Exhibit A.14

MACKENZIE.

DESIGN DRIVEN | CLIENT FOCUSED

# STORM WATER REPORT

**To**
Multnomah County

**For**
Oregon Department of Transportation
Skyline Road Stockpile Facility
Tax lot 2500, Map 2N-1W-31C

**Submitted**
January 19, 2021

**Project Number**
2200357.00


MACKENZIE
Since 1960

RiverEast Center | 1515 SE Water Ave, Suite 100, Portland, OR 97214
PO Box 14310, Portland, OR 97293 | T 503.224.9560 | www.mcknze.com

## TABLE OF CONTENTS



## APPENDICES

## DESIGNER'S CERTIFICATION

I hereby certify that this Stormwater Report for the Oregon Department of Transportation Stockpile Facility has been prepared by me or under my supervision and meets minimum standards of Multnomah County and normal standards of engineering practice. I hereby acknowledge and agree that the jurisdiction does not and will not assume liability for the sufficiency, suitability, or performance of drainage facilities designed by me.

## I. SITE OVERVIEW AND DESCRIPTION

This report documents the stormwater management calculations and design by Mackenzie to manage stormwater runoff and provide water quality treatment for The Oregon Department of Transportation (ODOT) Stockpile facility. The project proposes the development of the lot in the southeast corner of the intersection of SW Skyline Boulevard and NW Cornelius Pass Road. The new facility will serve as a as a storage and stockpile area for roadway deicing equipment and supplies. The total site area is 2.39 acres. The site area impacted by the project is 1.11 acres. The site contains 1 building and a yard, half comprised of gravel surfacing and the other half asphalt surfacing. The development sits on top of a small hill with an approximate 480’ long driveway connecting to NW Skyline Blvd.

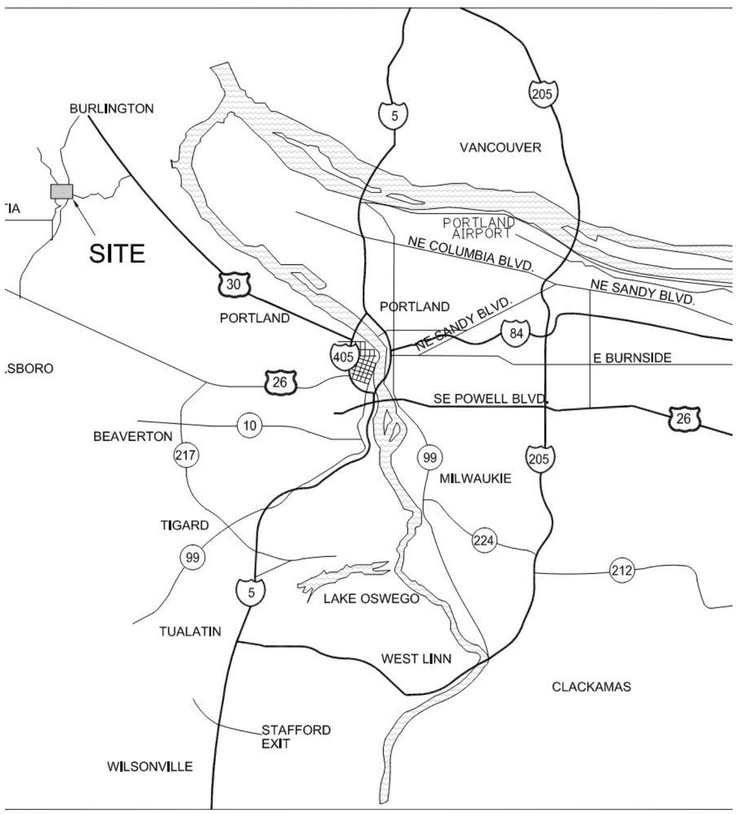



**Figure 1: Vicinity Map**

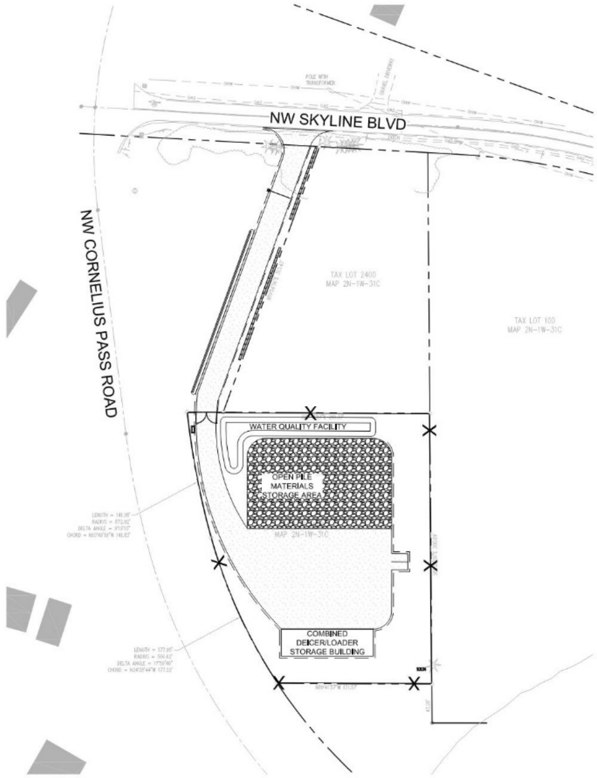



**Figure 2: Site Map**

## II. FACILITY DESIGN

The site lies in Multnomah County and follows Multnomah County Design Standards for projects and drainage systems within the right-of-way (ROW). This includes standards for water allowed entering or leaving the ROW. Since the project develops or redevelops more than 500 sf of impervious surface, it is required to comply with stormwater management practices.

The County requires flow control that limits post-development peak runoff rates to pre-development rates for the 5-, 10-, and 25-year events, as well as one-half the 2-year event before any development. For water quality facilities the County follows the City of Portland 2020 Stormwater Management Manual (SWMM). Pollution reduction requirement much achieve 70% total suspended solids (TSS) removal from the runoff resulting from 90% of the average annual rainfall (1.61 inches over 24 hours, or 0.19 in/hr for a time of concentration of 5-minutes).

The City of Portland requires stormwater detention and infiltration facilities to provide enough storage to retain and manage the 10-year stormwater and have a tested infiltration rate of at least 2 in/hr. Facilities that cannot achieve total onsite infiltration and that discharge offsite to the separated stormwater system must meet Level 2 requirements of the stormwater hierarchy.

Since the project eventually discharges to the Willamette River, hydromodification is not required.

Calculations for conveyance were performed using the Rational Method and meet Multnomah County minimum pipe sizing requirements.

The project is located in Zone 7. Rainfall intensity was gathered from IDF curves for Zone 7.

| Storm event (24 hour) | Rainfall Intensity |
|---|---|
| WQ | 1.61 |
| 2-year | 2.4 |
| 5-year | 2.9 |
| 10-year | 3.4 |
| 25-year | 3.8 |

Water quality and detention requirements for the development are met by collecting and conveying stormwater to an on-site planter designed to BES detail SW-241 – "Basin with underdrain" prior to discharge off-site. The basin is sloped at 0.5%. For purpose of design, we used a bottom of facility design elevation of 100.00' to design depth of storm water.

The storm facilities were designed using HydroFlow following the City's Performance Approach. See table below for summary. See Appendix C for reports.

| Basin Data | |
|---|---|
| Width | 21 ft |
| Depth | 2.05 ft |
| Bottom Width | 9 ft |

| | |
|---|---|
| Bottom Slope | 0.5 |
| Side Slope | 3H:1V |
| Growing Medium Depth | 24 in |
| Rock Depth | 12 in |
| Freeboard | 3.48 in |
| Storage Depth | 20.88 in |

See Table below for storm event mitigation summary:

| Storm event (24-hour) | Pre-Development Peak Flow (cfs) | Post-Devlopment Peak Flow (cfs) | Water Elevation (ft) |
|---|---|---|---|
| Water Quality | 0.035 | 0.035 | 588.85 |
| 2-year | 0.154 | 0.078 | 589.11 |
| 5-year | 0.247 | 0.120 | 589.23 |
| 10-year | 0.349 | 0.183 | 589.33 |
| 25-year | 0.459 | 0.253 | 589.44 |

See table below for orifice sizing:

| Orifice | Size (diameter in inches) | Elevation |
|---|---|---|
| 1 | 1.52 | 587.70 |
| 2 | 2.22 | 588.86 |
| 3 | 4 | 589.12 |

The top of basin elevation is 189.73, allowing 3.48 in. of freeboard.

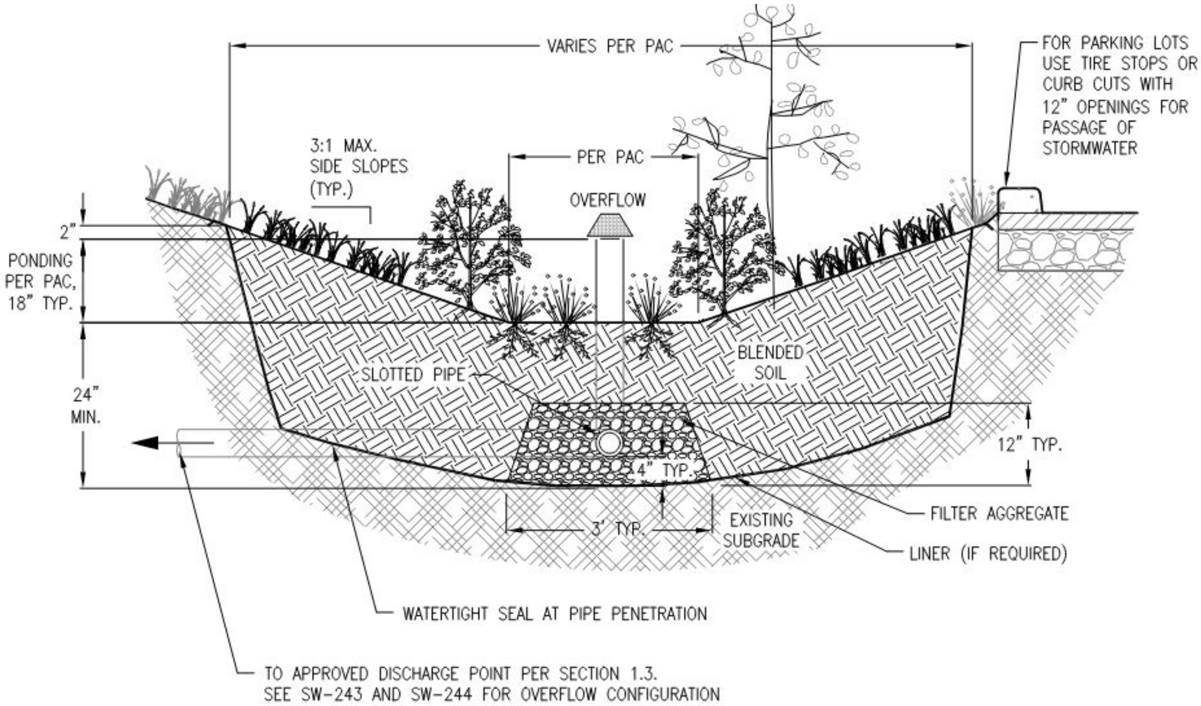
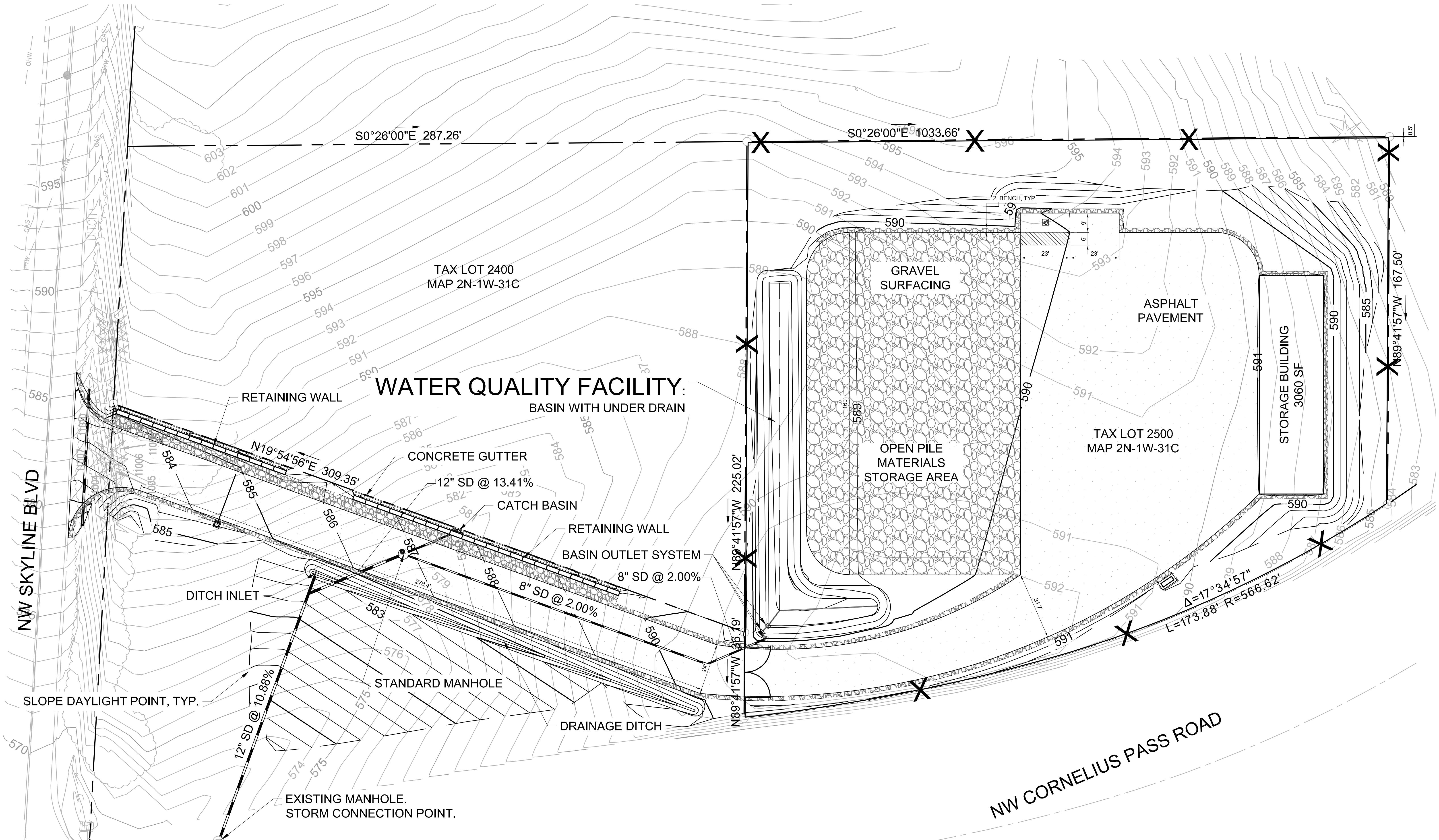



**Figure 3: Bureau of Envirnomental Service standard detail SW- 231, " Planter – Unlined"**




**OREGO DEPARTMENT OF TRANSPORTATION**
**SKYLINE STOCKPILE**
01.18.2021

**APPENDIX A - FACILITY PLAN**
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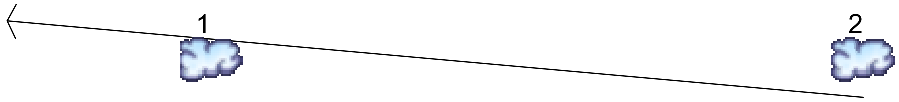
# Hydraflow Table of Contents

# Watershed Model Schematic

Hydraflow Hydrographs Extension for Autodesk® Civil 3D® 2019 by Autodesk, Inc. v2020




**Legend**

| **Hyd.** | **Origin** | **Description** |
|---|---|---|
| 1 | SCS Runoff | Pre Development |
| 2 | SCS Runoff | Post development |
| 3 | Reservoir | Basin with Underdrain |

# Hydrograph Summary Report

Hydraflow Hydrographs Extension for Autodesk® Civil 3D® 2019 by Autodesk, Inc. v2020

| Hyd. No. | Hydrograph type (origin) | Peak flow (cfs) | Time interval (min) | Time to Peak (min) | Hyd. volume (cuft) | Inflow hyd(s) | Maximum elevation (ft) | Total strge used (cuft) | Hydrograph Description |
|---|---|---|---|---|---|---|---|---|---|
| 1 | SCS Runoff | 0.035 | 2 | 480 | 1,176 | ------ | ------ | ------ | Pre Development |
| 2 | SCS Runoff | 0.378 | 2 | 472 | 5,245 | ------ | ------ | ------ | Post development |
| 3 | Reservoir | 0.035 | 2 | 1360 | 5,223 | 2 | 101.15 | 3,121 | Basin with Underdrain |

