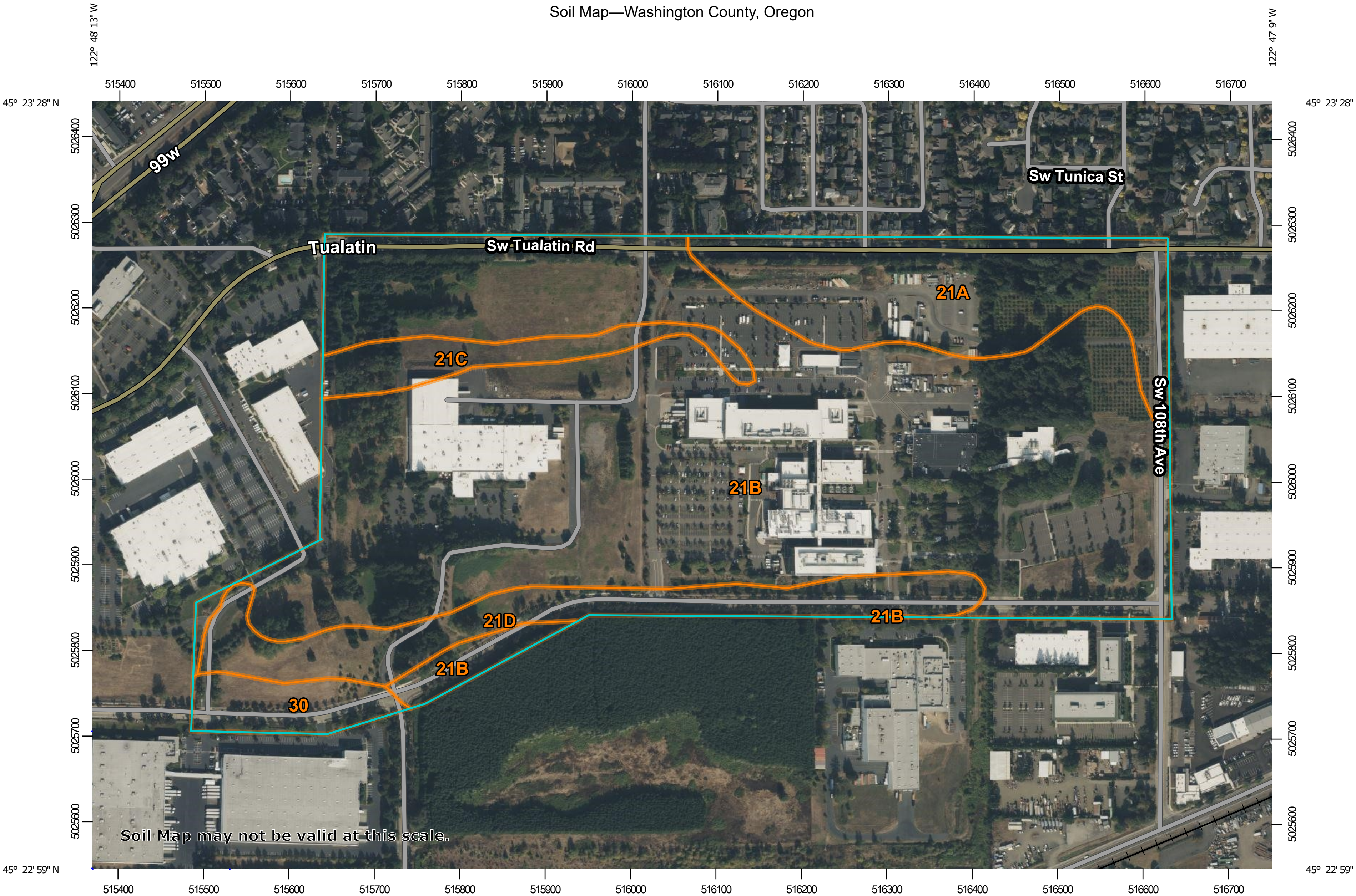
Det. Pond_DP-G-Post

Hydrograph type	= Reservoir	Peak discharge	= 0.735 cfs
Storm frequency	= 25 yrs	Time to peak	= 552 min
Time interval	= 2 min	Hyd. volume	= 31,477 cuft
Inflow hyd. No.	= 1 - Post_Developed_PK2	Max. Elevation	= 175.82 ft
Reservoir name	= Detention Pond-G	Max. Storage	= 12,138 cuft

Storage Indication method used.



APPENDIX C
USDA HYDROLOGIC SOIL GROUPS MAP REPORT



Map Scale: 1:6,310 if printed on A landscape (11" x 8.5") sheet.

Meters: 0, 50, 100, 200, 300

Feet: 0, 300, 600, 1200, 1800

N

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

Map Unit Legend

Map Unit Symbol	Map Unit Name	Acres in AOI	Percent of AOI
21A	Hillsboro loam, 0 to 3 percent slopes	16.2	13.2%
21B	Hillsboro loam, 3 to 7 percent slopes	87.7	71.6%
21C	Hillsboro loam, 7 to 12 percent slopes	4.5	3.6%
21D	Hillsboro loam, 12 to 20 percent slopes	10.6	8.7%
30	McBee silty clay loam	3.5	2.8%
Totals for Area of Interest		**122.5**	**100.0%**

Engineering Properties

This table gives the engineering classifications and the range of engineering properties for the layers of each soil in the survey area.

Hydrologic soil group is a group of soils having similar runoff potential under similar storm and cover conditions. The criteria for determining Hydrologic soil group is found in the National Engineering Handbook, Chapter 7 issued May 2007(http://directives.sc.egov.usda.gov/OpenNonWebContent.aspx?content=17757.wba). Listing HSGs by soil map unit component and not by soil series is a new concept for the engineers. Past engineering references contained lists of HSGs by soil series. Soil series are continually being defined and redefined, and the list of soil series names changes so frequently as to make the task of maintaining a single national list virtually impossible. Therefore, the criteria is now used to calculate the HSG using the component soil properties and no such national series lists will be maintained. All such references are obsolete and their use should be discontinued. Soil properties that influence runoff potential are those that influence the minimum rate of infiltration for a bare soil after prolonged wetting and when not frozen. These properties are depth to a seasonal high water table, saturated hydraulic conductivity after prolonged wetting, and depth to a layer with a very slow water transmission rate. Changes in soil properties caused by land management or climate changes also cause the hydrologic soil group to change. The influence of ground cover is treated independently. There are four hydrologic soil groups, A, B, C, and D, and three dual groups, A/D, B/D, and C/D. In the dual groups, the first letter is for drained areas and the second letter is for undrained areas.

The four hydrologic soil groups are described in the following paragraphs:

Group A. Soils having a high infiltration rate (low runoff potential) when thoroughly wet. These consist mainly of deep, well drained to excessively drained sands or gravelly sands. These soils have a high rate of water transmission.

Group B. Soils having a moderate infiltration rate when thoroughly wet. These consist chiefly of moderately deep or deep, moderately well drained or well drained soils that have moderately fine texture to moderately coarse texture. These soils have a moderate rate of water transmission.

Group C. Soils having a slow infiltration rate when thoroughly wet. These consist chiefly of soils having a layer that impedes the downward movement of water or soils of moderately fine texture or fine texture. These soils have a slow rate of water transmission.

Group D. Soils having a very slow infiltration rate (high runoff potential) when thoroughly wet. These consist chiefly of clays that have a high shrink-swell potential, soils that have a high water table, soils that have a claypan or clay layer at or near the surface, and soils that are shallow over nearly impervious material. These soils have a very slow rate of water transmission.

Depth to the upper and lower boundaries of each layer is indicated.

Texture is given in the standard terms used by the U.S. Department of Agriculture. These terms are defined according to percentages of sand, silt, and clay in the fraction of the soil that is less than 2 millimeters in diameter. "Loam," for example, is soil that is 7 to 27 percent clay, 28 to 50 percent silt, and less than 52 percent sand. If the content of particles coarser than sand is 15 percent or more, an appropriate modifier is added, for example, "gravelly."

Classification of the soils is determined according to the Unified soil classification system (ASTM, 2005) and the system adopted by the American Association of State Highway and Transportation Officials (AASHTO, 2004).

The Unified system classifies soils according to properties that affect their use as construction material. Soils are classified according to particle-size distribution of the fraction less than 3 inches in diameter and according to plasticity index, liquid limit, and organic matter content. Sandy and gravelly soils are identified as GW, GP, GM, GC, SW, SP, SM, and SC; silty and clayey soils as ML, CL, OL, MH, CH, and OH; and highly organic soils as PT. Soils exhibiting engineering properties of two groups can have a dual classification, for example, CL-ML.

The AASHTO system classifies soils according to those properties that affect roadway construction and maintenance. In this system, the fraction of a mineral soil that is less than 3 inches in diameter is classified in one of seven groups from A-1 through A-7 on the basis of particle-size distribution, liquid limit, and plasticity index. Soils in group A-1 are coarse grained and low in content of fines (silt and clay). At the other extreme, soils in group A-7 are fine grained. Highly organic soils are classified in group A-8 on the basis of visual inspection.

If laboratory data are available, the A-1, A-2, and A-7 groups are further classified as A-1-a, A-1-b, A-2-4, A-2-5, A-2-6, A-2-7, A-7-5, or A-7-6. As an additional refinement, the suitability of a soil as subgrade material can be indicated by a group index number. Group index numbers range from 0 for the best subgrade material to 20 or higher for the poorest.

Percentage of rock fragments larger than 10 inches in diameter and 3 to 10 inches in diameter are indicated as a percentage of the total soil on a dry-weight basis. The percentages are estimates determined mainly by converting volume percentage in the field to weight percentage. Three values are provided to identify the expected Low (L), Representative Value (R), and High (H).

Percentage (of soil particles) passing designated sieves is the percentage of the soil fraction less than 3 inches in diameter based on an ovendry weight. The sieves, numbers 4, 10, 40, and 200 (USA Standard Series), have openings of 4.76, 2.00, 0.420, and 0.074 millimeters, respectively. Estimates are based on laboratory tests of soils sampled in the survey area and in nearby areas and on estimates made in the field. Three values are provided to identify the expected Low (L), Representative Value (R), and High (H).

Liquid limit and *plasticity index* (Atterberg limits) indicate the plasticity characteristics of a soil. The estimates are based on test data from the survey area or from nearby areas and on field examination. Three values are provided to identify the expected Low (L), Representative Value (R), and High (H).

References:

American Association of State Highway and Transportation Officials (AASHTO). 2004. Standard specifications for transportation materials and methods of sampling and testing. 24th edition.

American Society for Testing and Materials (ASTM). 2005. Standard classification of soils for engineering purposes. ASTM Standard D2487-00.

Report—Engineering Properties

Absence of an entry indicates that the data were not estimated. The asterisk '*' denotes the representative texture; other possible textures follow the dash. The criteria for determining the hydrologic soil group for individual soil components is found in the National Engineering Handbook, Chapter 7 issued May 2007(http://directives.sc.egov.usda.gov/OpenNonWebContent.aspx?content=17757.wba). Three values are provided to identify the expected Low (L), Representative Value (R), and High (H).

Engineering Properties–Washington County, Oregon

Map unit symbol and soil name	Pct. of map unit	Hydrologic group	Depth	USDA texture	Classification		Pct Fragments		Percentage passing sieve number—				Liquid limit	Plasticity index
					Unified	AASHTO	>10 inches	3-10 inches	4	10	40	200		
			In				*L-R-H*	*L-R-H*	*L-R-H*	*L-R-H*	*L-R-H*	*L-R-H*	*L-R-H*	*L-R-H*
21A—Hillsboro loam, 0 to 3 percent slopes														
Hillsboro	90	B	0-15	Loam	ML	A-4	0- 0- 0	0- 0- 0	100-100-100	100-100-100	95-98-100	75-83-90	30-33-35	NP-3 -5
			15-48	Loam, silt loam	ML	A-4	0- 0- 0	0- 0- 0	100-100-100	100-100-100	95-98-100	75-83-90	30-33-35	5-8 -10
			48-57	Fine sandy loam	SM	A-2, A-4	0- 0- 0	0- 0- 0	100-100-100	100-100-100	95-98-100	30-40-50	20-25-30	NP-3 -5
			57-81	Loamy fine sand, fine sand	SM	A-2	0- 0- 0	0- 0- 0	100-100-100	100-100-100	65-73-80	20-28-35	20-23-25	NP-3 -5
21B—Hillsboro loam, 3 to 7 percent slopes														
Hillsboro	90	B	0-15	Loam	ML	A-4	0- 0- 0	0- 0- 0	100-100-100	100-100-100	95-98-100	75-83-90	30-33-35	NP-3 -5
			15-48	Silt loam, loam	ML	A-4	0- 0- 0	0- 0- 0	100-100-100	100-100-100	95-98-100	75-83-90	30-33-35	5-8 -10
			48-57	Fine sandy loam	SM	A-2, A-4	0- 0- 0	0- 0- 0	100-100-100	100-100-100	95-98-100	30-40-50	20-25-30	NP-3 -5
			57-81	Loamy fine sand, fine sand	SM	A-2	0- 0- 0	0- 0- 0	100-100-100	100-100-100	65-73-80	20-28-35	20-23-25	NP-3 -5