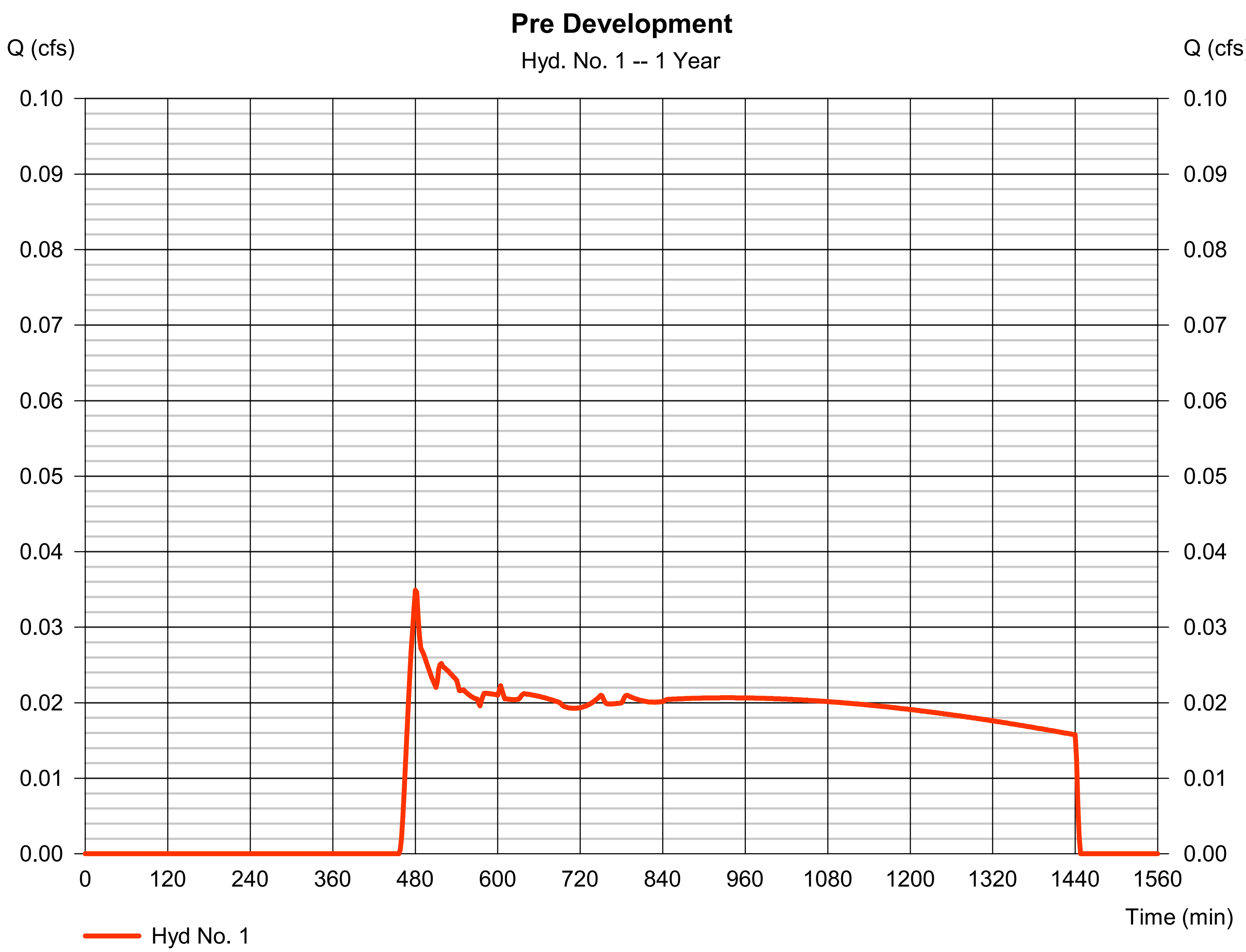
# Hydrograph Report

Hydraflow Hydrographs Extension for Autodesk® Civil 3D® 2019 by Autodesk, Inc. v2020 | Thursday, 01 / 14 / 2021

## Hyd. No. 1

Pre Development

| | | | |
|---|---|---|---|
| Hydrograph type | = SCS Runoff | Peak discharge | = 0.035 cfs |
| Storm frequency | = 1 yrs | Time to peak | = 480 min |
| Time interval | = 2 min | Hyd. volume | = 1,176 cuft |
| Drainage area | = 1.110 ac | Curve number | = 79 |
| Basin Slope | = 0.0 % | Hydraulic length | = 0 ft |
| Tc method | = User | Time of conc. (Tc) | = 5.00 min |
| Total precip. | = 1.61 in | Distribution | = Type IA |
| Storm duration | = 24 hrs | Shape factor | = 484 |



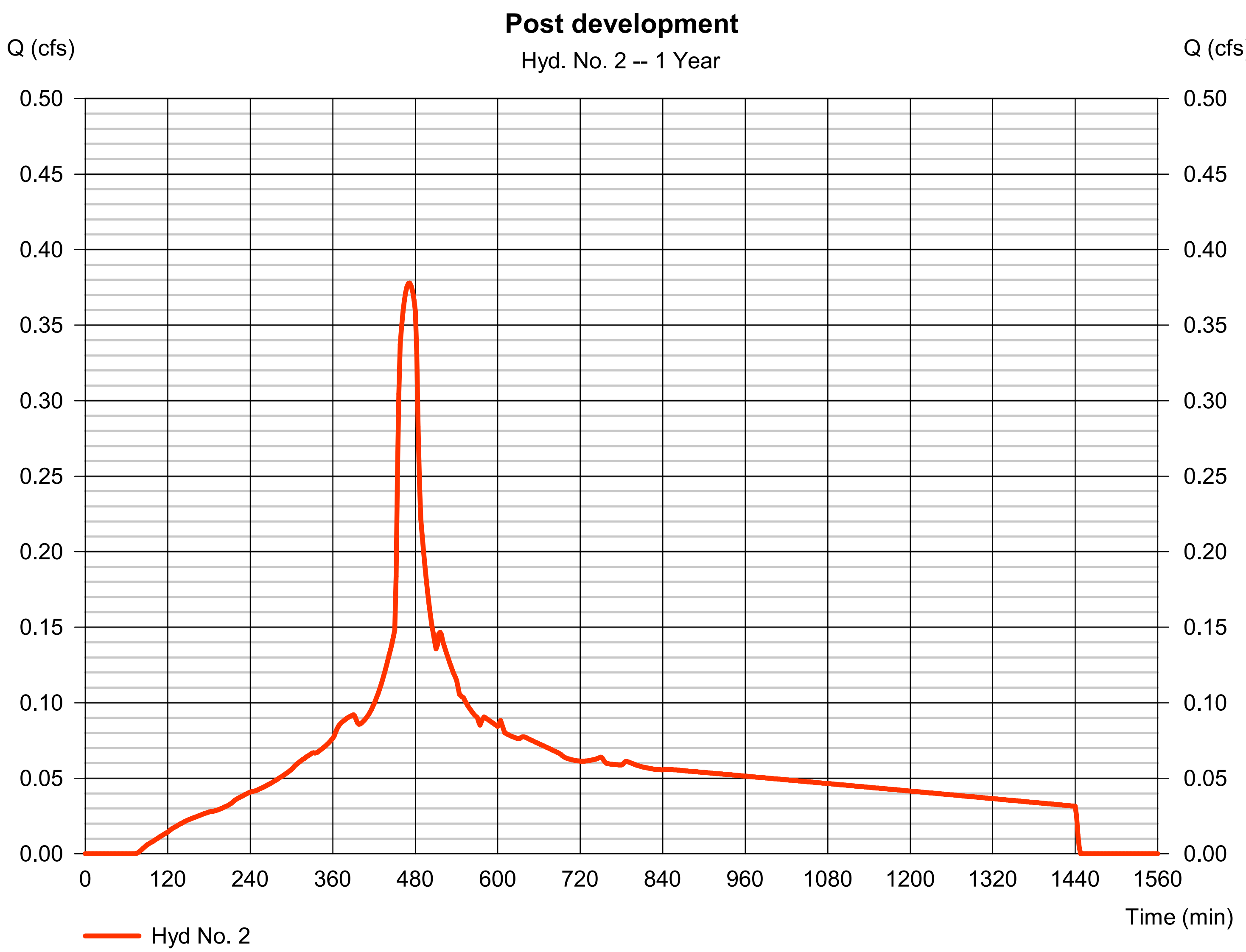
# Hydrograph Report

Hydraflow Hydrographs Extension for Autodesk® Civil 3D® 2019 by Autodesk, Inc. v2020 Thursday, 01 / 14 / 2021

## Hyd. No. 2

Post development

| | | | |
|---|---|---|---|
| Hydrograph type | = SCS Runoff | Peak discharge | = 0.378 cfs |
| Storm frequency | = 1 yrs | Time to peak | = 472 min |
| Time interval | = 2 min | Hyd. volume | = 5,245 cuft |
| Drainage area | = 1.110 ac | Curve number | = 98 |
| Basin Slope | = 0.0 % | Hydraulic length | = 0 ft |
| Tc method | = User | Time of conc. (Tc) | = 5.00 min |
| Total precip. | = 1.61 in | Distribution | = Type IA |
| Storm duration | = 24 hrs | Shape factor | = 484 |



# Hydrograph Report

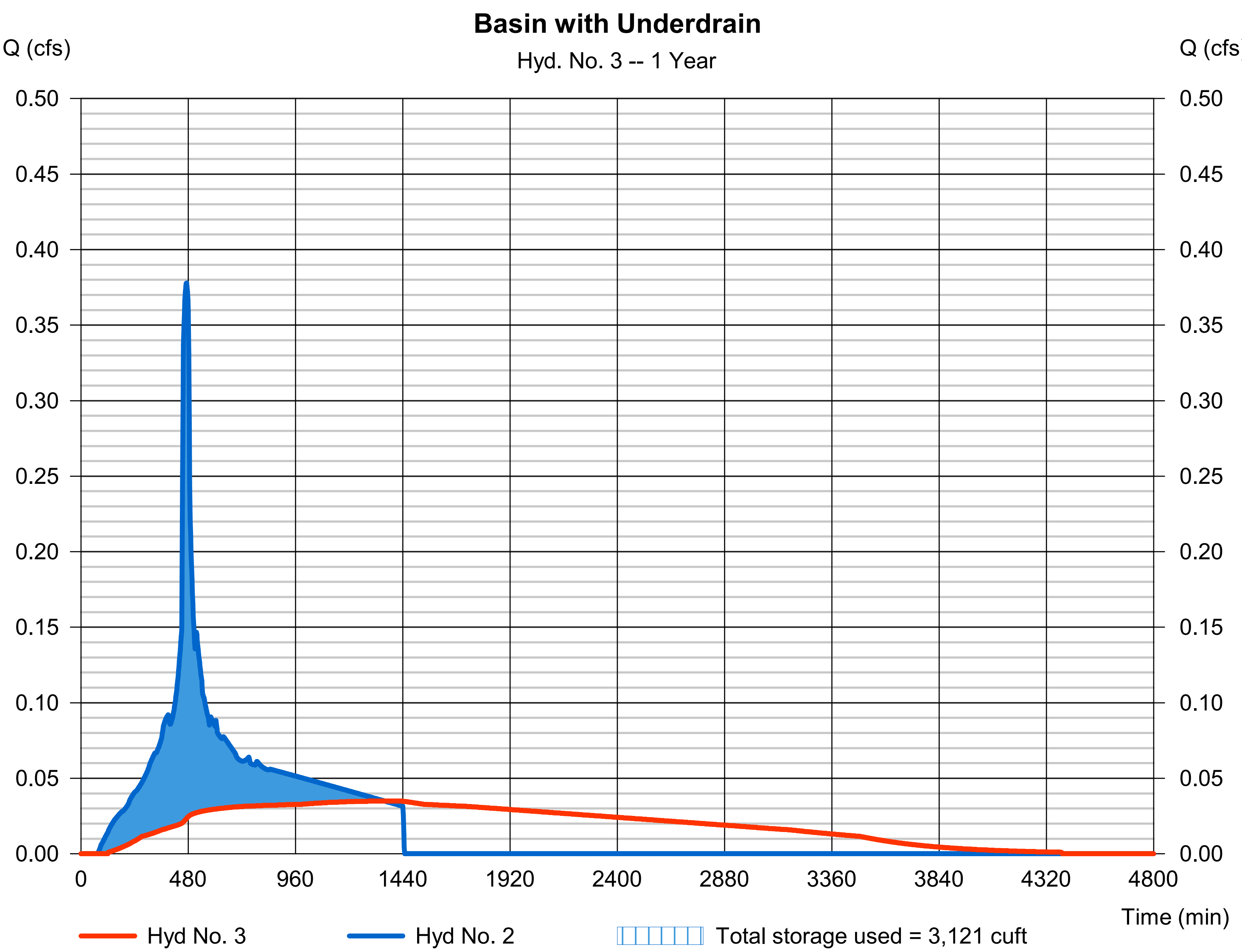
Hydraflow Hydrographs Extension for Autodesk® Civil 3D® 2019 by Autodesk, Inc. v2020 Thursday, 01 / 14 / 2021

## Hyd. No. 3

Basin with Underdrain

| | | | |
|---|---|---|---|
| Hydrograph type | = Reservoir | Peak discharge | = 0.035 cfs |
| Storm frequency | = 1 yrs | Time to peak | = 1360 min |
| Time interval | = 2 min | Hyd. volume | = 5,223 cuft |
| Inflow hyd. No. | = 2 - Post development | Max. Elevation | = 101.15 ft |
| Reservoir name | = Basin | Max. Storage | = 3,121 cuft |

Storage Indication method used.



# Pond Report

Hydraflow Hydrographs Extension for Autodesk® Civil 3D® 2019 by Autodesk, Inc. v2020 Thursday, 01 / 14 / 2021

## Pond No. 1 - Basin

## Pond Data

**Contours** -User-defined contour areas. Conic method used for volume calculation. Begining Elevation = 100.00 ft

## Stage / Storage Table

| Stage (ft) | Elevation (ft) | Contour area (sqft) | Incr. Storage (cuft) | Total storage (cuft) |
|---|---|---|---|---|
| 0.00 | 100.00 | 1,947 | 0 | 0 |
| 1.00 | 101.00 | 3,200 | 2,547 | 2,547 |
| 2.00 | 102.00 | 4,514 | 3,838 | 6,385 |

## Culvert / Orifice Structures

| | [A] | [B] | [C] | [PrfRsr] |
|---|---|---|---|---|
| **Rise (in)** | = 1.52 | 2.22 | 4.00 | 0.00 |
| **Span (in)** | = 1.52 | 2.22 | 4.00 | 0.00 |
| **No. Barrels** | = 1 | 1 | 1 | 0 |
| **Invert El. (ft)** | = 100.00 | 101.16 | 101.42 | 0.00 |
| **Length (ft)** | = 20.00 | 20.00 | 20.00 | 0.00 |
| **Slope (%)** | = 1.00 | 1.00 | 1.00 | n/a |
| **N-Value** | = .013 | .013 | .013 | n/a |
| **Orifice Coeff.** | = 0.60 | 0.60 | 0.60 | 0.60 |
| **Multi-Stage** | = n/a | No | No | No |

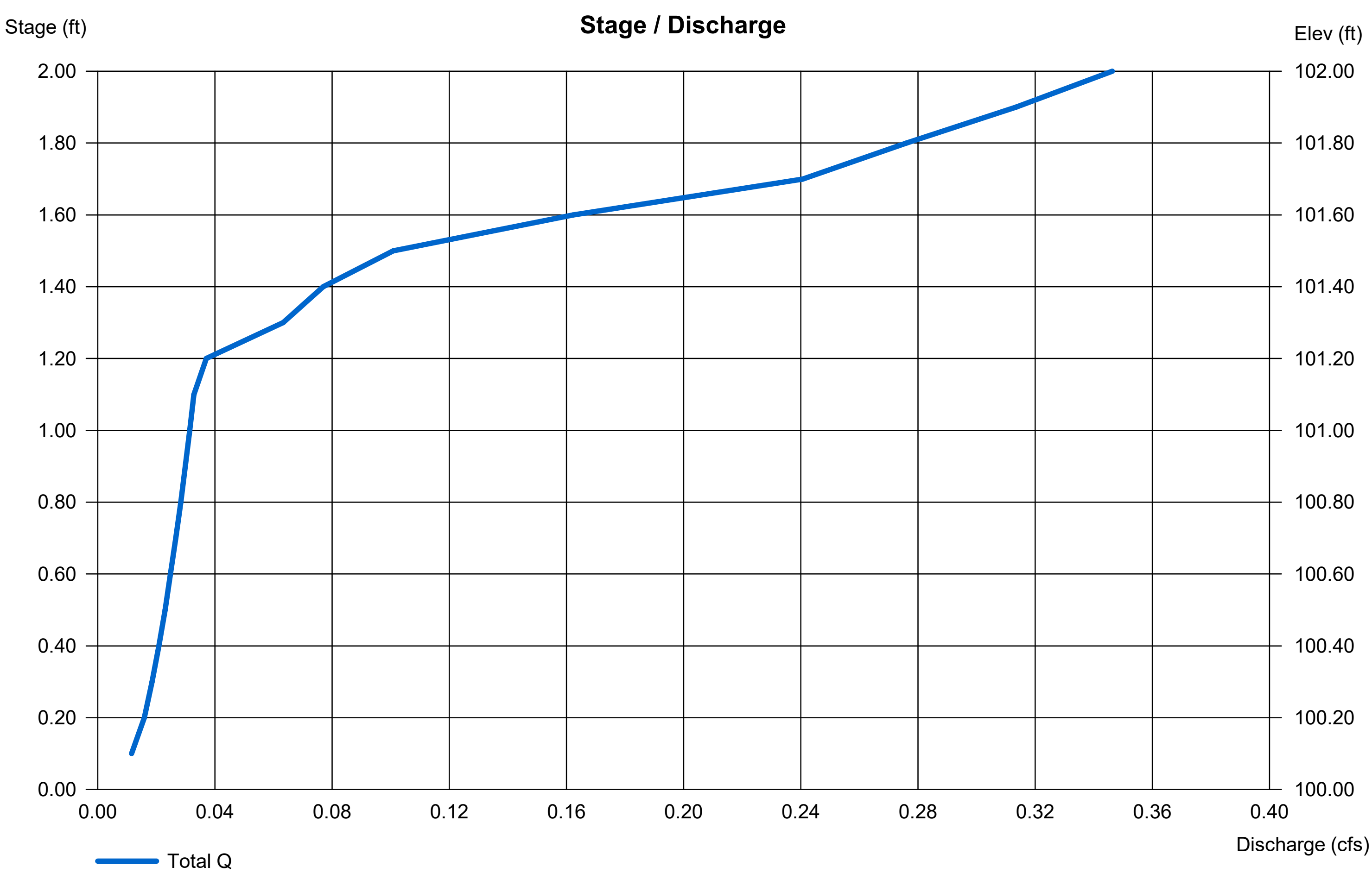
## Weir Structures

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

**Exfil.(in/hr)** = 0.000 (by Contour)

**TW Elev. (ft)** = 0.00

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



# Hydrograph Summary Report

Hydraflow Hydrographs Extension for Autodesk® Civil 3D® 2019 by Autodesk, Inc. v2020

| Hyd. No. | Hydrograph type (origin) | Peak flow (cfs) | Time interval (min) | Time to Peak (min) | Hyd. volume (cuft) | Inflow hyd(s) | Maximum elevation (ft) | Total strge used (cuft) | Hydrograph Description |
|---|---|---|---|---|---|---|---|---|---|
| 1 | SCS Runoff | 0.154 | 2 | 480 | 2,913 | ------ | ------ | ------ | Pre Development |
| 2 | SCS Runoff | 0.584 | 2 | 470 | 8,202 | ------ | ------ | ------ | Post development |
| 3 | Reservoir | 0.078 | 2 | 944 | 8,180 | 2 | 101.41 | 4,107 | Basin with Underdrain |

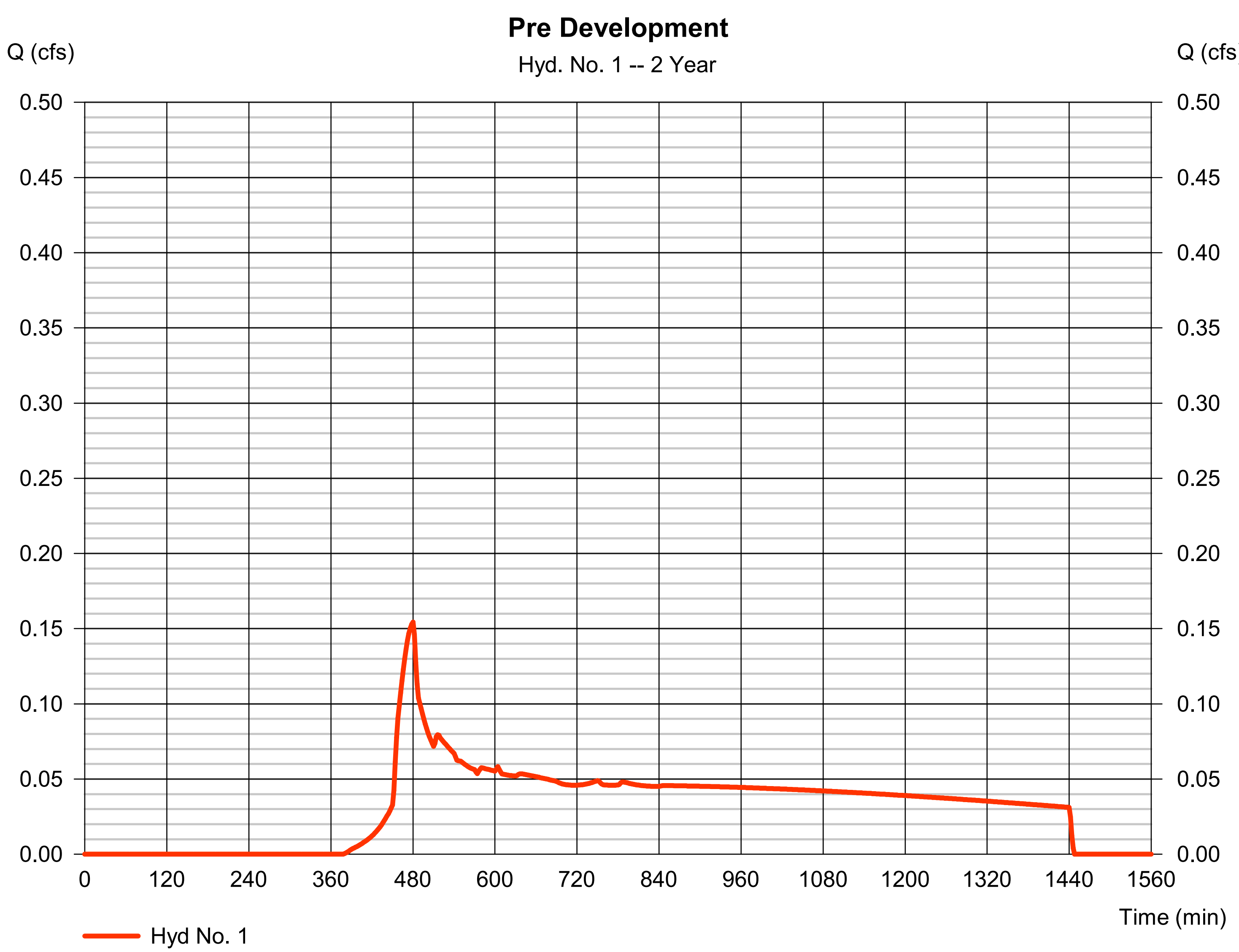
# Hydrograph Report

Hydraflow Hydrographs Extension for Autodesk® Civil 3D® 2019 by Autodesk, Inc. v2020 Thursday, 01 / 14 / 2021

## Hyd. No. 1

Pre Development

| | | | |
|---|---|---|---|
| Hydrograph type | = SCS Runoff | Peak discharge | = 0.154 cfs |
| Storm frequency | = 2 yrs | Time to peak | = 480 min |
| Time interval | = 2 min | Hyd. volume | = 2,913 cuft |
| Drainage area | = 1.110 ac | Curve number | = 79 |
| Basin Slope | = 0.0 % | Hydraulic length | = 0 ft |
| Tc method | = User | Time of conc. (Tc) | = 5.00 min |
| Total precip. | = 2.40 in | Distribution | = Type IA |
| Storm duration | = 24 hrs | Shape factor | = 484 |



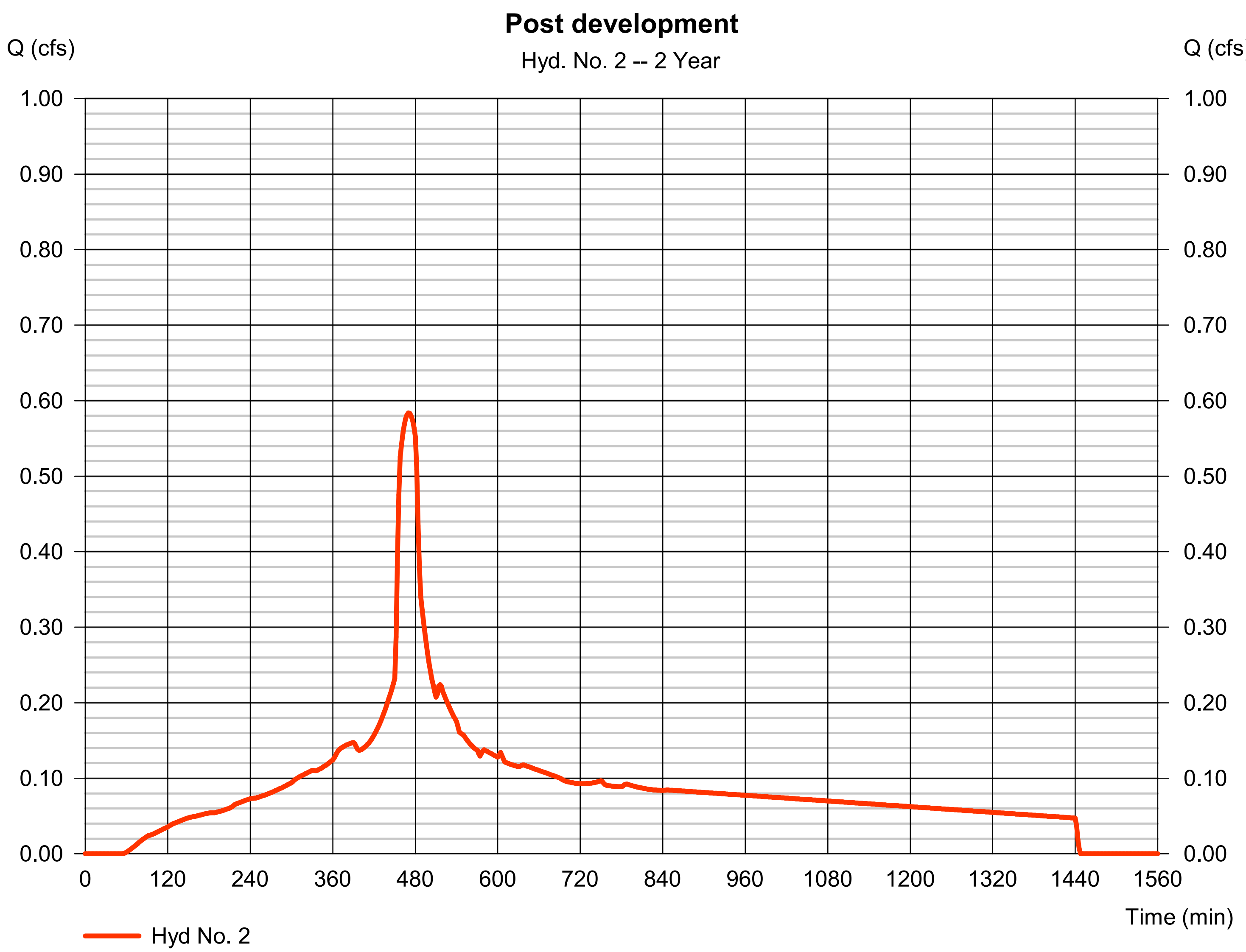
# Hydrograph Report



## Hyd. No. 2

Post development

| | | | |
|---|---|---|---|
| Hydrograph type | = SCS Runoff | Peak discharge | = 0.584 cfs |
| Storm frequency | = 2 yrs | Time to peak | = 470 min |
| Time interval | = 2 min | Hyd. volume | = 8,202 cuft |
| Drainage area | = 1.110 ac | Curve number | = 98 |
| Basin Slope | = 0.0 % | Hydraulic length | = 0 ft |
| Tc method | = User | Time of conc. (Tc) | = 5.00 min |
| Total precip. | = 2.40 in | Distribution | = Type IA |
| Storm duration | = 24 hrs | Shape factor | = 484 |



# Hydrograph Report

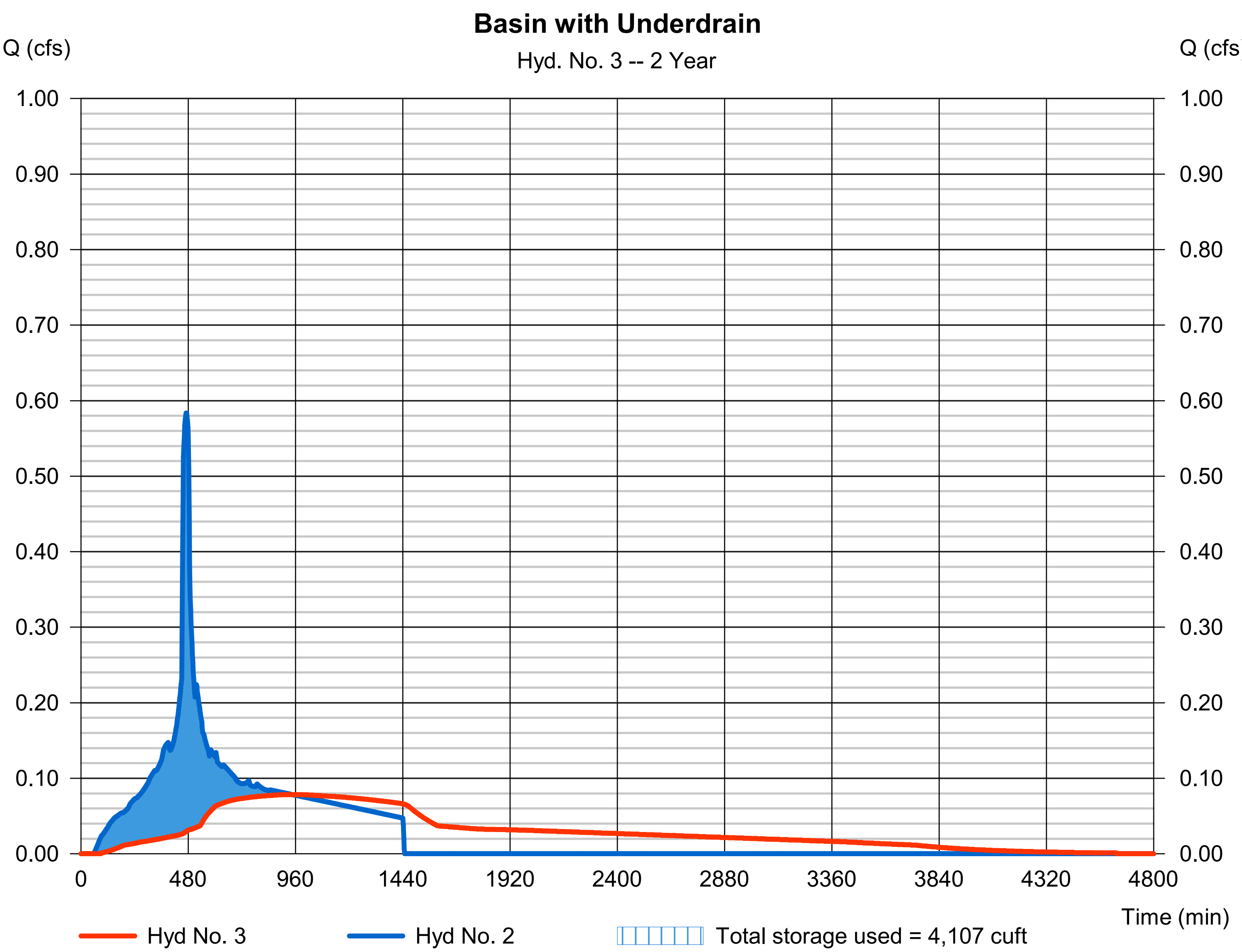
Hydraflow Hydrographs Extension for Autodesk® Civil 3D® 2019 by Autodesk, Inc. v2020 Thursday, 01 / 14 / 2021

## Hyd. No. 3

Basin with Underdrain

| | | | |
|---|---|---|---|
| Hydrograph type | = Reservoir | Peak discharge | = 0.078 cfs |
| Storm frequency | = 2 yrs | Time to peak | = 944 min |
| Time interval | = 2 min | Hyd. volume | = 8,180 cuft |
| Inflow hyd. No. | = 2 - Post development | Max. Elevation | = 101.41 ft |
| Reservoir name | = Basin | Max. Storage | = 4,107 cuft |

Storage Indication method used.