Engineering Properties–Washington County, Oregon														
Map unit symbol and soil name	Pct. of map unit	Hydrologic group	Depth	USDA texture	Classification		Pct Fragments		Percentage passing sieve number—				Liquid limit	Plasticity index
					Unified	AASHTO	>10 inches	3-10 inches	4	10	40	200		
			In				*L-R-H*	*L-R-H*	*L-R-H*	*L-R-H*	*L-R-H*	*L-R-H*	*L-R-H*	*L-R-H*
21C—Hillsboro loam, 7 to 12 percent slopes														
Hillsboro	90	B	0-15	Loam	ML	A-4	0- 0- 0	0- 0- 0	100-100 -100	100-100 -100	95-98-100	75-83-90	30-33 -35	NP-3 -5
			15-48	Loam, silt loam	ML	A-4	0- 0- 0	0- 0- 0	100-100 -100	100-100 -100	95-98-100	75-83-90	30-33 -35	5-8 -10
			48-57	Fine sandy loam	SM	A-2, A-4	0- 0- 0	0- 0- 0	100-100 -100	100-100 -100	95-98-100	30-40-50	20-25 -30	NP-3 -5
			57-81	Fine sand, loamy fine sand	SM	A-2	0- 0- 0	0- 0- 0	100-100 -100	100-100 -100	65-73-80	20-28-35	20-23 -25	NP-3 -5
21D—Hillsboro loam, 12 to 20 percent slopes														
Hillsboro	90	B	0-15	Loam	ML	A-4	0- 0- 0	0- 0- 0	100-100 -100	100-100 -100	95-98-100	75-83-90	30-33 -35	NP-3 -5
			15-48	Loam, silt loam	ML	A-4	0- 0- 0	0- 0- 0	100-100 -100	100-100 -100	95-98-100	75-83-90	30-33 -35	5-8 -10
			48-57	Fine sandy loam	SM	A-2, A-4	0- 0- 0	0- 0- 0	100-100 -100	100-100 -100	95-98-100	30-40-50	20-25 -30	NP-3 -5
			57-81	Loamy fine sand, fine sand	SM	A-2	0- 0- 0	0- 0- 0	100-100 -100	100-100 -100	65-73-80	20-28-35	20-23 -25	NP-3 -5

Engineering Properties–Washington County, Oregon														
Map unit symbol and soil name	Pct. of map unit	Hydrologic group	Depth	USDA texture	Classification		Pct Fragments		Percentage passing sieve number—				Liquid limit	Plasticity index
					Unified	AASHTO	>10 inches	3-10 inches	4	10	40	200		
			In				*L-R-H*	*L-R-H*	*L-R-H*	*L-R-H*	*L-R-H*	*L-R-H*	*L-R-H*	*L-R-H*
30—McBee silty clay loam														
Mcbee	85	C	0-11	Silty clay loam	ML	A-6	0- 0- 0	0- 0- 0	100-100-100	100-100-100	95-98-100	85-90-95	35-38-40	10-13-15
			11-45	Silty clay loam, clay loam	CL, ML	A-6	0- 0- 0	0- 0- 0	100-100-100	100-100-100	95-98-100	80-88-95	34-37-40	10-13-15
			45-65	Clay, clay loam, silty clay loam, gravelly clay loam, silty clay	CL, GC	A-6, A-7	0- 0- 0	0- 0- 0	55-78-100	50-75-100	45-73-100	40-68-95	30-35-45	10-15-25

Data Source Information

Soil Survey Area: Washington County, Oregon
Survey Area Data: Version 23, Sep 7, 2023

APPENDIX D
EXCERPTS OF GEOTECHNICAL REPORT

Report of Geotechnical Engineering Services

Lam Lab Repositioning

Tualatin, Oregon

December 20, 2023

Geotechnical ■ Environmental ■ Special Inspections

Columbia West

Engineering, Inc




www.columbiawestengineering.com

December 20, 2023

Lam Research Corporation
c/o Mackenzie
River East Center
1515 SE Water Avenue, Suite 100
Portland, OR 97214

Attention: Bill Bezio

Report of Geotechnical Engineering Services
Lam Lab Repositioning
Lam Research Corporation Campus
11361 SW Leveton Drive
Tualatin, Oregon 97062
Columbia West Project: Lam-2-01-1

Columbia West is pleased to present this report of geotechnical engineering services for the Lam Lab Repositioning project at the Lam Research Corporation campus located at 11361 SW Leveton Drive in Tualatin, Oregon. Our services were conducted in accordance with our proposal dated October 30, 2023.

We appreciate the opportunity to work on the project. Please contact us if you have any questions regarding this document.

Sincerely,

Columbia West

Najib A. Kalas, PE
Principal Engineer

NAK:glw
Attachments
Document ID: Lam-2-01-1-122023-geor

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EXECUTIVE SUMMARY

This section provides a summary of the geotechnical considerations associated with the Lam Lab Repositioning project in Tualatin, Oregon. 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.

- The proposed building sub-fab level and at-grade level can be supported on a mat foundation or spread footings. Foundations that support column loads of less than 300 kips can be supported on undisturbed native soil or on structural fill underlain by undisturbed native soil. Column loads of 300 to 600 kips should be supported on spread footings underlain by 24-inch-thick granular pads. Topsoil and prior fill (if present) should be removed from under foundations.

- Undocumented fill was encountered to depths of up to approximately 12 feet BGS in the explorations. Foundation elements should not be supported on undocumented fill material. Based on the preliminary grading plan, we anticipate that the majority of the fill will be removed from the buildings. However, for any proposed at-grade foundation elements, any undocumented fill material remaining after cutting to finish floor level should be removed and re-compacted or replaced with structural fill.

- There is a risk for poor performance of floor slabs and pavements established directly over undocumented fill soil. Removal and replacement of the undocumented fill reduce this risk significantly. 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 and pavement areas.

- Fill of up to approximately 15 feet is proposed. The construction of settlement-sensitive structures such as footings, floor slabs, and pavements should not commence until fill-induced settlement is complete.

- Our explorations in vegetated portions of the site encountered topsoil in the upper 8 to 12 inches, which includes an approximately 2- to 4-inch-thick root zone. The topsoil will provide poor support for foundations, floor slabs, and pavement. We recommend the topsoil be improved or stabilized after the root zone has been stripped.