# Hydrograph Summary Report

Hydraflow Hydrographs Extension for Autodesk® Civil 3D® 2019 by Autodesk, Inc. v2020

| Hyd. No. | Hydrograph type (origin) | Peak flow (cfs) | Time interval (min) | Time to Peak (min) | Hyd. volume (cuft) | Inflow hyd(s) | Maximum elevation (ft) | Total strge used (cuft) | Hydrograph Description |
|---|---|---|---|---|---|---|---|---|---|
| 1 | SCS Runoff | 0.247 | 2 | 480 | 4,215 | ------ | ------ | ------ | Pre Development |
| 2 | SCS Runoff | 0.713 | 2 | 470 | 10,081 | ------ | ------ | ------ | Post development |
| 3 | Reservoir | 0.120 | 2 | 692 | 10,059 | 2 | 101.53 | 4,586 | Basin with Underdrain |

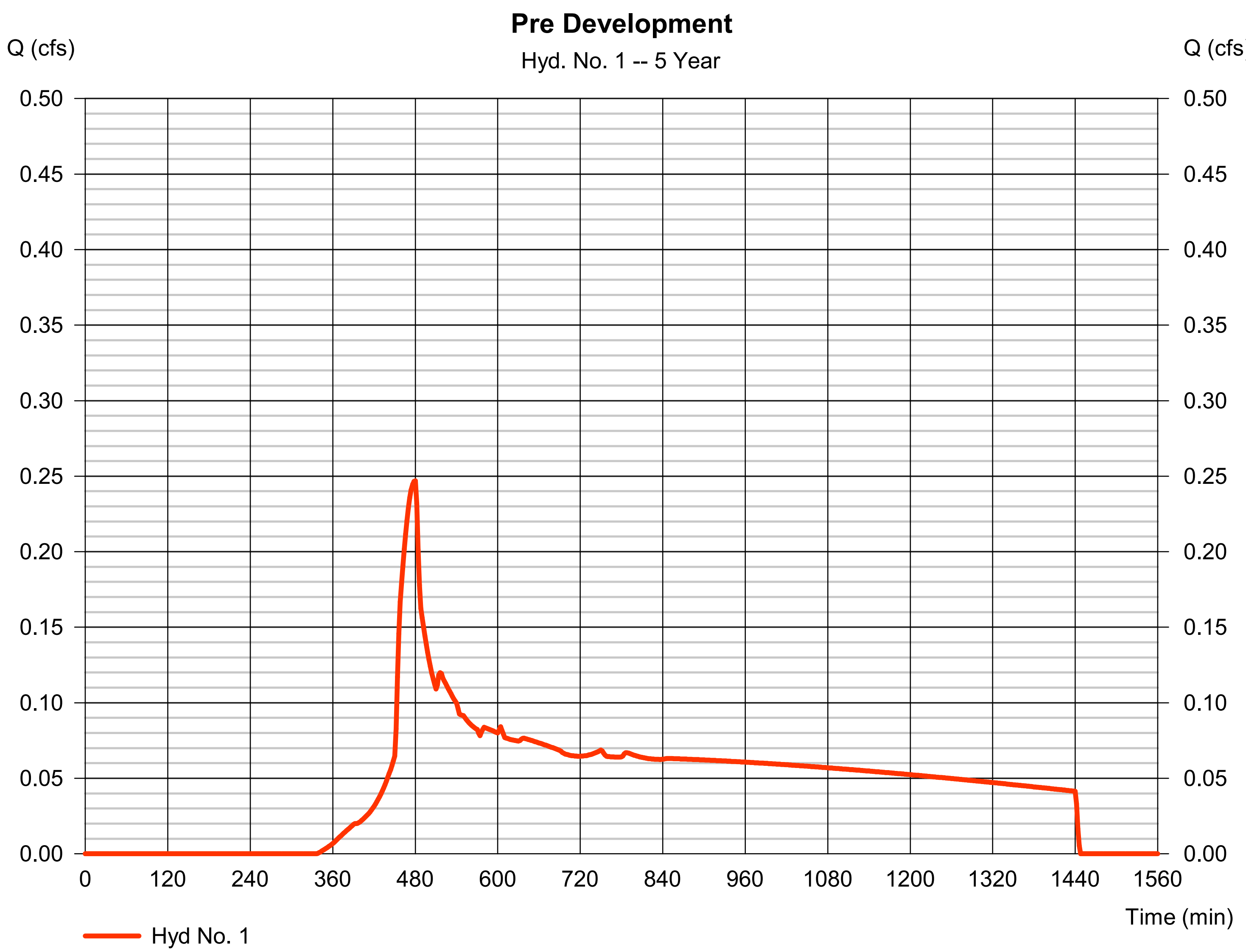
# Hydrograph Report

Hydraflow Hydrographs Extension for Autodesk® Civil 3D® 2019 by Autodesk, Inc. v2020 Thursday, 01 / 14 / 2021

## Hyd. No. 1

Pre Development

| | | | |
|---|---|---|---|
| Hydrograph type | = SCS Runoff | Peak discharge | = 0.247 cfs |
| Storm frequency | = 5 yrs | Time to peak | = 480 min |
| Time interval | = 2 min | Hyd. volume | = 4,215 cuft |
| Drainage area | = 1.110 ac | Curve number | = 79 |
| Basin Slope | = 0.0 % | Hydraulic length | = 0 ft |
| Tc method | = User | Time of conc. (Tc) | = 5.00 min |
| Total precip. | = 2.90 in | Distribution | = Type IA |
| Storm duration | = 24 hrs | Shape factor | = 484 |



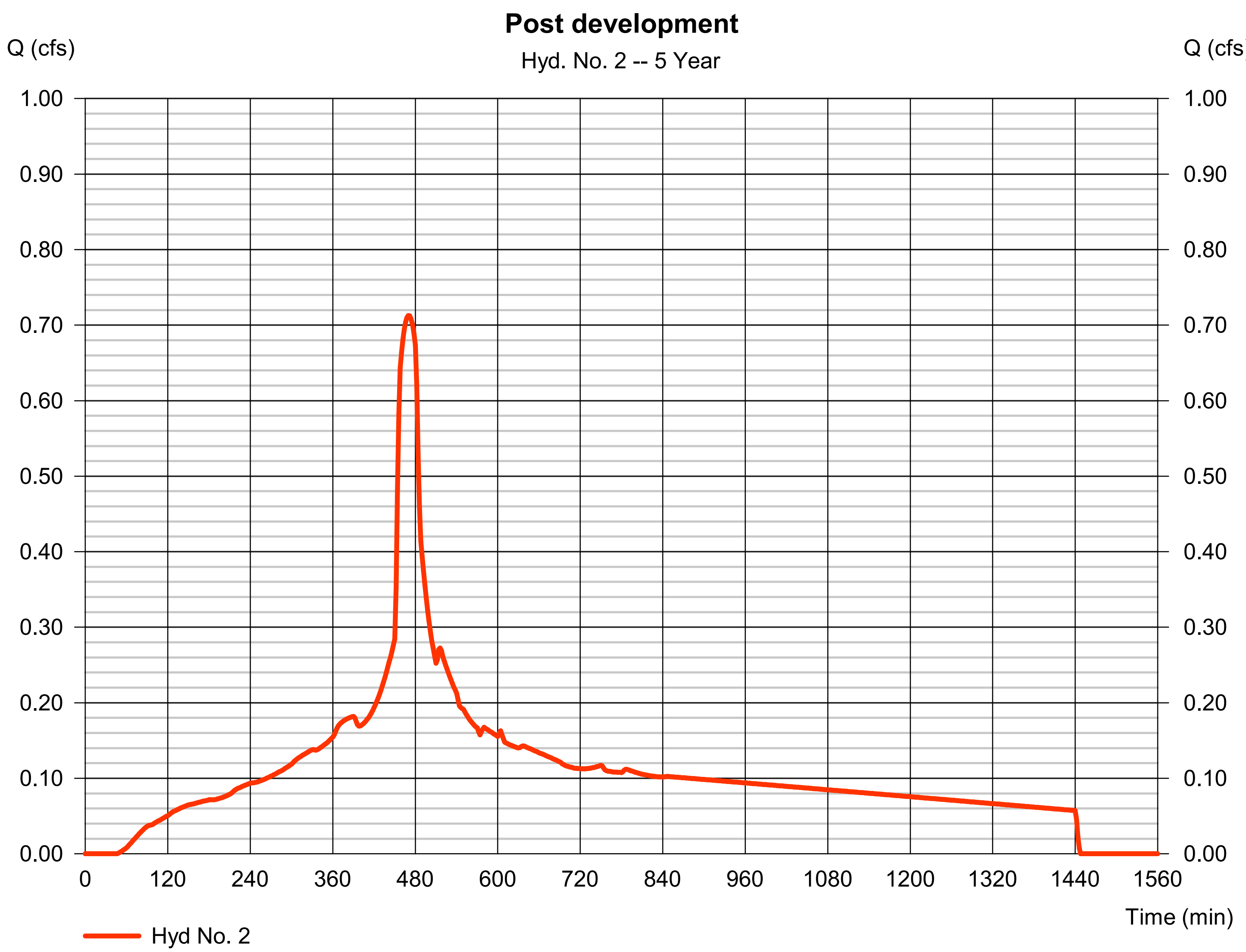
# Hydrograph Report

Hydraflow Hydrographs Extension for Autodesk® Civil 3D® 2019 by Autodesk, Inc. v2020 Thursday, 01 / 14 / 2021

## Hyd. No. 2

Post development

| | | | |
|---|---|---|---|
| Hydrograph type | = SCS Runoff | Peak discharge | = 0.713 cfs |
| Storm frequency | = 5 yrs | Time to peak | = 470 min |
| Time interval | = 2 min | Hyd. volume | = 10,081 cuft |
| Drainage area | = 1.110 ac | Curve number | = 98 |
| Basin Slope | = 0.0 % | Hydraulic length | = 0 ft |
| Tc method | = User | Time of conc. (Tc) | = 5.00 min |
| Total precip. | = 2.90 in | Distribution | = Type IA |
| Storm duration | = 24 hrs | Shape factor | = 484 |



# Hydrograph Report

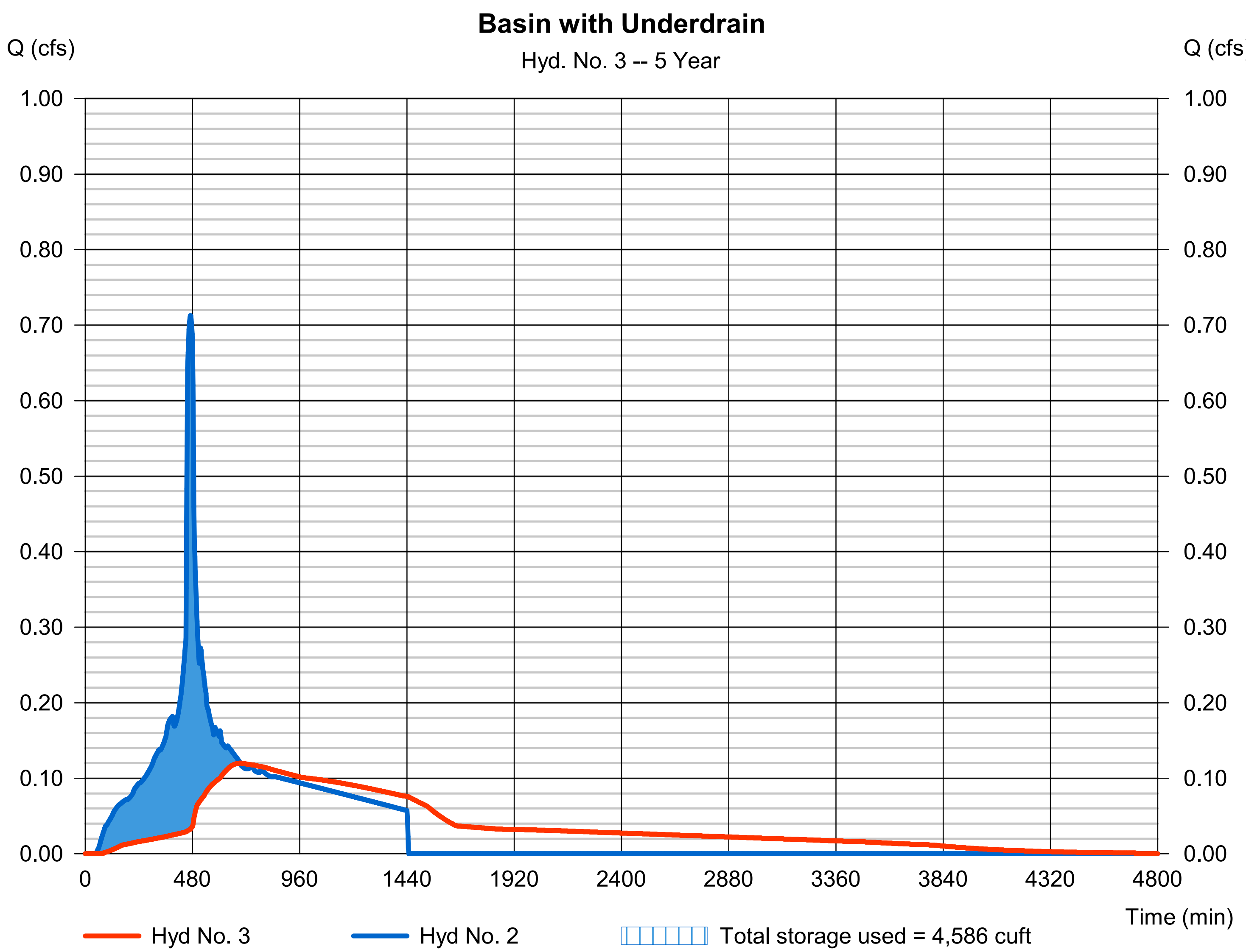
Hydraflow Hydrographs Extension for Autodesk® Civil 3D® 2019 by Autodesk, Inc. v2020 Thursday, 01 / 14 / 2021

## Hyd. No. 3

Basin with Underdrain

| | | | |
|---|---|---|---|
| Hydrograph type | = Reservoir | Peak discharge | = 0.120 cfs |
| Storm frequency | = 5 yrs | Time to peak | = 692 min |
| Time interval | = 2 min | Hyd. volume | = 10,059 cuft |
| Inflow hyd. No. | = 2 - Post development | Max. Elevation | = 101.53 ft |
| Reservoir name | = Basin | Max. Storage | = 4,586 cuft |

Storage Indication method used.



# Hydrograph Summary Report

Hydraflow Hydrographs Extension for Autodesk® Civil 3D® 2019 by Autodesk, Inc. v2020

| Hyd. No. | Hydrograph type (origin) | Peak flow (cfs) | Time interval (min) | Time to Peak (min) | Hyd. volume (cuft) | Inflow hyd(s) | Maximum elevation (ft) | Total strge used (cuft) | Hydrograph Description |
|---|---|---|---|---|---|---|---|---|---|
| 1 | SCS Runoff | 0.349 | 2 | 478 | 5,624 | ------ | ------ | ------ | Pre Development |
| 2 | SCS Runoff | 0.841 | 2 | 470 | 11,962 | ------ | ------ | ------ | Post development |
| 3 | Reservoir | 0.183 | 2 | 606 | 11,940 | 2 | 101.63 | 4,950 | Basin with Underdrain |

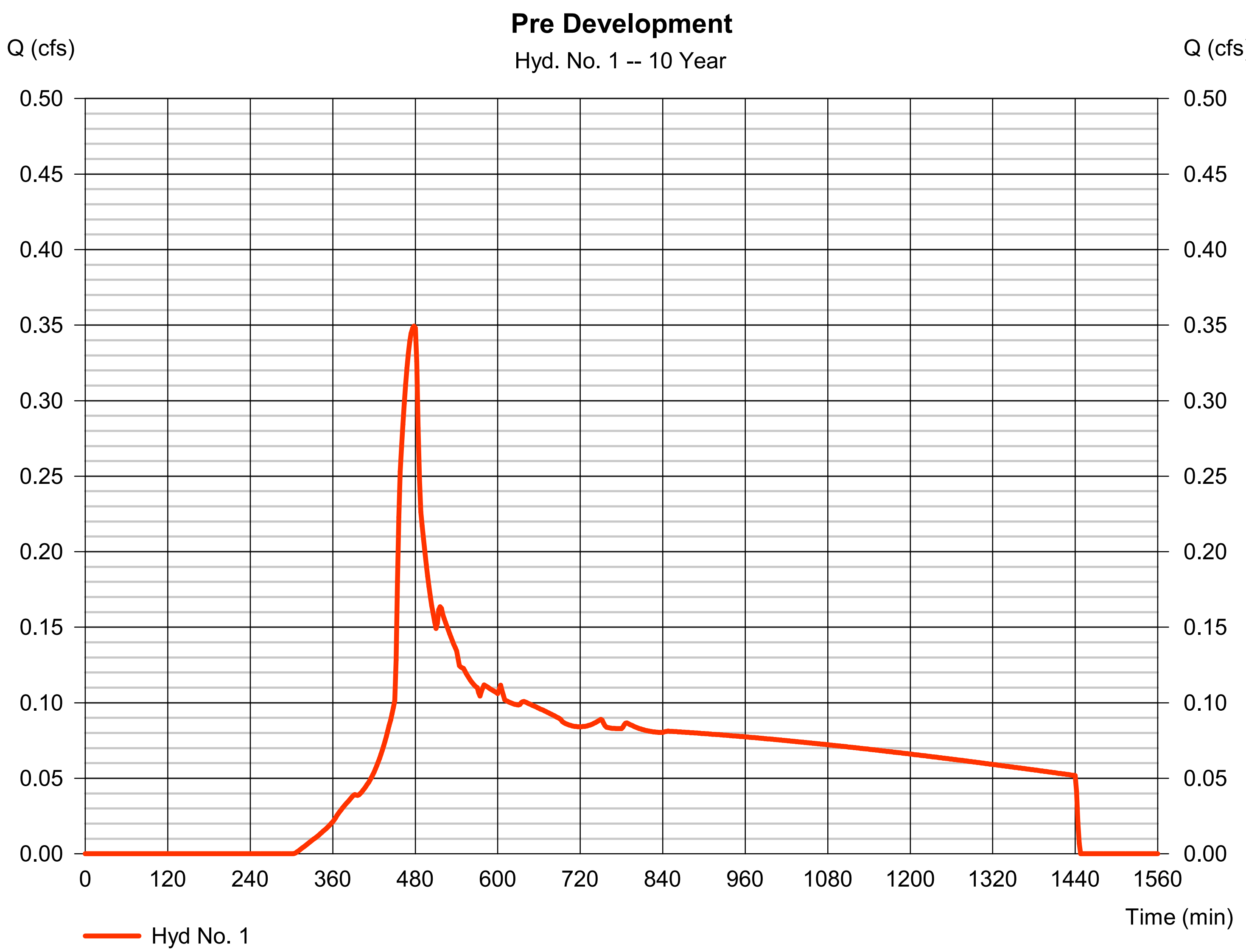
# Hydrograph Report

Hydraflow Hydrographs Extension for Autodesk® Civil 3D® 2019 by Autodesk, Inc. v2020 Thursday, 01 / 14 / 2021

## Hyd. No. 1

Pre Development

| | | | |
|---|---|---|---|
| Hydrograph type | = SCS Runoff | Peak discharge | = 0.349 cfs |
| Storm frequency | = 10 yrs | Time to peak | = 478 min |
| Time interval | = 2 min | Hyd. volume | = 5,624 cuft |
| Drainage area | = 1.110 ac | Curve number | = 79 |
| Basin Slope | = 0.0 % | Hydraulic length | = 0 ft |
| Tc method | = User | Time of conc. (Tc) | = 5.00 min |
| Total precip. | = 3.40 in | Distribution | = Type IA |
| Storm duration | = 24 hrs | Shape factor | = 484 |



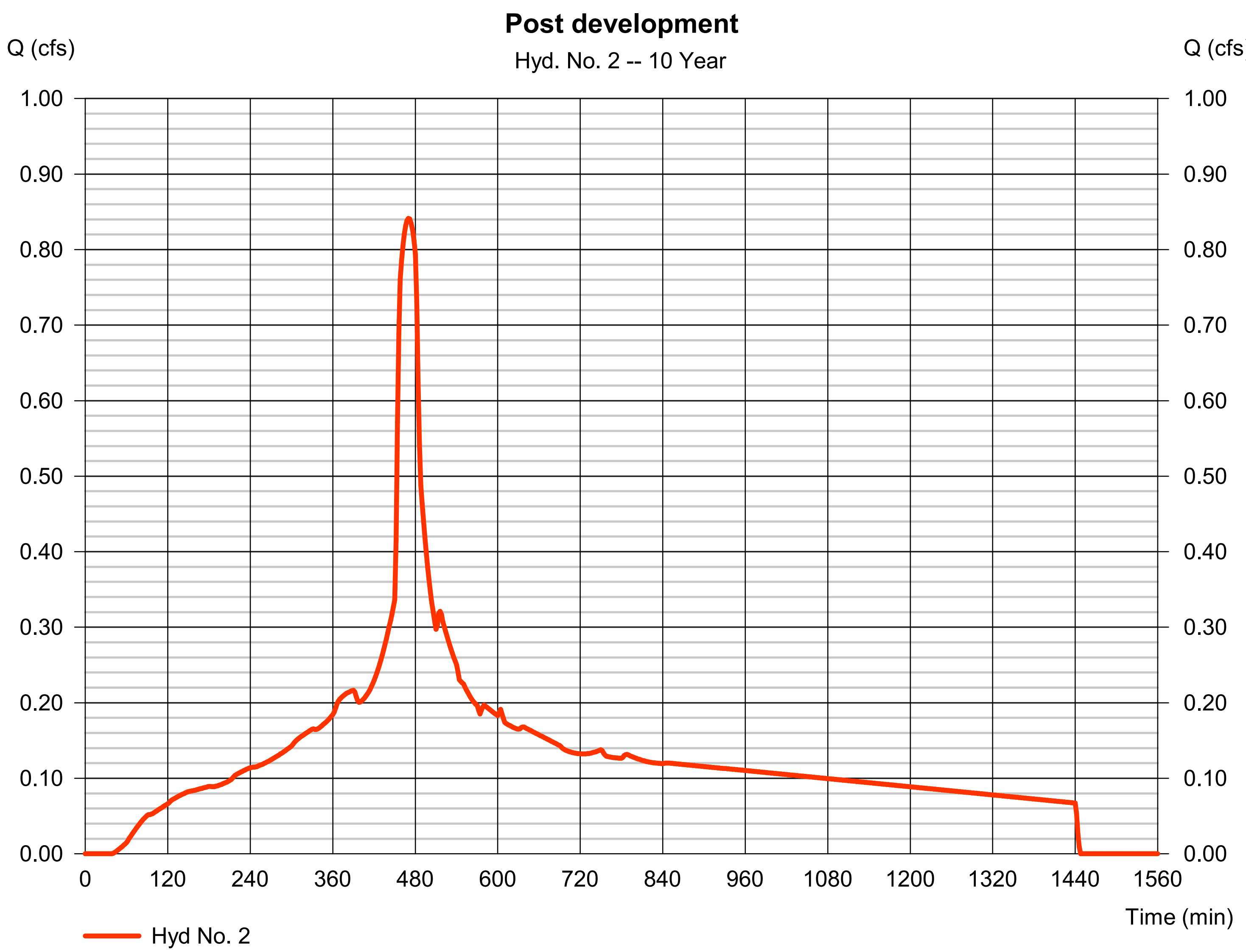
# Hydrograph Report

Hydraflow Hydrographs Extension for Autodesk® Civil 3D® 2019 by Autodesk, Inc. v2020 Thursday, 01 / 14 / 2021

## Hyd. No. 2

Post development

| | | | |
|---|---|---|---|
| Hydrograph type | = SCS Runoff | Peak discharge | = 0.841 cfs |
| Storm frequency | = 10 yrs | Time to peak | = 470 min |
| Time interval | = 2 min | Hyd. volume | = 11,962 cuft |
| Drainage area | = 1.110 ac | Curve number | = 98 |
| Basin Slope | = 0.0 % | Hydraulic length | = 0 ft |
| Tc method | = User | Time of conc. (Tc) | = 5.00 min |
| Total precip. | = 3.40 in | Distribution | = Type IA |
| Storm duration | = 24 hrs | Shape factor | = 484 |



# Hydrograph Report

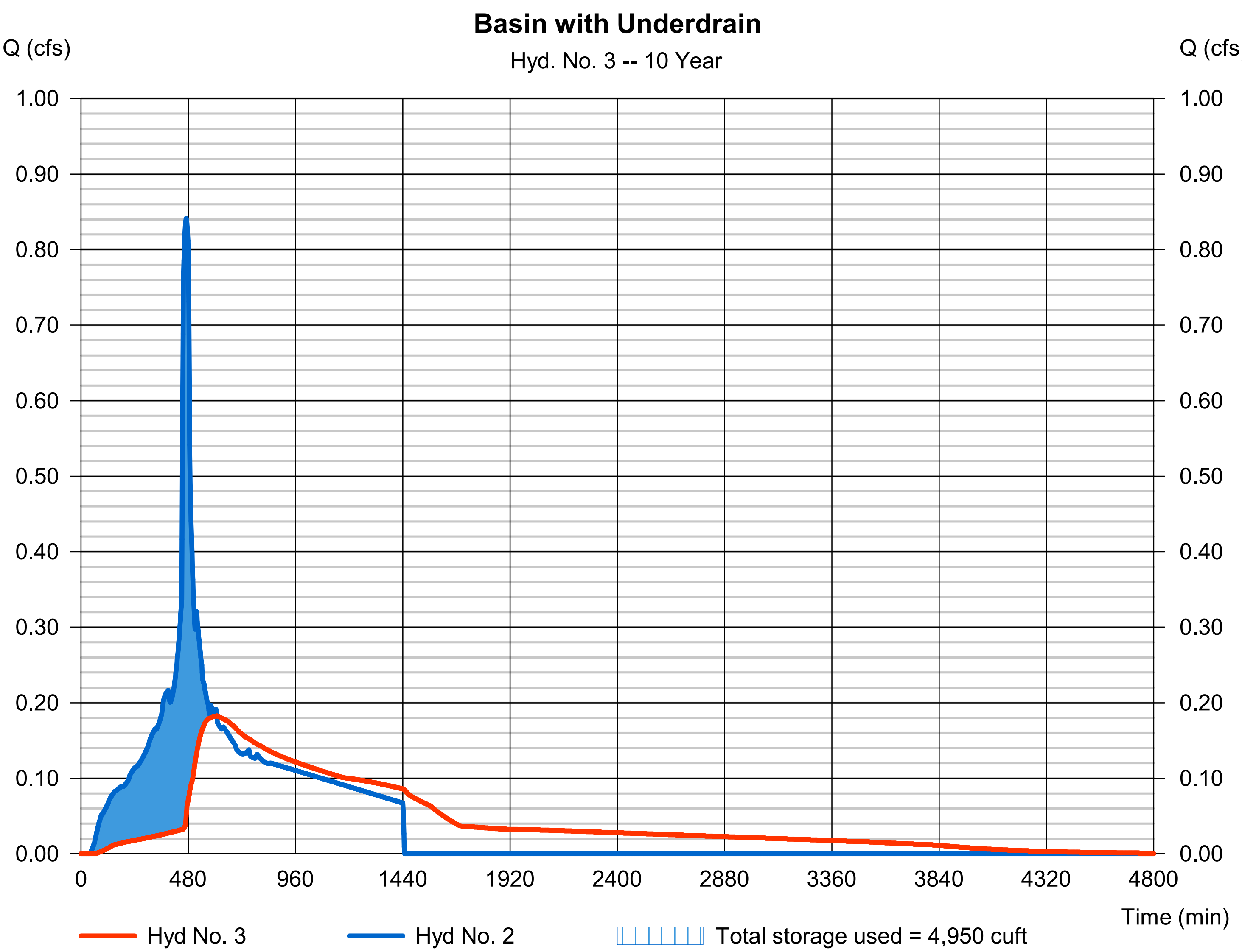
Hydraflow Hydrographs Extension for Autodesk® Civil 3D® 2019 by Autodesk, Inc. v2020 Thursday, 01 / 14 / 2021

## Hyd. No. 3

Basin with Underdrain

| | | | |
|---|---|---|---|
| Hydrograph type | = Reservoir | Peak discharge | = 0.183 cfs |
| Storm frequency | = 10 yrs | Time to peak | = 606 min |
| Time interval | = 2 min | Hyd. volume | = 11,940 cuft |
| Inflow hyd. No. | = 2 - Post development | Max. Elevation | = 101.63 ft |
| Reservoir name | = Basin | Max. Storage | = 4,950 cuft |

Storage Indication method used.



# Hydrograph Summary Report

Hydraflow Hydrographs Extension for Autodesk® Civil 3D® 2019 by Autodesk, Inc. v2020

| Hyd. No. | Hydrograph type (origin) | Peak flow (cfs) | Time interval (min) | Time to Peak (min) | Hyd. volume (cuft) | Inflow hyd(s) | Maximum elevation (ft) | Total strge used (cuft) | Hydrograph Description |
|---|---|---|---|---|---|---|---|---|---|
| 1 | SCS Runoff | 0.459 | 2 | 478 | 7,112 | ------ | ------ | ------ | Pre Development |
| 2 | SCS Runoff | 0.969 | 2 | 470 | 13,846 | ------ | ------ | ------ | Post development |
| 3 | Reservoir | 0.253 | 2 | 552 | 13,824 | 2 | 101.74 | 5,371 | Basin with Underdrain |

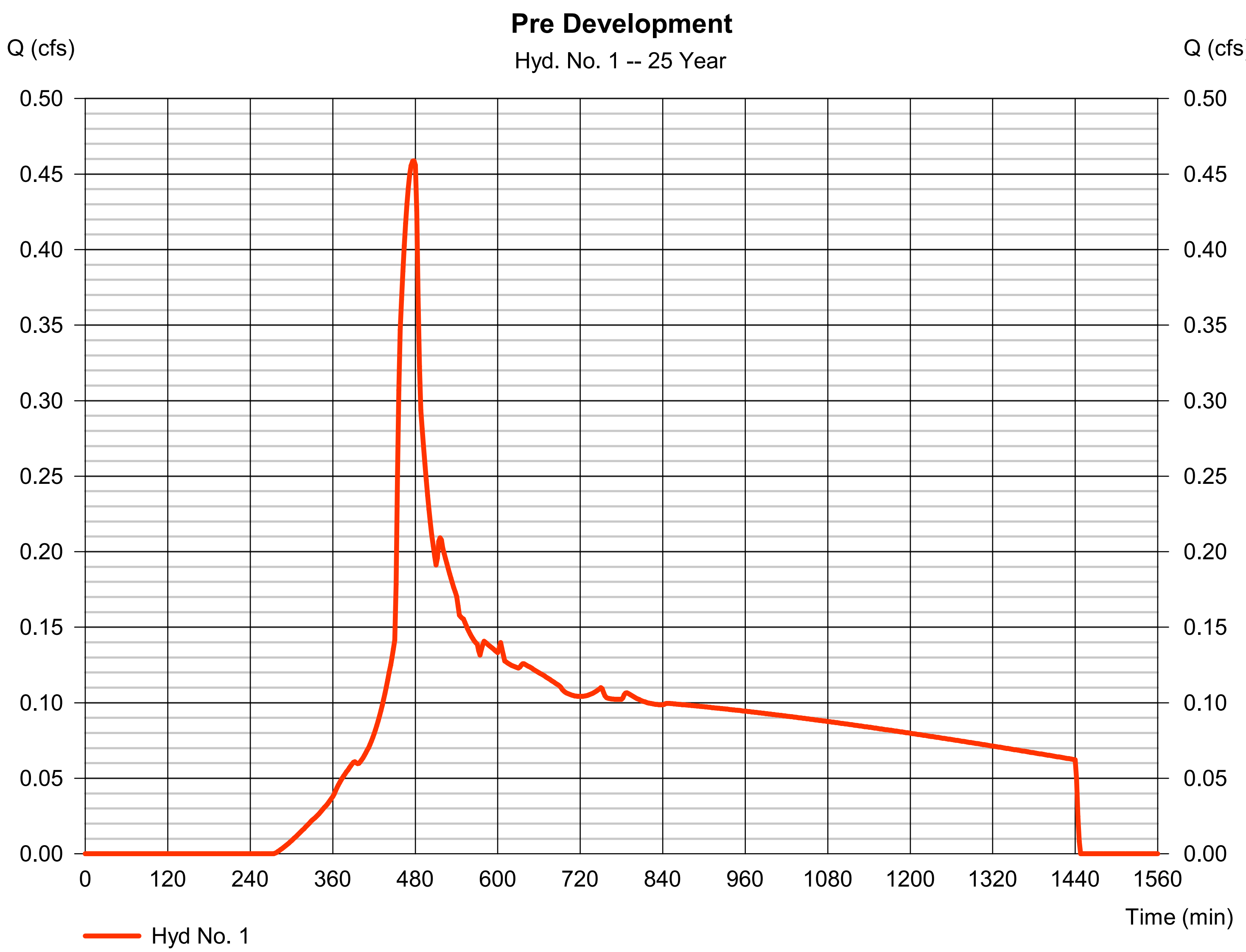
# Hydrograph Report

Hydraflow Hydrographs Extension for Autodesk® Civil 3D® 2019 by Autodesk, Inc. v2020 Thursday, 01 / 14 / 2021

## Hyd. No. 1

Pre Development

| | | | |
|---|---|---|---|
| Hydrograph type | = SCS Runoff | Peak discharge | = 0.459 cfs |
| Storm frequency | = 25 yrs | Time to peak | = 478 min |
| Time interval | = 2 min | Hyd. volume | = 7,112 cuft |
| Drainage area | = 1.110 ac | Curve number | = 79 |
| Basin Slope | = 0.0 % | Hydraulic length | = 0 ft |
| Tc method | = User | Time of conc. (Tc) | = 5.00 min |
| Total precip. | = 3.90 in | Distribution | = Type IA |
| Storm duration | = 24 hrs | Shape factor | = 484 |



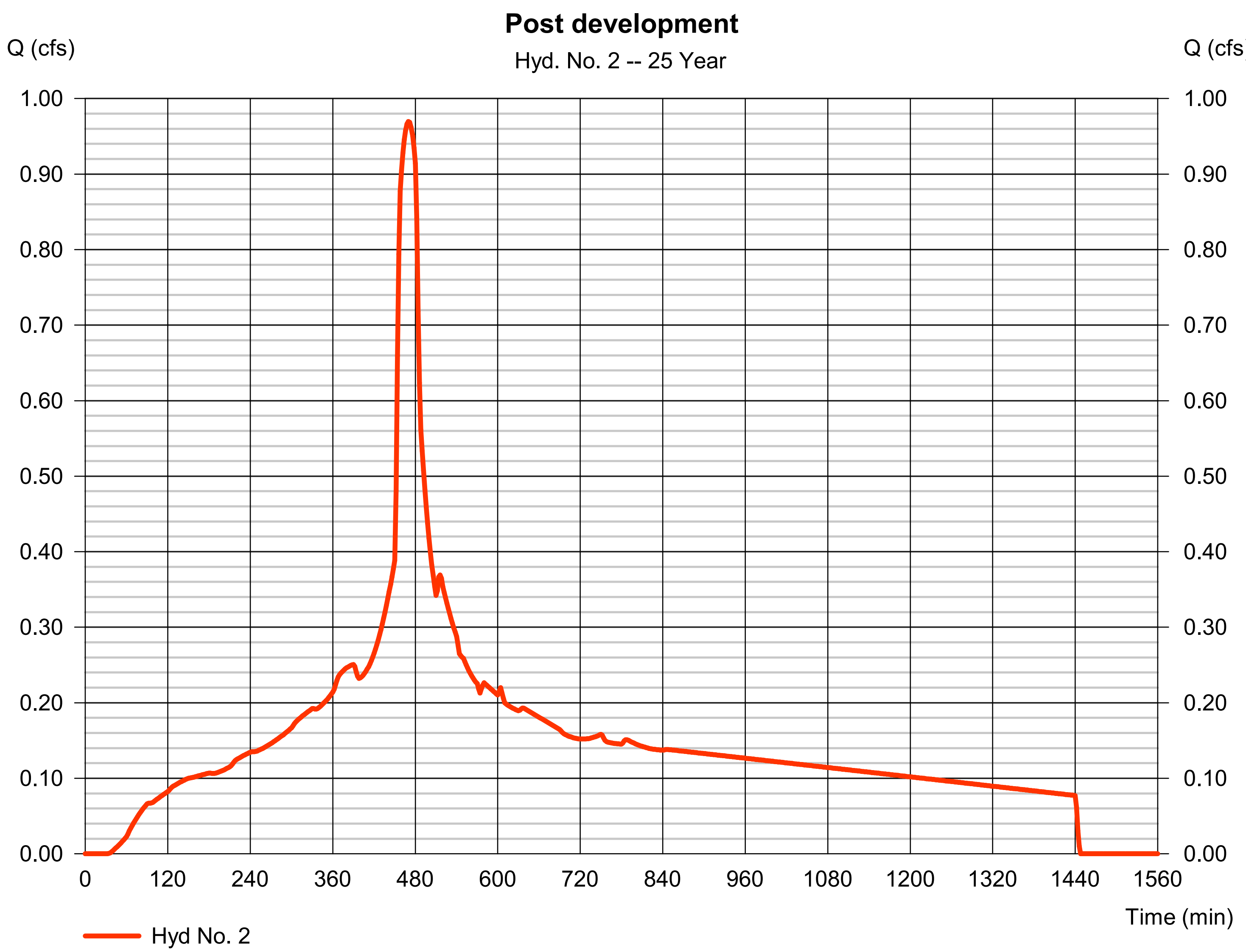
# Hydrograph Report

Hydraflow Hydrographs Extension for Autodesk® Civil 3D® 2019 by Autodesk, Inc. v2020 Thursday, 01 / 14 / 2021