- Based on pore water pressure dissipation tests in the CPTs and groundwater measurements in the piezometers on December 5, 2023, groundwater is anticipated to be at approximately elevation 129. Additional groundwater measurements will be required to evaluate the seasonal high groundwater levels at the site. Based on the proposed buildings and stormwater ponds layout and elevations, we recommend the sub-fab and basement levels of the buildings include perimeter foundation drains and underslab drainage.

- Considering the proximity of the stormwater ponds to Building H, which has a basement, and proposed grading of the ponds and the building, the basement should be sufficiently dry-proofed to limit infiltration of stormwater and perched water.

- A conventional soldier pile retaining wall with permanent tieback anchors will likely be required to support the proposed up to 20-foot-deep excavation for the new access road. Cantilever wall might be possible to support excavations of less than approximately 15 feet below existing grades if some settlement or horizontal movement can be tolerated with adjacent utilities, structures, or other improvements. MSE-type retaining walls can be constructed, however these walls will require larger excavation to accommodate the installation of wall elements including reinforced backfill.

- Based on subsurface conditions at the site, liquefaction and lateral spreading are not considered design considerations for the project.

- The planned project will require demolition of structures and pavement. Demolition should include complete removal of the floor slabs and buried foundation elements to allow for evaluating subgrades. After evaluation, the excavations should be backfilled with compacted structural fill.

- Moisture conditioning (drying) will be required to use the onsite soils as structural fill.

- The site soil is sensitive to moisture and is easily disturbed when at a moisture content that is above optimum. The subgrade should be protected from construction traffic. A granular working blanket consisting of imported granular material may be required to support construction activities. The contractor should select the thickness of the working blanket as they are in control of the type and frequency of construction traffic.

- Infiltration rates in the native sandy soils were measured at between 3.5 inch and 5.5 inches per hour.

- Pore water pressure dissipation tests completed in the CPTs indicated groundwater was present at depths between approximately 36.5 and 48.9 feet BGS (approximate elevations 121 to 126.6) at the time of our explorations (November 2023). Groundwater measurements in the piezometers on December 5, 2023, indicated groundwater at depths of more than 39 feet BGS at the locations of the piezometers. Perched groundwater is possible in the upper soil at the site during the wet winter to spring months.

- Caving and heaving (below groundwater) were observed in the sandy soil in prior explorations in the site vicinity. Site cuts and trench excavations will likely experience similar caving and heaving.

5.3.3 Groundwater

The depth and approximate elevation of groundwater measurements at the time of exploration and subsequent piezometers readings are shown in Table 1.

Table 1. Groundwater Depths and Elevations Summary

Location	Measurement Date	Groundwater Depth (feet BGS)	Groundwater Elevation
CPT-1	11/10/23	>51[1]	<126
CPT-2	11/10/23	48.9[1]	126.6
CPT-3	11/10/23	39.5[1]	126.5
CPT-4	11/10/23	36.5[1]	121
CPT-5	11/10/23	37.1[1]	124.4
CPT-6	11/10/23	46.8[1]	123.7
B-7 (P-1)	12/5/23	>39[2]	<127.5
B-8 (P-2)	12/5/23	>39[2]	<131

1. Groundwater depths were inferred from pore water pressure dissipation tests in the CPTs
2. Groundwater depths based on piezometers measurements

Based on pore water pressure dissipation tests in the CPTs and groundwater measurements in the piezometers on December 5, 2023, groundwater is anticipated to be at approximately elevation 129. Additional groundwater measurements will be required to evaluate the seasonal high groundwater levels at the site.

The depth to groundwater may fluctuate in response to seasonal changes, prolonged rainfall, changes in surface topography, and other factors not observed in this study. Perched groundwater zones are also likely in the upper soil at the site, particularly during extended periods of wet weather.

Seeps may become evident during site grading, primarily along slopes or in areas cut below existing grade. Structures, pavements, and drainage design should be planned accordingly.

5.3.4 Caving and Heaving

Caving was observed in the sandy soil in prior explorations in the site vicinity. Also, heaving/caving were encountered in prior borings in the site vicinity that extend into the sand (below groundwater), using hollow stem auger drilling methods. Site cuts and trench excavations will likely experience similar caving and heaving.

5.4 *INFILTRATION TESTING*

We understand stormwater infiltration systems are proposed for the project. We conducted an infiltration test in borings B-3, B-4, B-5, and B-9 at a depth of 9.5 feet BGS. Infiltration testing was completed using the single-ring, falling-head test method. Testing was completed until consistent rates were achieved. The results of our field infiltration testing are presented in the "Infiltration Systems" section. Locations of the tests are shown on Figure 2.

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 amendment thicknesses (if installed) do not account for construction traffic, and haul roads and staging areas should be used as described in the "Construction" section.

6.8 DRAINAGE

6.8.1 Temporary

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

6.8.2 Surface

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

6.8.3 Foundation Drains

Assuming the site grades around the buildings (without sub-fab or basement) will be sloped as discussed previously, it is our opinion that perimeter footing drains will not be required around these buildings. We recommend that perimeter foundation drains be installed in all areas of sub-fab and basement levels of the buildings, considering the proposed buildings and stormwater ponds layout and elevations. Also, the use of these drains should be considered in areas where landscaping planters are placed proximate to the foundations or where surface grades cannot be completed as outlined above.

Foundation drains should be constructed at a minimum slope of approximately ½ percent and pumped or drained by gravity to a suitable discharge. The perforated drainpipe should not be tied to a stormwater drainage system without backflow provisions. Foundation drains should consist of 4-inch-diameter, perforated drainpipe embedded in a minimum 2-foot-wide zone of crushed drain rock that extends to the ground surface. The invert elevation of the drainpipe should be installed at least 18 inches below the elevation of the floor slab.

The drain rock and geotextile should meet the requirements specified in the "Materials" section. The drain rock and geotextile should extend up the side of embedded walls to within a foot of the ground surface, geotextile wrapped over the top of the drain rock, as recommended in the "Retaining Structures" section.

6.8.4 Under Slab Drainage

In addition to the recommendations for foundation drains (see above), we recommend under slab drains be installed for the subfab and basement levels of the buildings, considering the proposed buildings and stormwater ponds layout and elevations. Further details regarding permanent dewatering systems will need to be developed once grading plans have been finalized and/or site conditions are exposed. Typical under slab drainage detail is presented on Figure 7.

The basements should be sufficiently dry-proofed to limit infiltration of perched water or infiltration water associated with the stormwater ponds.

6.9 INFILTRATION SYSTEMS

We understand stormwater infiltration systems are being considered for the proposed development. The locations and configurations were conceptual at the time of this report. The infiltration tests were performed to evaluate the infiltration potential for the proposed infiltration systems. The results of our field infiltration testing are presented in Table 6.