## Hyd. No. 2

Post development

| | | | |
|---|---|---|---|
| Hydrograph type | = SCS Runoff | Peak discharge | = 0.969 cfs |
| Storm frequency | = 25 yrs | Time to peak | = 470 min |
| Time interval | = 2 min | Hyd. volume | = 13,846 cuft |
| Drainage area | = 1.110 ac | Curve number | = 98 |
| Basin Slope | = 0.0 % | Hydraulic length | = 0 ft |
| Tc method | = User | Time of conc. (Tc) | = 5.00 min |
| Total precip. | = 3.90 in | Distribution | = Type IA |
| Storm duration | = 24 hrs | Shape factor | = 484 |



# Hydrograph Report

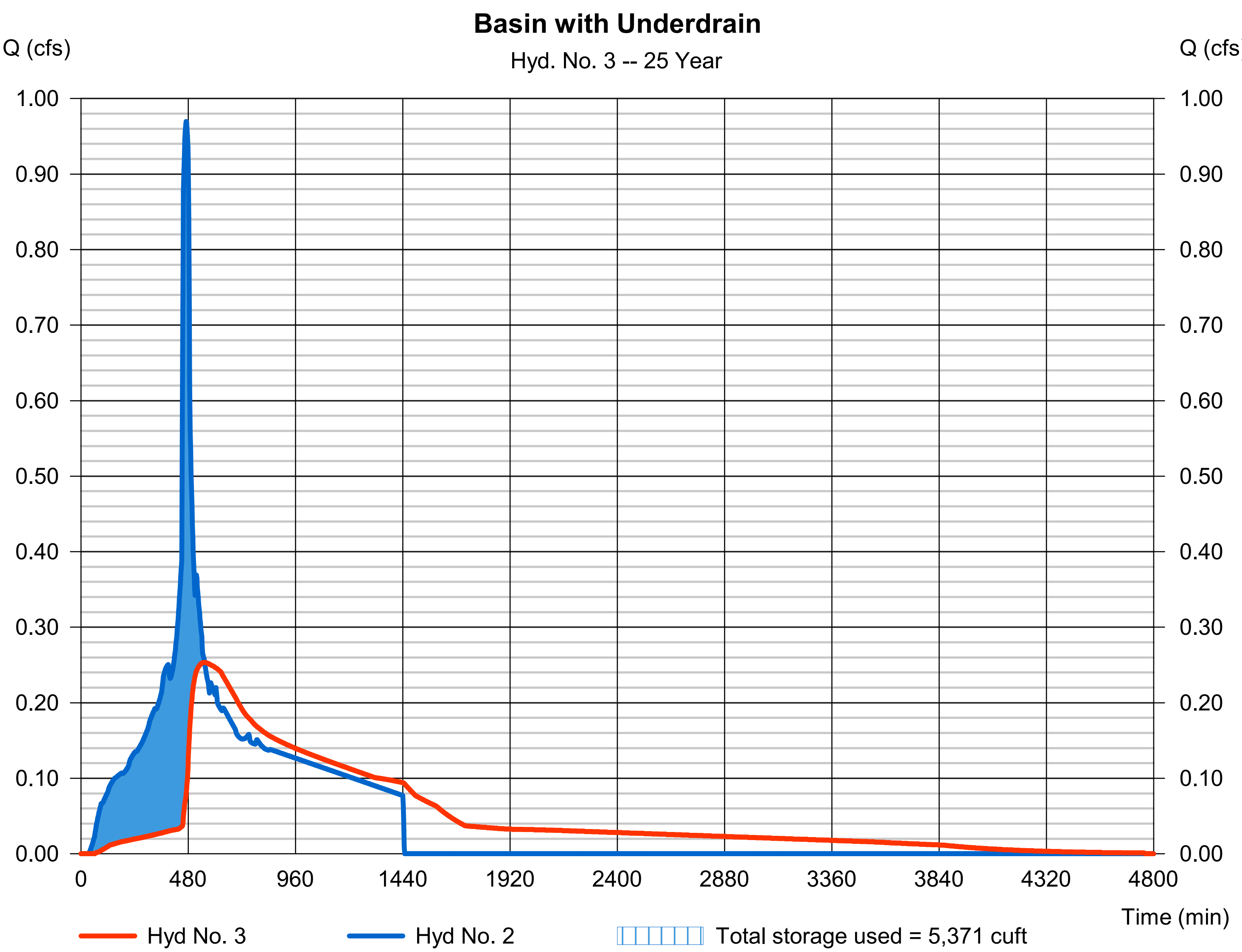
Hydraflow Hydrographs Extension for Autodesk® Civil 3D® 2019 by Autodesk, Inc. v2020 Thursday, 01 / 14 / 2021

## Hyd. No. 3

Basin with Underdrain

| | | | |
|---|---|---|---|
| Hydrograph type | = Reservoir | Peak discharge | = 0.253 cfs |
| Storm frequency | = 25 yrs | Time to peak | = 552 min |
| Time interval | = 2 min | Hyd. volume | = 13,824 cuft |
| Inflow hyd. No. | = 2 - Post development | Max. Elevation | = 101.74 ft |
| Reservoir name | = Basin | Max. Storage | = 5,371 cuft |

Storage Indication method used.



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# Report of
# Geotechnical Investigation
# ODOT Skyline Storage Shed
# Tax Lot 2500, NW Cornelius Pass Road
# Portland, Oregon

## CGT Project Number G2005344

Prepared for

Mr. Luis Umana, CPM
Oregon Department of Transportation (ODOT)
3700 SE 92nd Avenue
Portland, Oregon, 97266

September 28, 2020

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September 28, 2020

Mr. Luis Umana, CPM
Oregon Department of Transportation (ODOT)
3700 SE 92nd Avenue
Portland, Oregon, 97266

**Report of**
**Geotechnical Investigation**
**ODOT Skyline Storage Shed**
**Tax Lot 2500, NW Cornelius Pass Road**
**Portland, Oregon**

CGT Project Number G2005344

Dear Mr. Umana:

Carlson Geotechnical (CGT), a division of Carlson Testing, Inc. (CTI), is pleased to submit this report summarizing the results of our geotechnical investigation and infiltration testing for the proposed ODOT Skyline Storage Shed project. The site is located at Tax Lot 2500, NW Cornelius Pass Road in Portland, Oregon. We performed our work in general accordance with CGT Proposals GP9047 and GP9068, dated August 11, 2020 and August 26, 2020, respectively. Written authorization for our services was received on August 26, 2020.

We appreciate the opportunity to work with you on this project. Please contact us at (503) 601-8250 if you have any questions regarding this report.

Respectfully Submitted,
**CARLSON GEOTECHNICAL**

REGISTERED PROFESSIONAL ENGINEER
94977PE
OREGON
JULY 11, 2019
JIM PRESTON TOMKINS
EXPIRES: 06-30-22

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## TABLE OF CONTENTS



## ATTACHMENTS

# 1.0 INTRODUCTION

Carlson Geotechnical (CGT), a division of Carlson Testing, Inc. (CTI), is pleased to submit this report summarizing the results of our geotechnical investigation for the proposed ODOT Skyline Storage Shed project. The site is located at Tax Lot 2500, NW Cornelius Pass Road in Portland, Oregon, as shown on the attached Site Location, Figure 1.

## 1.1 Project Information

CGT developed an understanding of the proposed project based on our email correspondence with you and review of Conceptual Site Plan prepared by ODOT, dated April 13, 2020, and provided to us on August 6, 2020. Based on our review, we understand the project will include:

- Construction of a new Storage storage building. Although no architectural plans have been provided, we anticipate the structure will be one story, steel-framed, incorporate a slab on grade floor, and footprint of roughly 3,060 square feet. For the purposes of this proposal, we have assumed maximum column, continuous wall, and uniform floor slab loads will be on the order of 50 kips, 4 kips per lineal foot (klf), and 250 pounds per square foot (psf), respectively.
- Installation of six 10,000 gallon deicer tanks within the west portion of the proposed storage building.
- Construction of new access road and material storage area. We assume the new access road and storage area will be surfaced with flexible asphalt concrete pavements.
- Stormwater collected from new impervious areas of the site will be collected and disposed of, at least in part, via on-site infiltration. Design of infiltration facilities will rest with others.
- Although no grading plans have been provided, we anticipate permanent grade changes will include cuts and fills up to about 3 feet relative to existing grades.

## 1.2 Scope of Services

Our scope of work included the following:

- Contact the Oregon Utilities Notification Center to mark the locations of public utilities within a 20-foot radius of our explorations at the site.
- Explore subsurface conditions at the site by observing the advancement of six test pits to depths of up to about 10 feet below ground surface (bgs). Details of the subsurface investigation are presented in Appendix A.
- Conduct infiltration testing in one of the test pits. Results of the infiltration testing are presented in Appendix B.
- Classify the soils encountered in the explorations in general accordance with ASTM D2488 (Visual-Manual Procedure).
- Prepare an Engineering Geology Report (EGR) for the referenced property. Details of the EGR are presented in Appendix C.
- Provide a technical narrative describing surface and subsurface deposits, and local geology of the site, based on the results of our explorations and published geologic mapping.
- Provide recommendations for the Seismic Site Class, mapped maximum considered earthquake spectral response accelerations, and site seismic coefficients.
- Provide a qualitative evaluation of seismic hazards at the site, including earthquake-induced liquefaction, landsliding, and surface rupture due to faulting or lateral spread.

- Provide geotechnical recommendations for site preparation and earthwork.
- Provide geotechnical engineering recommendations for use in design and construction of shallow foundations, floor slabs, and pavements.
- Provide this written report summarizing the results of our geotechnical investigation and recommendations for the project.

## 2.0 SITE DESCRIPTION

### 2.1 Site Geology

Surficial geology is mapped as windblown sediments, consisting primarily of silt and clay[1]. The windblown sediments are generally about 20 to 30 feet thick in this portion of the Tualatin Mountains (Portland West Hills). These soils are susceptible to soil creep on moderate to steep slopes, and are susceptible to loss of strength, landslides, and flow failures, particularly when subjected to heavy rainfall or concentrated runoff. Bedrock in the area is mapped as the Sentinel Bluffs member of the Miocene Columbia River BaStorage Group[2], which extends to hundreds of feet below ground surface.

### 2.2 Site Surface Conditions

The project site is bordered by NW Skyline Boulevard to the northeast, NW Cornelius Pass Road to the southwest, and undeveloped grass land to the southeast. The vacant site is vegetated by tall grasses and scattered trees to the north and south ends of the site. In terms of topography, the site gently descended to the west at a gradient of approximately 10 horizontal to 1 vertical (10H:1V). Site layout, topography, and surface conditions are shown on the attached Site Plan and Site Photographs, Figures 2 and 3, respectively.

### 2.3 Subsurface Conditions

#### 2.3.1 Subsurface Investigation & Laboratory Testing

Our subsurface investigation consisted of six test pits (TP-1 through TP-6) completed on August 28, 2020. The approximate exploration locations are shown on the Site Plan, attached as Figure 2. In summary, the test pits were excavated to depths ranging from about 4 to 10 feet bgs. Details regarding the subsurface investigation, logs of the explorations, and results of laboratory testing are presented in Appendix A. Subsurface conditions encountered during our investigation are summarized below.

#### 2.3.2 Subsurface Materials

Logs of the explorations are presented in Appendix A. The following describes each of the subsurface materials encountered at the site.

*Silt (ML)*
Silt was encountered at the surface of test pits TP-1 through TP-6, and extended to depths ranging from 1½ to 2½ feet bgs. This soil was generally medium stiff to very stiff, light gray-brown, damp to moist, exhibited low to medium plasticity, contained abundant fine roots within the upper ½ to 1½ feet, trace fine-grained sand, and trace subrounded gravel up to ¼-inch in diameter within TP-1.

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[1] Ma, Lina, Madin, Ian P., et al., 2012. Lidar-based Surficial Geologic Map and Database of the Greater Portland, Oregon, Area. Oregon Department of Geology and Mineral Industries Open-File Report O-12-02.

[2] DOGAMI, 2006. Preliminary Geologic Map of the Linnton 7.5' Quadrangle, Multnomah and Washington Counties, Oregon. Oregon Department of Geology and Mineral Industries Open-File Report O-08-06.

*Lean Clay (CL)*

Underlying the silt the test pits, we encountered native lean clay. This soil was generally stiff to very stiff, brown with orange, tan, gray, and black mottling, moist, exhibited low to medium plasticity, and contained trace fine-grained sand. The lean clay extended to depths of about 4 to 10 feet bgs in the test pit.

#### 2.3.3 Groundwater

No groundwater was encountered in depths explored in the test pits on August 28, 2020. To determine approximate regional groundwater levels in the area, we researched well logs available on the Oregon Water Resources Department (OWRD)[3] website for wells located within Section 31, Township 2 North, Range 1 West, Willamette Meridian. Our review indicated that groundwater levels in the area generally ranged from about 180 to 270 feet bgs. Shallow monitoring wells constructed in 2003 and 2004 near the intersection of NW Cornelius Pass Road and NW Skyline Boulevard show a perched groundwater table with water levels at about 3 to 4 feet bgs. It should be noted groundwater levels vary with local topography. In addition, the groundwater levels reported on the OWRD logs often reflect the purpose of the well, so water well logs may only report deeper, confined groundwater, while geotechnical or environmental borings will often report any groundwater encountered, including shallow, unconfined groundwater. Therefore, the levels reported on the OWRD well logs referenced above are considered generally indicative of local water levels and may not reflect actual groundwater levels at the project site. We anticipate that groundwater levels will fluctuate due to seasonal and annual variations in precipitation, changes in site utilization, or other factors. Additionally, the on-site fine-grained soils (ML, CL) are conducive to formation of perched groundwater.

The depth to groundwater map for the Portland area[4] indicates groundwater is present at depths of 273 feet bgs in the vicinity of the site. It should be noted that the levels reported by the referenced map are average values for a given location and incorporate a degree of uncertainty. For this location the uncertainty is described as “High.”

## 3.0 SEISMIC CONSIDERATIONS

### 3.1 Seismic Design

Section 1613.2.2 of the 2019 Oregon Structural Specialty Code (2019 OSSC) requires that the determination of the seismic site class be in accordance with Chapter 20 of the American Society of Civil Engineers Minimum Design Loads for Buildings and Other Structures (ASCE 7-16). We have assigned the site as Site Class D (“Stiff Soil”) based on geologic mapping and subsurface conditions encountered during our investigation.

Earthquake ground motion parameters for the site were obtained in accordance with the 2019 OSSC using the Seismic Hazards by Location calculator on the ATC website[5]. The site Latitude 45.606924° North and Longitude 122.862562° West were input as the site location. The following table shows the recommended seismic design parameters for the site.

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3 Oregon Water Resources Department, 2020. Well Log Records, *accessed September 2020*, from OWRD web site: http://apps.wrd.state.or.us/apps/gw/well_log/.

4 Snyder, D.T., 2008, Estimated depth to ground water and configuration of the water table in the Portland, Oregon area: U.S. Geological Survey, Scientific Investigations Report SIR-2008-5059, scale 1:60,000.

5 Applied Technology Council (ATC), 2020. USGS seismic design parameters determined using “Seismic Hazards by Location,” *accessed September 2020*, from the ATC website https://hazards.atcouncil.org/.