Table 6. Infiltration Testing Results

Exploration	Depth (feet BGS)	Observed Infiltration Rate[1] (inches per hour)	Soil Type	Percent Fines[2]
B-3	9.5	5.5	Sand with silt	10
B-4	9.5	Negligible	Silty sand - Fill	Not tested
B-5	9.5	3.5	Silty sand	18
B-9	9.5	3.5	Silty sand	15

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

The infiltration rates shown in Table 6 are short-term field rates and factors of safety have not been applied.

We recommend all infiltration systems be installed in the native sand (below the upper silt and clay or fill) and be at least 9.5 feet deep. Also, we recommend a minimum separation of 5 feet between the bottom of the infiltration systems and the groundwater table (approximate elevation of 129). We recommend the following unfactored field infiltration rates:

- For infiltration systems in the sand with silt, we recommend an unfactored field infiltration rate of 5.5 inches per hour.
- For infiltration systems in the silty sand, we recommend an unfactored field infiltration rate of 3.5 inch per hour.

We note that the high variability in the observed infiltration rates is due to high variability in fines content as presented in Table 6.

The recommended infiltration rates are measured rates and are unfactored. Correction factors should be applied to the recommended infiltration rates by the civil engineer during design to account for the degree of long-term maintenance and influent/pre-treatment control, as well as the potential for long-term clogging due to siltation and buildup of organic material, depending on the proposed length, location, and type of infiltration facility. We recommend a minimum factor of safety of at least 2 be applied to the recommended unfactored rates.

The actual depths and estimated infiltration rates can vary significantly from the values presented above. We recommend that the design infiltration values for the stormwater systems be confirmed by field testing completed during installation of the systems. The results of this field testing might necessitate that the stormwater system be enlarged to achieve the design infiltration rate.

6.10 PERMANENT SLOPES

Fill slopes should consist of structural fill material as discussed in the "Structural Fill" section. Final cut or fill slopes at the site should not exceed 2H:1V or 10 feet in height without individual slope stability analysis. Slopes that will be maintained by mowing should not be constructed steeper than 3H:1V. Access roads and pavements should be located at least 5 feet from the top of cut and fill slopes. The horizontal setback should be increased to 10 feet from the face of slopes for buildings.

Concentrated drainage or water flow over the face of slopes should be prohibited, and adequate protection against erosion is required. Fill slopes should be overbuilt, compacted, and trimmed at least two feet horizontally to provide adequate compaction of the outer slope face.

7.0 CONSTRUCTION

7.1 SITE PREPARATION

7.1.1 Demolition

Demolition includes complete removal of the existing buildings, retaining walls, pavement, concrete curbs, abandoned utilities, and any subsurface elements within 5 feet of areas to receive new pavement, buildings, retaining walls, or engineered fills. Demolished material should be transported off site for disposal. In general, this material will not be suitable for re-use as engineered fill. However, concrete pavement and base rock material may be recycled in accordance with the requirements set forth by the project jurisdiction and the recommendations provided in the "Structural Fill" section.

Excavations remaining from removing basements, foundations, utilities, and other subsurface elements should be backfilled with structural fill where these are below planned site grades. The base of the excavations should be excavated to expose firm subgrade before filling. The sides of the excavations should be cut into firm material and sloped a minimum of 1.5H:1V. Utility lines abandoned under new structural components should be completely removed and backfilled with structural fill or grouted full if left in place. Soft or disturbed soil encountered during demolition should be removed and replaced with structural fill.

Considerable subgrade damage can occur during demolition activities and we recommend that the subgrade protection measures discussed in the "Construction Considerations" section be implemented.

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 explorations in vegetated areas, the average depth of stripping will be approximately 2 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 areas of 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 Contaminated Media Management Plan.

7.1.3 Topsoil

An approximately 8- to 12-inch-deep agricultural topsoil was observed at the ground surface in our explorations in vegetated portions of the site. Reliable strength properties are extremely difficult to predict for the topsoil material. There is a high risk for poor performance of floor slabs and pavement established directly over topsoil. In order to reduce the risk of settlement, we recommend the topsoil be improved during site preparation in areas where planned cuts do not extend to the bottom of the topsoil (up to 12 inches). Prior to fill placement and construction, the topsoil should be improved by removing and replacing with structural fill or scarifying and re-compacting to structural fill requirements.

As discussed in the "Structural Fill" section, the native soil can be sensitive to small changes in moisture content and will be difficult, if not impossible, to compact adequately during wet weather. While scarification and compaction of the subgrade is the best option for subgrade improvement, it will likely only be possible during extended dry periods and following moisture conditioning of the soil. As discussed further on in this report, cement amendment is an option for conditioning the soil for use as structural fill during periods of wet weather or when drying the soil is not an option.

7.1.4 Undocumented Fill

7.1.4.1 General

Undocumented fill was encountered in eight explorations to depths of up to 12 feet BGS. 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.

7.1.4.2 Foundation Areas

Based on the preliminary grading plan, we anticipate that the majority of the fill will be removed from the buildings. However, for any proposed at-grade foundation elements, any undocumented fill material remaining after cutting to finish floor level should be removed and re-compacted or replaced with structural fill.

7.1.4.3 Floor Slab and Pavement Areas

There is a small risk for poor performance of floor slabs and pavements established directly over undocumented fill soil. Removal and replacement of the undocumented fill reduce this risk significantly. However, 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 and pavement areas.

7.1.4.4 Subgrade Observations

Considerable soil processing, including moisture conditioning and the removal of deleterious material from the undocumented fill, may be required during scarification and re-compaction or when using the excavated material as structural fill. We recommend that the exposed subgrade be closely evaluated by a geotechnical engineer during the process. Compaction should be performed as described in the "Structural Fill" section. As discussed further on in this report, cement amendment is an option for conditioning the soil for use as structural fill.

7.1.5 Subgrade Evaluation

Upon completion of stripping and prior to the placement of structural fill or pavement improvements, exposed subgrade soil should be evaluated by proof rolling with a fully loaded dump truck or similar heavy, rubber tire construction equipment. When the subgrade is too wet for proof rolling, a foundation probe may be used to identify areas of soft, loose, or unsuitable soil. Subgrade evaluation should be performed by Columbia West. If soft or yielding subgrade areas are identified during evaluation, we recommend the subgrade be over-excavated and backfilled with compacted imported granular fill.

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 areas and haul roads with increased thicknesses of base rock will be required. The amount of staging areas and haul roads, 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 are 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 "Materials" section.

7.3 EXCAVATION

7.3.1 General

The site was explored to a maximum depth of 51.8 feet BGS with a CPT rig. Conventional earthmoving equipment in proper working condition should be capable of making necessary site excavations of the on-site soil for site cuts and utilities.

If buried construction debris from prior on-site structures is encountered beneath the ground surface, this material will result in difficult trench excavations and may require additional effort or special equipment. If difficult excavations are encountered, trenches may also be wider than anticipated, increasing the amount of backfill material required.

Soil with higher sand content may be prone to raveling, and shoring will be required to maintain vertical excavation walls and protect adjacent facilities.

7.3.2 Temporary Slopes

The use of temporary cut slopes during construction is likely not possible where the excavation is adjacent to the existing buildings. For areas where cut-back construction slopes are feasible, temporary slopes for excavation of the sub-fab and basement of 1.5H:1V may be used to vertical depths of 10 feet BGS, provided groundwater is not encountered or is lowered to below the base of the excavation. We recommend a minimum horizontal distance of 5 feet from the edge of the existing foundations to the top of the 1.5H:1V sloped excavation. All cut slopes should be protected from erosion by covering them with plastic sheeting or other stabilizing cover during the rainy season. If sloughing or instability is observed, the slope might need to be flattened or the cut supported by shoring.

7.3.3 Utility Trench Excavation

Temporary excavation sidewalls should stand vertical to a depth of approximately 4 feet, provided groundwater seepage is not observed in the sidewalls and surcharge loads will not be present within H feet, where H is the depth of the trench. Open excavation techniques may be used to excavate trenches with depths between 4 and 8 feet, provided the walls of the excavation are cut at a slope of 1.5H:1V and groundwater seepage and surcharge loads are not present. At this inclination, slopes with loose sand may ravel and require some ongoing repair. Excavations should be flattened to 2H:1V if excessive sloughing or raveling occurs. In lieu of large and open cuts, approved temporary shoring may be used for excavation support. Use of approved temporary shoring is recommended where the slopes cannot be cut back, within the influence area of structural elements, and for cuts below the water table. The influence area can be defined as a 1H:1V slope extending down from a 5-foot setback from the edge of a foundation element. A wide variety of shoring and dewatering systems are available. Consequently, we recommend the contractor be responsible for selecting the appropriate shoring and dewatering systems.

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

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

7.3.4 Safety

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

7.4 DEWATERING

Based on pore water pressure dissipation tests in the CPTs and groundwater measurements in the piezometers on December 5, 2023, groundwater is anticipated to be approximately elevation 129. Additional groundwater measurements will be required to evaluate the seasonal high groundwater levels at the site.

Excavation dewatering will be necessary if groundwater is encountered during excavation of the sub-fab or basement levels. Significant dewatering is not anticipated for excavations less than 10 feet BGS. However, perched or static groundwater could be present at shallower depths after prolonged wet periods. Dewatering systems are best designed by the contractor. For excavations that do not exceed more than approximately 6 to 8 feet BGS, it should be possible to remove groundwater encountered by pumping from a sump in trenches. More intense use of pumps may be required at certain times of the year and where more intense seepage occurs. Removed water should be routed to a suitable discharge point.

If perched groundwater is encountered at the base of utility excavations, we recommend placing 1.5 to 2 feet of stabilization material at the base of the excavation. The use of a subgrade geotextile fabric may reduce the amount of stabilization material required. The actual thickness should be based on field observations during construction.

Trench stabilization material and the subgrade geotextile fabric should meet the requirements described in the "Materials" section. Trench stabilization material should be placed in one lift and compacted until well keyed. While we have described certain approaches to excavation dewatering, it is the contractor's responsibility to select the dewatering methods.

7.5 MATERIALS

7.5.1 Structural Fill

7.5.1.1 General

Fill should be placed on subgrade that has been prepared in conformance with the "Site Preparation" section. A variety of material may be used as structural fill at the site. However, all material used as structural fill should be free of organic material or other unsuitable material. A brief characterization of some of the acceptable materials and our recommendations for their use as structural fill are provided below.

7.5.1.2 On-Site Soil

The material at the site should be suitable for use as general structural fill, provided it is properly moisture conditioned and free of debris, organic material, and particles over 6 inches in diameter. Moisture conditioning (drying) will likely be required to use on-site fine-grained soil for structural fill. Accordingly, extended dry weather will be required to adequately condition and place the soil as structural fill. It will be difficult, if not impossible, to adequately compact on-site soil during the rainy season or during prolonged periods of rainfall.

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

7.5.1.3 Imported Granular Material

Imported granular material used as structural fill should be pit- or quarry-run rock, crushed rock, or crushed gravel and sand. The imported granular material should also be angular and fairly well graded between coarse and fine material, should have less than 5 percent fines by dry weight passing the U.S. Standard No. 200 sieve, and should have at least two mechanically fractured faces.

Imported granular material should be placed in lifts with a maximum uncompacted thickness of 12 inches and compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D1557. During the wet season or when wet subgrade conditions exists, the initial lift should be approximately 18 inches in uncompacted thickness and should be compacted by rolling with a smooth-drum roller without using vibratory action.

7.5.1.4 Stabilization Material

Stabilization material used in staging or haul road areas or in trenches should consist of 4- or 6-inch-minus pit- or quarry-run rock, crushed rock, or crushed gravel and sand. The material should have a maximum particle size of 6 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. Stabilization material should be placed in lifts between 12 and 24 inches thick and compacted to a firm condition.

7.5.1.5 Trench Backfill

Trench backfill placed beneath, adjacent to, and for at least 12 inches above utility lines (i.e., the pipe zone) should consist of durable, well-graded granular material with a maximum particle size of 1½ inches, should have less than 7 percent fines by dry weight, and should have at least two mechanically fractured faces. 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 roadway alignments, the remainder of the trench backfill up to the subgrade elevation should consist of durable, well-graded granular material with a maximum particle size of 2½ inches, should have less than 7 percent fines by dry weight, and should have at least two mechanically fractured faces. This material should be compacted to at least 92 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.

Outside of structural improvement areas (e.g., roadway alignments or building pads), trench backfill placed above the pipe zone may consist of general fill material that is free of organic material and material over 6 inches in diameter. This general 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.

7.5.1.6 Drain Rock

Drain rock should consist of angular, granular 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.

7.5.1.7 Aggregate Base Rock

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

7.5.1.8 Retaining Wall Select Backfill

Backfill material placed behind retaining walls and extending a horizontal distance of ½H, where H is the height of the retaining wall, should consist of imported granular material as described above and should have less than 7 percent fines by dry weight and have at least two mechanically fractured faces. We recommend the wall backfill be separated from general fill, native soil, and/or topsoil using a geotextile fabric that meets the specifications provided below for drainage geotextiles.