**Seismic Ground Motion Values**

| Parameter | | Value |
|---|---|---|
| Mapped Acceleration Parameters | Spectral Acceleration, 0.2 second ($S_s$) | 0.919g |
| | Spectral Acceleration, 1.0 second ($S_1$) | 0.426g |
| Coefficients (Site Class D) | Site Coefficient, 0.2 second ($F_A$) | 1.132 |
| | Site Coefficient, 1.0 second ($F_V$)[1] | 1.874 |
| Adjusted MCE Spectral Response Parameters | MCE Spectral Acceleration, 0.2 second ($S_{MS}$) | 1.041g |
| | MCE Spectral Acceleration, 1.0 second ($S_{M1}$) | 0.798g |
| Design Spectral Response Accelerations | Design Spectral Acceleration, 0.2 second ($S_{DS}$) | 0.694g |
| | Design Spectral Acceleration, 1.0 second ($S_{D1}$) | 0.532g |
| Seismic Design Category (Risk Category II) | | D |
| [1] Value determined from 2019 OSSC Table 1613.2.3(2). | | |

### 3.2 Seismic Hazards

CGT performed an Engineering Geology Report (EGR) for the site, the results of which are presented in the attached Appendix C.

## 4.0 CONCLUSIONS

Based on the results of our field explorations and analyses, the site may be developed as described in Section 1.1 of this report, provided the recommendations presented in this report are incorporated into the design and development.

## 5.0 RECOMMENDATIONS

The recommendations presented in this report are based on the information provided to us, results of our field investigation and analyses, laboratory data, and professional judgment. CGT has observed only a small portion of the pertinent subsurface conditions. The recommendations are based on the assumptions that the subsurface conditions do not deviate appreciably from those found during the field investigation. CGT should be consulted for further recommendations if the design of the proposed development changes and/or variations or undesirable geotechnical conditions are encountered during site development.

### 5.1 Site Preparation

#### 5.1.1 Stripping

Existing vegetation, topsoil, and rooted soils should be removed from within, and for a minimum 5-foot margin around, proposed building pad and pavement areas. Based on the results of our field explorations, topsoil stripping depths are anticipated to range from ½ to 1½ feet bgs. These materials may be deeper or shallower at locations away from the completed explorations. The geotechnical engineer’s representative should provide recommendations for actual stripping depths based on observations during site stripping. Stripped surface vegetation and rooted soils should be transported off-site for disposal, or stockpiled for later use in landscaped areas.

#### 5.1.2 Grubbing

Grubbing of trees should include the removal of the root mass and roots greater than ½-inch in diameter. Grubbed materials should be transported off-site for disposal. Root masses from larger trees may extend greater than 3 feet bgs. Where root masses are removed, the resulting excavation should be properly backfilled with structural fill in conformance with Section 5.4 of this report.

#### 5.1.3 Test Pit Backfills

The test pits conducted at the site were loosely backfilled during our field investigation. Where test pits are located within finalized building, structural fill, or pavement areas, the loose backfill materials should be re-excavated. The resulting excavations should be backfilled with structural fill in conformance with Section 5.4 of this report.

#### 5.1.4 Existing Utilities & Below-Grade Structures

All existing utilities at the site should be identified prior to excavation. Abandoned utility lines beneath the new building, pavements, and hardscaping features should be completely removed or grouted full. Soft, loose, or otherwise unsuitable soils encountered in utility trench excavations should be removed and replaced with structural fill in conformance with Section 5.4 this report. Buried structures (i.e. footings, foundation walls, retaining walls, slabs-on-grade, tanks, etc.), if encountered during site development, should be completely removed and replaced with structural fill in conformance with Section 5.4 of this report.

#### 5.1.5 Subgrade Preparation

After site preparation as recommended above, but prior to placement of structural fill and/or aggregate base, the geotechnical engineer's representative should observe the exposed subgrade soils in order to identify areas of excessive yielding through either proof rolling or probing. Proof rolling of subgrade soils is typically conducted during dry weather using a fully-loaded, 10- to 12-cubic-yard, tandem-axle, tire-mounted, dump truck or equivalent weighted water truck. Areas of limited access or that appear too soft or wet to support proof rolling equipment should be evaluated by probing. During wet weather, subgrade preparation should be performed in general accordance with the recommendations presented in Section 5.3 of this report. If areas of soft soil or excessive yielding are identified, the affected material should be over-excavated to firm, unyielding subgrade, and replaced with imported granular structural fill in conformance with Section 5.4.2 of this report.

#### 5.1.6 Erosion Control

Erosion and sedimentation control measures should be employed in accordance with applicable City, County, and State regulations.

### 5.2 Temporary Excavations

#### 5.2.1 Overview

Conventional earthmoving equipment in proper working condition should be capable of making necessary excavations for the anticipated site cuts as described earlier in this report. All excavations should be in accordance with applicable OSHA and state regulations. It is the contractor's responsibility to select the excavation methods, to monitor site excavations for safety, and to provide any shoring required to protect personnel and adjacent improvements. A "competent person," as defined by OR-OSHA, should be on-site during construction in accordance with regulations presented by OR-OSHA. CGT's current role on the project does not include review or oversight of excavation safety.

#### 5.2.2 OSHA Soil Type

For use in the planning and construction of temporary excavations up to 10 feet in depth, an OSHA soil type "A" should be used for the on-site lean clay (CL) soils encountered in the borings. Similarly, an OSHA soil type "B" should be used for the near surface silt (ML) soils.

#### 5.2.3 Utility Trenches

Temporary trench cuts should stand near vertical to depths of approximately 4 feet in the native, lean clay (CL) encountered near the surface of the site. If groundwater seepage undermines the stability of the trench, or if sidewall caving is observed during excavation, the sidewalls should be flattened or shored. Depending on the time of year trench excavations occur, trench dewatering may be required in order to maintain dry working conditions. Pumping from sumps located within the trench will likely be effective in removing water resulting from seepage. If groundwater is encountered, we recommend placing trench stabilization material at the base of the excavations. Trench stabilization material should be in conformance with Section 5.4.4.

#### 5.2.4 Excavations Near Foundations

Excavations near footings should not extend within a 1½ horizontal to 1 vertical (1½H:1V) plane projected out and down from the outside, bottom edge of the footings. In the event excavation needs to extend below the referenced plane, temporary shoring of the excavation and/or underpinning of the subject footing may be required. The geotechnical engineer should be consulted to review proposed excavation plans for this design case to provide specific recommendations.

#### 5.2.5 Draping of Cut Slopes

In wet weather conditions, we recommend temporary cut slopes in excess of 4 feet in height (created during construction) be draped with minimum 10-mil plastic sheeting (e.g. polyethylene). Draping of cut slopes less than 4 feet in height may also be performed. The draping should extend from the base of the cut slope and back from the top of the cut slope sufficient to limit runoff from flowing under the covering. The plastic sheets should be lapped sufficiently to prevent water from flowing directly onto the slope and should extend at least several feet beyond each side of the cut area. The plastic should be weighted or otherwise anchored so that it remains on the slope during construction. Runoff from the sheeting should not be allowed to pond or infiltrate into the subsurface at the toe of the slope, but should be collected and diverted away from the cut slope to a suitable discharge point.

### 5.3 Wet Weather Considerations

For planning purposes, the wet season should be considered to extend from late September to late June. It is our experience that dry weather working conditions should prevail between early July and mid-September. Notwithstanding the above, soil conditions should be evaluated in the field by the geotechnical engineer's representative at the initial stage of site preparation to determine whether the recommendations within this section should be incorporated into construction.

#### 5.3.1 Overview

Due to the fines content, the on-site fine-grained soils (ML, CL) is susceptible to disturbance during wet weather. Trafficability of these soils may be difficult, and significant damage to subgrade soils could occur, if earthwork is undertaken without proper precautions at times when the exposed soils are more than a few percentage points above optimum moisture content. For wet weather construction, site preparation activities may need to be accomplished using track-mounted equipment, loading removed material onto trucks supported on granular haul roads, or other methods to limit soil disturbance. The geotechnical engineer's representative should evaluate the subgrade during excavation by probing rather than proof rolling. Soils that have been disturbed during site preparation activities, or soft or loose areas identified during probing, should be over-excavated to firm, unyielding subgrade, and replaced with imported granular structural fill in conformance with Section 5.4.2.

#### 5.3.2 Geotextile Separation Fabric

We recommend a geotextile separation fabric be placed to serve as a barrier between the prepared subgrade and granular fill/base rock in areas of repeated or heavy construction traffic. The geotextile fabric should meet the requirements presented in the current Oregon Department of Transportation (ODOT) Standard Specification for Construction (ODOT SSC), Section 02320.

#### 5.3.3 Granular Working Surfaces (Haul Roads & Staging Areas)

Haul roads subjected to repeated heavy, tire-mounted, construction traffic (e.g. dump trucks, concrete trucks, etc.) will require a minimum of 18 inches of imported granular material. For light staging areas, 12 inches of imported granular material is typically sufficient. Additional granular material or geo-grid reinforcement may be recommended based on site conditions and/or loading at the time of construction. The imported granular material should be in conformance with Section 5.4.2 and have less than 5 percent material passing the U.S. Standard No. 200 Sieve. The prepared subgrade should be covered with geotextile fabric (Section 5.3.2) prior to placement of the imported granular material. The imported granular material should be placed in a single lift (up to 24 inches deep) and compacted using a smooth-drum, non-vibratory roller until well-keyed.

#### 5.3.4 Footing Subgrade Protection

A minimum of 6 inches of imported granular material is recommended to serve as leveling course, and protect fine-grained, footing subgrades from foot traffic during inclement weather. The imported granular material should be in conformance with Section 5.4.2. The maximum particle size should be limited to 1 inch. The imported granular material should be placed in one lift over the prepared, undisturbed subgrade, and compacted using non-vibratory equipment until well keyed.

Surface water should not be allowed to collect in footing excavations. The excavations should be draped and/or provided with sumps to preclude water accumulation during inclement weather.

### 5.4 Structural Fill

The geotechnical engineer should be provided the opportunity to review all materials considered for use as structural fill (prior to placement). Samples of the proposed fill materials should be submitted to the geotechnical engineer a minimum of 5 business days prior their use on site[6]. The geotechnical engineer's representative should be contacted to evaluate compaction of structural fill as the material is being placed. Evaluation of compaction may take the form of in-place density tests and/or proof roll tests with suitable equipment. Structural fill should be evaluated at intervals not exceeding every 2 vertical feet as the fill is being placed.

[6] Laboratory testing for moisture density relationship (Proctor) is required. Tests for gradation may be required.

#### 5.4.1 On-Site Soils – General Use

Recognizing the relatively limited grading (fill placement) associated with this project and their moisture sensitivity, we do not recommend re-using the onsite silt and clay soils as structural fill. We recommend using imported granular material for structural fill.

#### 5.4.2 Imported Granular Structural Fill – General Use

Imported granular structural fill should consist of angular pit or quarry run rock, crushed rock, or crushed gravel that is fairly well graded between coarse and fine particle sizes. The granular fill should contain no organic matter, debris, or particles larger than 4 inches, and have less than 5 percent material passing the U.S. Standard No. 200 Sieve. For fine-grading purposes, the maximum particle size should be limited to 1½inches. The percentage of fines can be increased to 12 percent of the material passing the U.S. Standard No. 200 Sieve if placed during dry weather, and provided the fill material is moisture-conditioned, as necessary, for proper compaction. Imported granular fill material should be placed in lifts with a maximum thickness of about 12 inches, and compacted to not less than 95 percent of the material's maximum dry density, as determined in general accordance with ASTM D1557 (Modified Proctor). Proper moisture conditioning and the use of vibratory equipment will facilitate compaction of these materials.

Granular fill materials with high percentages of particle sizes in excess of 1½ inches are considered non-moisture-density testable materials. As an alternative to conventional density testing, compaction of these materials should be evaluated by proof roll test observation (deflection tests), where accepted by the geotechnical engineer.

#### 5.4.3 Floor Slab Base Rock

Floor slab base rock should consist of well-graded granular material (crushed rock) containing no organic matter or debris, have a maximum particle size of ¾ inch, and have less than 5 percent material passing the U.S. Standard No. 200 Sieve. Floor slab base rock should be placed in one lift and compacted to not less than 95 percent of the material's maximum dry density as determined in general accordance with ASTM D1557 (Modified Proctor). We recommend "choking" the surface of the base rock with sand just prior to concrete placement. Choking means the voids between the largest aggregate particles are filled with sand, but does not provide a layer of sand above the base rock. Choking the base rock surface reduces the lateral restraint on the bottom of the concrete during curing.

#### 5.4.4 Trench Base Stabilization Material

If groundwater is present at the base of utility excavations, trench base stabilization material should be placed. Trench base stabilization material should consist of a minimum of 1 foot of well-graded granular material with a maximum particle size of 4 inches and less than 5 percent material passing the U.S. Standard No. 4 Sieve. The material should be free of organic matter and other deleterious material, placed in one lift, and compacted until well-keyed.

#### 5.4.5 Trench Backfill Material

Trench backfill for the utility pipe base and pipe zone should consist of granular material as recommended by the utility pipe manufacturer. Trench backfill above the pipe zone should consist of well-graded granular material containing no organic matter or debris, have a maximum particle size of ¾ inch, and have less than 8 percent material passing the U.S. Standard No. 200 Sieve. As a guideline, trench backfill should be placed in maximum 12-inch-thick lifts. The earthwork contractor may elect to use alternative lift thicknesses based on their experience with specific equipment and fill material conditions during construction in order to achieve the required compaction. The following table presents recommended relative compaction percentages for utility trench backfill.

**Table 1 Utility Trench Backfill Compaction Recommendations**

| Backfill Zone | Recommended Minimum Relative Compaction | |
|---|---|---|
| | Structural Areas[1,2] | Landscaping Areas |
| Pipe Base and Within Pipe Zone | 90% ASTM D1557 or pipe manufacturer's recommendation | 88% ASTM D1557 or pipe manufacturer's recommendation |
| Above Pipe Zone | 92% ASTM D1557 | 90% ASTM D1557 |
| Within 3 Feet of Design Subgrade | 95% ASTM D1557 | 90% ASTM D1557 |

1 Includes proposed building, pavement areas, structural fill areas, exterior hardscaping, etc.
2 Or as specified by the local jurisdiction where located in the public right of way.

### 5.4.6 Controlled Low-Strength Material (CLSM)

CLSM is a self-compacting, cementitious material that is typically considered when backfilling localized areas. CLSM is sometimes referred to as "controlled density fill" or CDF. Due to its flowable characteristics, CLSM typically can be placed in restricted-access excavations where placing and compacting fill is difficult. If chosen for use at this site, we recommend the CLSM be in conformance with Section 00442 of the most recent, ODOT SSC. The geotechnical engineer's representative should observe placement of the CLSM and obtain samples for compression testing in accordance with ASTM D4832. As a guideline, for each day's placement, two compressive strength specimens from the same CLSM sample should be tested. The results of the two individual compressive strength tests should be averaged to obtain the reported 28-day compressive strength. If CLSM is considered for use on this site, please contact the geotechnical engineer for site-specific and application-specific recommendations.

## 5.5 Permanent Slopes

### 5.5.1 Overview

Permanent cut or fill slopes constructed at the site, if any, should be graded at 2H:1V or flatter. Constructed slopes should be overbuilt by a few feet depending on their size and gradient so that they can be properly compacted prior to being cut to final grade. The surface of all slopes should be protected from erosion by seeding, sodding, or other acceptable means. Adjacent on-site and off-site structures should be located at least 5 feet from the top of slopes.

### 5.5.2 Placement of Fill on Slopes

New fill should be placed and compacted against horizontal surfaces. Where slopes exceed 5H:1V, the slopes should be keyed and benched prior to structural fill placement in general accordance with the attached Fill Slope Detail, Figure 4. If subdrains are needed on benches, subject to the review of the CGT geotechnical representative, they should be placed as shown on the attached Fill Slope Detail. In order to achieve well-compacted slope faces, slopes should be overbuilt by a few feet and then trimmed back to proposed final grades. A representative from CGT should observe the benches, keyways, and associated subdrains, if needed, prior to placement of structural fill.

### 5.6 Shallow Foundations

#### 5.6.1 Subgrade Preparation

Satisfactory subgrade support for shallow foundations can be obtained from the native, stiff to better lean clay (CL) or new structural fill that is properly placed and compacted on these materials during construction. These soils were first encountered at depths of about ½ to 1½ feet bgs within our explorations in the vicinity of the building pad. The geotechnical engineer's representative should be contacted to observe subgrade conditions prior to placement of forms, reinforcement steel, or granular backfill (if required). If soft, loose, or otherwise unsuitable soils are encountered, they should be over-excavated as recommended by the geotechnical representative at the time of construction. The resulting over-excavation should be brought back to grade with imported granular structural fill in conformance with Section 5.4.2. The maximum particle size of over-excavation backfill should be limited to 1½ inches. All granular pads for footings should be constructed a minimum of 6 inches wider on each side of the footing for every vertical foot of over-excavation.

#### 5.6.2 Minimum Footing Width & Embedment

Minimum footing widths should be in conformance with the current OSSC. As a guideline, CGT recommends individual spread footings have a minimum width of 24 inches. We recommend continuous wall footings have a minimum width of 18 inches. All footings should be founded at least 18 inches below the lowest, permanent adjacent grade to develop lateral capacity and for frost protection.