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

7.5.1.9 Retaining Wall Leveling Pad

Imported granular material placed at the base of retaining wall footings should consist of select granular material. The granular material should be 1"-0 to ¾"-0 aggregate size and have at least two mechanically fractured faces. The leveling pad material should be placed in a 6- to 12-inch-thick lift and compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D1557.

7.5.1.10 Existing Concrete and Crushed Rock

Concrete and crushed rock from the existing pavement areas and improvements can be used in general structural fill, provided particles greater than 3 inches are not present, it is thoroughly mixed and well graded so that there are no voids between the fragments, and the resulting mix is moisture conditioned for compaction. 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.

7.5.2 Geotextile Fabric

7.5.2.1 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.

7.5.2.2 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.

7.5.3 Soil Amendment with Cement

7.5.3.1 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, testing, and (if necessary) treating runoff from sites where cement amendment is used.

7.5.3.2 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 organic content and moisture of the topsoil, we recommend using a higher cement ratio when stabilizing topsoil zone, likely a minimum of 7 to 8 percent.

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 the 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.

7.5.3.3 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.

7.5.3.4 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 amendment of 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.

7.5.3.5 Testing

Cement-amendment of site soils should be observed and tested by Columbia West to document conformance with design recommendations. Cement spread rate should be verified with a pan sample test conducted at one random location per lift per 20,000 square feet of cement-amended fill. Treatment depth should be verified through excavation of a small test pit and measurement at one random location per lift of cement amended fill. Adequate compaction and moisture content should be verified by conducting nuclear gauge density testing at a frequency of approximately one test per 5,000 square feet of cement amended fill in accordance with ASTM D6938. At least one representative sample should be collected per day of cement-amendment, cured for 7 days, and tested for unconfined compressive strength in accordance with ASTM D1633. The tested samples should have a minimum 7-day, unconfined compressive strength of 100 psi.

7.5.4 AC

7.5.4.1 ACP

The AC should be Level 2, ½-inch, dense ACP according to OSSC 00744 (Asphalt Concrete Pavement) and compacted to 91 percent of the theoretical maximum density of the mix, as determined by AASHTO T 209. The minimum and maximum lift thicknesses are 2 and 3 inches, respectively, for ½-inch ACP. Asphalt binder should be performance graded and conform to PG 64-22 or better. 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.

7.5.4.2 Cold Weather Paving Considerations

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

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

7.6 EROSION CONTROL

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

8.0 OBSERVATION OF CONSTRUCTION

Satisfactory pavement, earthwork, and foundation performance depends to a large degree on the quality of construction. Sufficient observation of the contractor's activities is a key part of determining that the work is completed in accordance with the construction drawings and specifications. Columbia West should be retained to observe subgrade preparation, fill placement, foundation excavations, shoring installation, walls installation, drainage system installation, and pavement placement and to review laboratory compaction and field moisture-density information.

Subsurface conditions observed during construction should be compared with those encountered during the subsurface explorations. Recognition of changed conditions requires experience; therefore, qualified personnel should visit the site with sufficient frequency to detect whether subsurface conditions change significantly from those anticipated.

9.0 LIMITATIONS

We have prepared this report for use by Lam Research Corporation, Mackenzie, and members of the design and construction team for the proposed project. The data and report can be used for bidding or estimating purposes, but our report, conclusions, and interpretations should not be construed as warranty of the subsurface conditions and are not applicable to other sites.

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

The site development 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 generally accepted practices in this area at the time this report was prepared. No warranty, express or implied, should be understood.

♦ ♦ ♦

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

Sincerely,
Columbia West Engineering, Inc.

REGISTERED PROFESSIONAL
ENGINEER
84829PE
OREGON
November 10, 2015
NAJIB A. KALAS
EXPIRES: 06/30/25

Najib A. Kalas, PE
Principal Engineer

NAK

Document ID: Lam-2-01-1-122023-geor

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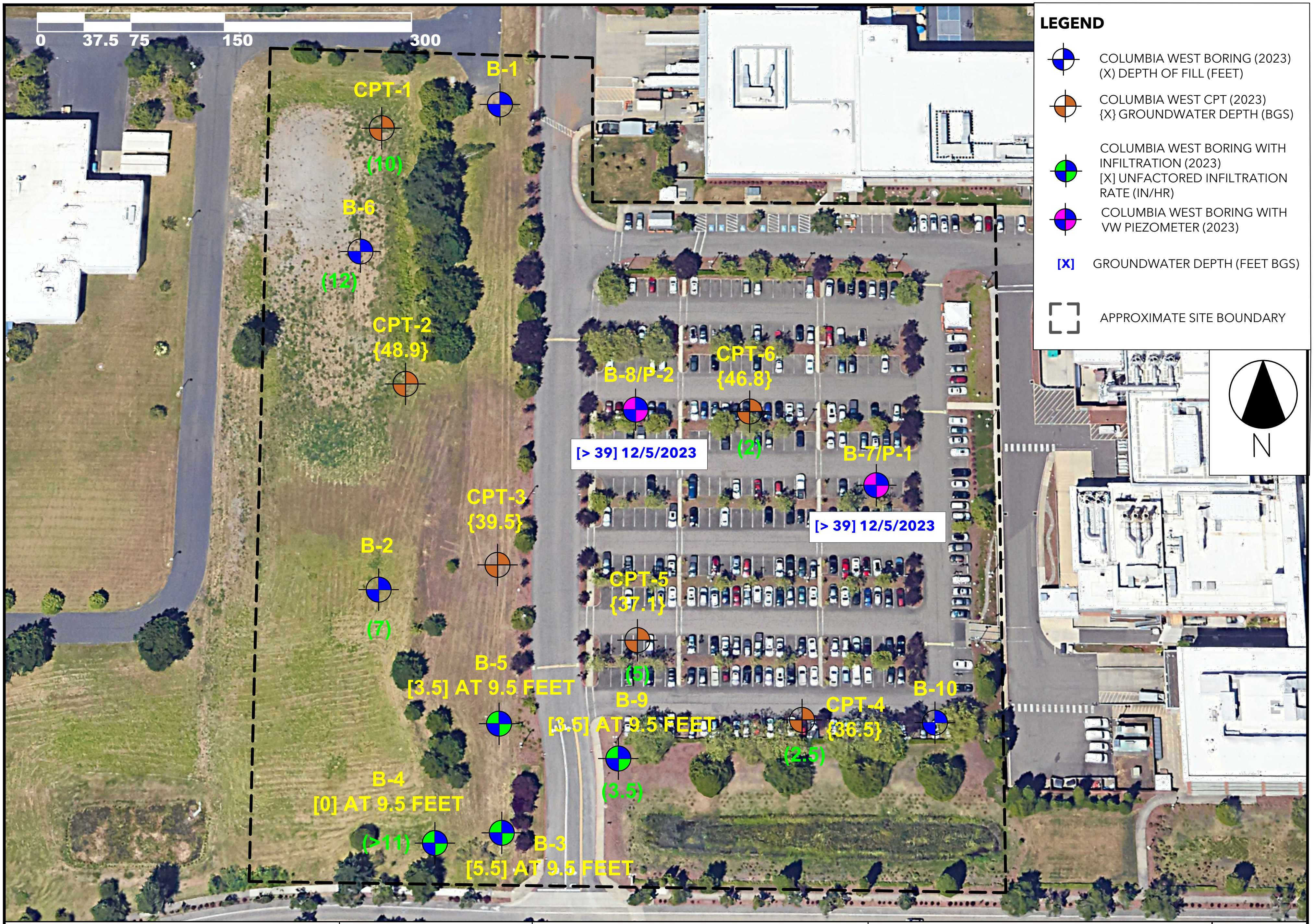
FIGURES



Geotechnical ▪ Environmental ▪ Special Inspections
Columbia West
Engineering, Inc

PROJECT: LAM-2-01-1
DATE: 11/29/23
DRAWN: EMU
CHECKED: NAK

EXPLORATION MAP
LAM LAB REPOSTIONING

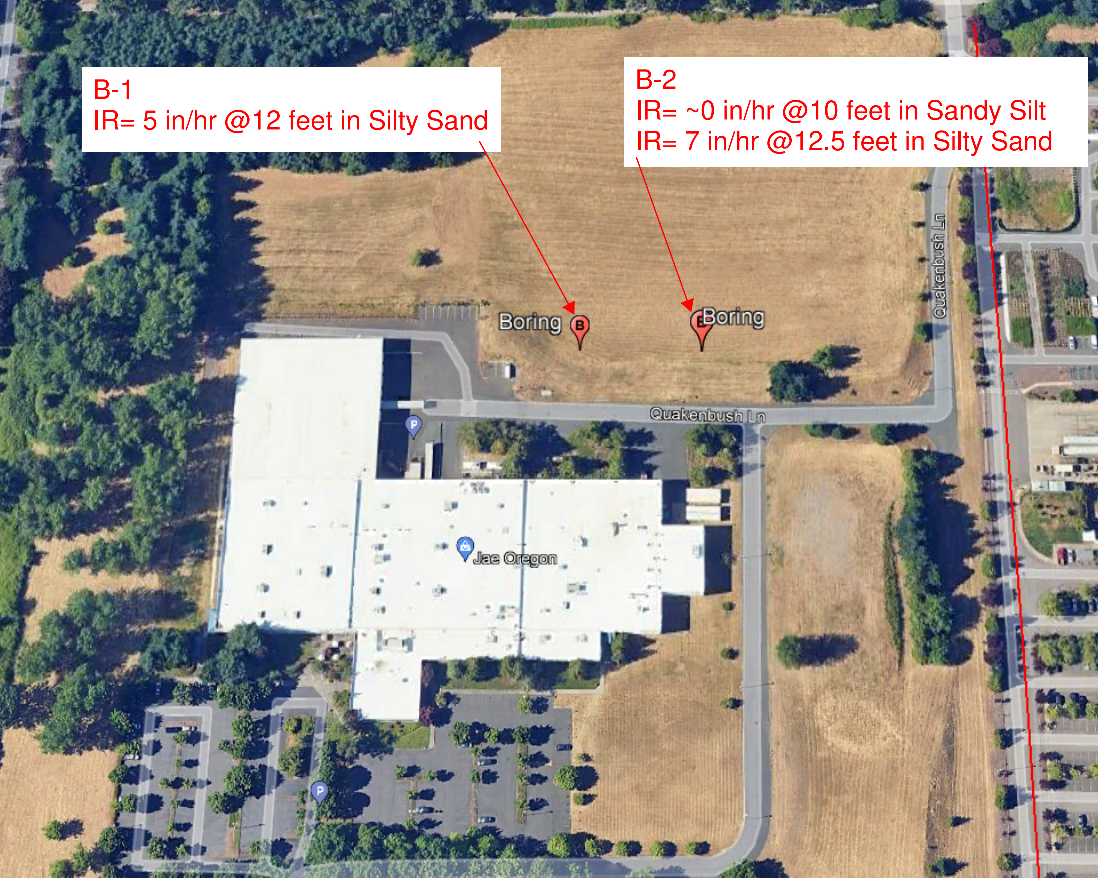
NOTES:
1. SITE LOCATION: 11361 SW LEVETON DRIVE, TUALATIN, OREGON.
2. AERIAL PHOTO SOURCED FROM GOOGLE EARTH.
3. EXPLORATION LOCATIONS ARE APPROXIMATE AND NOT SURVEYED.
4. BORINGS BACKFILLED WITH BENTONITE ON NOVEMBER 9, 10, 13, AND 14, 2023.
5. CONE PENETRATION TEST BACKFILLED WITH BENTONITE ON NOVEMBER 10, 2023.

FIGURE
2

Table 1. Groundwater Depths and Elevations Summary

Location	Measurement Date	Groundwater Depth (feet BGS)	Groundwater Elevation
CPT-1	11/10/23	>51[1]	<126
CPT-2	11/10/23	48.9[1]	126.6
CPT-3	11/10/23	39.5[1]	126.5
CPT-4	11/10/23	36.5[1]	121
CPT-5	11/10/23	37.1[1]	124.4
CPT-6	11/10/23	46.8[1]	123.7
B-7 (P-1)	12/5/23	>39[2]	<127.5
B-7 (P-1)	1/25/24	38.5	128
B-7 (P-1)	2/29/24	38.6	127.9
B-8 (P-2)	12/5/23	>39[2]	<131
B-8 (P-2)	1/23/24	38	132
B-8 (P-2)	1/25/24	37.9	132.1
B-8 (P-2)	2/26/24	38	132

1. Groundwater depths were inferred from pore water pressure dissipation tests in the CPTs
2. Groundwater depths based on piezometers measurements



B-1
IR= 5 in/hr @12 feet in Silty Sand
B-2
IR= ~0 in/hr @10 feet in Sandy Silt
IR= 7 in/hr @12.5 feet in Silty Sand
Boring
Boring
Quakenbush Ln
Quakenbush Ln
Jae Oregon