#### 5.6.3 Bearing Pressure & Settlement

Footings founded as recommended above should be proportioned for a maximum allowable soil bearing pressure of 2,000 pounds per square foot (psf). This bearing pressure is a net bearing pressure, applies to the total of dead and long-term live loads, and may be increased by one-third when considering seismic or wind loads. For foundations founded as recommended above, total settlement of foundations is anticipated to be less than 1 inch. Differential settlements between adjacent columns and/or bearing walls should not exceed ½ inch. If an increased allowable soil bearing pressure is desired, the geotechnical engineer should be consulted.

#### 5.6.4 Lateral Capacity

A maximum passive (equivalent fluid) earth pressure of 150 pounds per cubic foot (pcf) is recommended for design of footings cast neat into excavations in suitable native soil or confined by imported granular structural fill that is properly placed and compacted during construction. The recommended earth pressure was computed using a factor of safety of 1½, which is appropriate due to the amount of movement required to develop full passive resistance. In order to develop the above capacity, the following should be understood:

1. Concrete must be poured neat in excavations or the foundations must be backfilled with imported granular structural fill,
2. The adjacent grade must be level,
3. The static ground water level must remain below the base of the footings throughout the year.
4. Adjacent floor slabs, pavements, or the upper 18-inch-depth of adjacent, unpaved areas should not be considered when calculating passive resistance.

An ultimate coefficient of friction equal to 0.35 may be used when calculating resistance to sliding for footings founded on the native soils described above. An ultimate coefficient of friction equal to 0.45 may be used when calculating resistance to sliding for footings founded on a minimum of 6 inches of imported granular structural fill (crushed rock) that is properly placed and compacted during construction.

#### 5.6.5 Subsurface Drainage

Recognizing the fine-grained soils encountered at this site, we recommend placing foundation drains at the exterior, base elevations of perimeter continuous wall footings. Foundation drains should consist of a minimum 4-inch diameter, perforated, PVC drainpipe wrapped with a non-woven geotextile filter fabric. The drains should be backfilled with a minimum of 2 cubic feet of open graded drain rock per lineal foot of pipe. The drain rock should also be encased in a geotextile fabric in order to provide separation from the surrounding fine-grained soils. Foundation drains should be positively sloped and should outlet to a suitable discharge point. The geotechnical engineer's representative should observe the drains prior to backfilling. Roof drains should not be tied into foundation drains.

### 5.7 Floor Slabs

#### 5.7.1 Subgrade Preparation

Satisfactory subgrade support for slabs constructed on grade, supporting up to 250 psf area loading, can be obtained from the native, stiff to better lean clay (CL) or new structural fill that is properly placed and compacted on these materials during construction. The geotechnical engineer's representative should observe floor slab subgrade soils to evaluate surface consistencies. If soft, loose, or otherwise unsuitable soils are encountered, they should be over-excavated as recommended by the CGT geotechnical representative at the time of construction. The resulting over-excavation should be brought back to grade with imported granular structural fill as described in Section 5.4.2.

#### 5.7.2 Crushed Rock Base

Concrete floor slabs should be supported on a minimum 6-inch-thick layer of crushed rock (base rock) in conformance with Section 5.4.3.

#### 5.7.3 Design Considerations

For floor slabs constructed with a 6-inch thick base rock layer as recommended, an effective modulus of subgrade reaction of 175 pounds per cubic inch (pci) is recommended for the design of the floor slab. A higher effective modulus of subgrade reaction can be obtained by increasing the base rock thickness. Please contact the geotechnical engineer for additional recommendations if a higher modulus is desired. Floor slabs constructed as recommended will likely settle less than ½ inch. For general floor slab construction, slabs should be jointed around columns and walls to permit slabs and foundations to settle differentially.

#### 5.7.4 Subgrade Moisture Considerations

Liquid moisture and moisture vapor should be expected at the subgrade surface. The recommended crushed rock base is anticipated to provide protection against liquid moisture. Where moisture vapor emission through the slab must be minimized, e.g. impervious floor coverings, storage of moisture sensitive materials directly on the slab surface, etc., a vapor retarding membrane or vapor barrier below the slab should be considered. Factors such as cost, special considerations for construction, floor coverings, and end use suggest that the decision regarding a vapor retarding membrane or vapor barrier be made by the architect and owner.

If a vapor retarder or vapor barrier is placed below the slab, its location should be based on current American Concrete Institute (ACI) guidelines, ACI 302 Guide for Concrete Floor and Slab Construction. In some cases, this indicates placement of concrete directly on the vapor retarder or barrier. Please note that the placement of concrete directly on impervious membranes increases the risk of plastic shrinkage cracking and slab curling in the concrete. Construction practices to reduce or eliminate such risk, as described in ACI 302, should be employed during concrete placement.

### 5.8 Pavements

Pavement subgrade preparation should be performed in general accordance with the recommendations presented in Section 5.1.5 above. For relatively impermeable pavements, the subgrade surfaces should be crowned (or sloped) for proper drainage in accordance with specifications provided by the project civil engineer. For permeable pavements, the subgrade should be graded in accordance with the specification provided by the pavement designer.

#### 5.8.1 Design Section(s)

Pavement section design was not part of this current assignment. We would be pleased to provide recommendations for design section(s) for pavement areas at the site, upon request, for an additional fee.

### 5.9 Additional Considerations

#### 5.9.1 Drainage

Subsurface drains should be connected to the nearest storm drain, on-site infiltration system (to be designed by others) or other suitable discharge point. Paved surfaces and grading near or adjacent to the building should be sloped to drain away from the building. Surface water from paved surfaces and open spaces should be collected and routed to a suitable discharge point. Surface water should not be directed into foundation drains.

## 6.0 RECOMMENDED ADDITIONAL SERVICES

### 6.1 Design Review

Geotechnical design review is of paramount importance. We recommend the geotechnical design review take place prior to releasing bid packets to contractors.

### 6.2 Observation of Construction

Satisfactory earthwork, foundation, floor slab, and pavement 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. Subsurface conditions observed during construction should be compared with those encountered during subsurface explorations, and recognition of changed conditions often requires experience. We recommend that qualified personnel visit the site with sufficient frequency to detect whether subsurface conditions change significantly from those observed to date and anticipated in this report. We recommend geotechnical engineer’s representative attend a pre-construction meeting coordinated by the contractor and/or developer. The project geotechnical engineer’s representative should provide observations and/or testing of at least the following earthwork elements during construction:

- Site Stripping Grubbing

- Subgrade Preparation for Shallow Foundations, Structural Fills, Floor Slabs, and Pavements
- Compaction of Structural Fill and Utility Trench Backfill
- Compaction of Base Rock for Floor Slabs and Pavements
- Compaction of Asphalt Concrete for Pavements

It is imperative that the owner and/or contractor request earthwork observations and testing at a frequency sufficient to allow the geotechnical engineer to provide a final letter of compliance for the earthwork activities.

## 7.0 LIMITATIONS

We have prepared this report for use by the owner/developer and other members of the design and construction team for the proposed development. The opinions and recommendations contained within this report are forwarded to assist in the planning and design process and are not intended to be, nor should they be construed as, a warranty of subsurface conditions.

We have made observations based on our explorations that indicate the soil conditions at only those specific locations and only to the depths penetrated. These observations do not necessarily reflect soil types, strata thickness, or water level variations that may exist between or away from our explorations. If subsurface conditions vary from those encountered in our site explorations, CGT should be alerted to the change in conditions so that we may provide additional geotechnical recommendations, if necessary. Observation by experienced geotechnical personnel should be considered an integral part of the construction process.

The owner/developer is responsible for ensuring that the project designers and contractors implement our recommendations. When the design has been finalized, prior to releasing bid packets to contractors, we recommend that the design drawings and specifications be reviewed by our firm to see that our recommendations have been interpreted and implemented as intended. If design changes are made, we request that we be retained to review our conclusions and recommendations and to provide a written modification or verification. Design review and construction phase testing and observation services are beyond the scope of our current assignment, but will be provided for an additional fee.

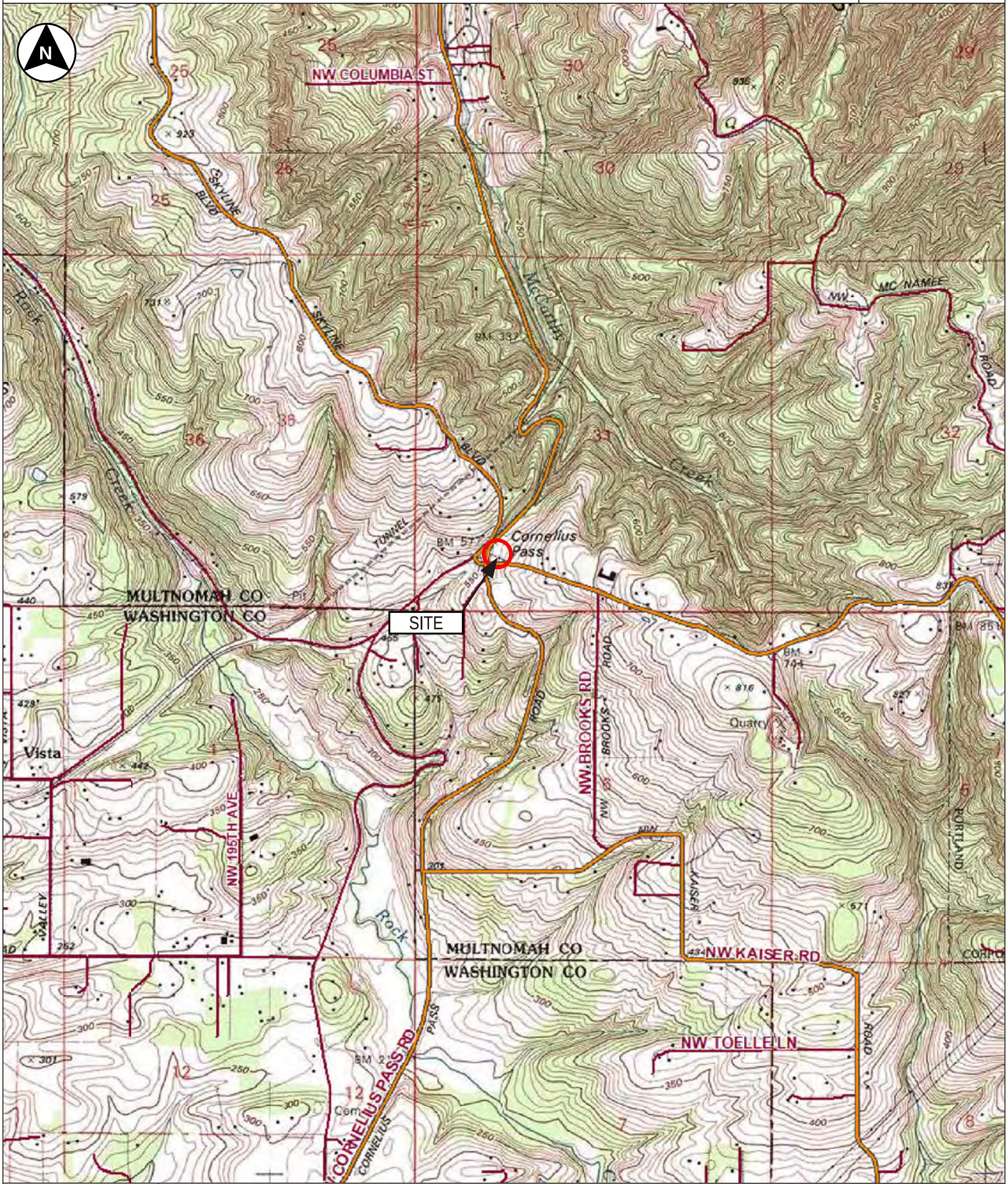
The scope of our services does not include services related to construction safety precautions, and our recommendations are not intended to direct the contractor’s methods, techniques, sequences, or procedures, except as specifically described in our report for consideration in design.

Geotechnical engineering and the geologic sciences are characterized by a degree of uncertainty. Professional judgments presented in this report are based on our understanding of the proposed construction, familiarity with similar projects in the area, and on general experience. Within the limitations of scope, schedule, and budget, our services have been executed in accordance with the generally accepted practices in this area at the time this report was prepared; no warranty, expressed or implied, is made. This report is subject to review and should not be relied upon after a period of three years.

## *ODOT SKYLINE STORAGE SHED - PORTLAND, OREGON*
## *Project Number G2005344*

**FIGURE 1**

**Site Location**

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Drafted by: BLN

Map created with TOPO!™, © 2006 National Geographic Holdings
USGS 7.5 Minute Topographic Map Series, Linnton, Oregon Quadrangle, 1990.
Township 2N, Range 1W, Section 31 Willamette Meridian

Latitude: 45.606924° North
Longitude: 122.862562° West

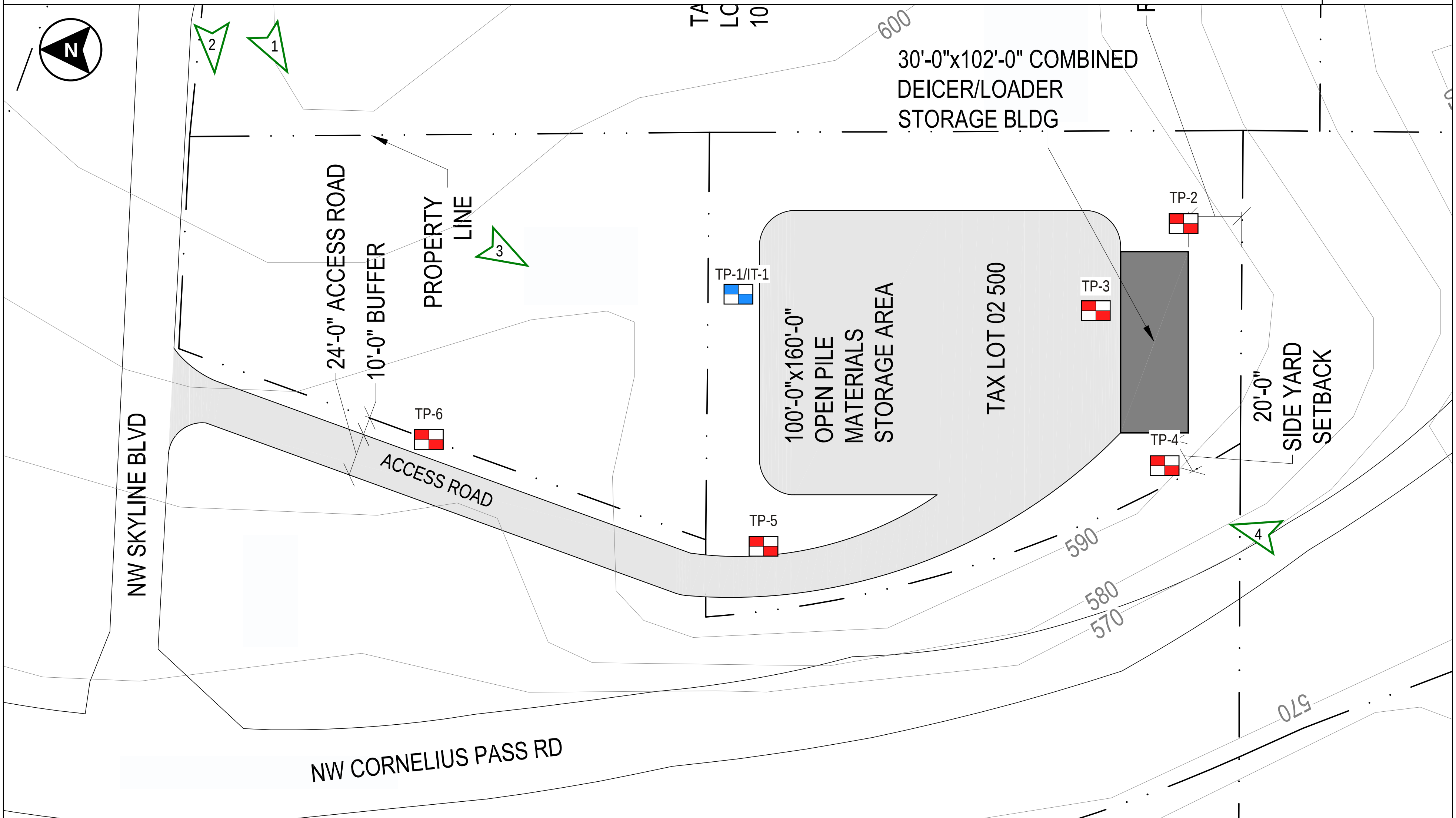
1 Inch = 2,000 feet

0 2000 4000

*ODOT SKYLINE STORAGE SHED - PORTLAND, OREGON*
*Project Number G2005344*

**FIGURE 2**

**Site Plan**

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LEGEND

TP-1 Approximate location of test pit exploration

TP-1/IT-1 Approximate location of infiltration test

1 Orientation of site photographs shown on Figure 3

1 Inch = 80 Feet

0 80 160

NOTES: Drawing based on observations made while on site and Sheet C101, "Site Plan", dated April 13, 2020 provided by client. All locations are approximate.

ODOT SKYLINE STORAGE SHED - PORTLAND, OREGON
*Project Number G2005344*

**FIGURE 3**

**Site Photographs**


Photograph 1


Photograph 2


Photograph 3


Photograph 4

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*See Figure 2 for approximate photograph locations and directions. Photographs were taken at the time of our fieldwork.*

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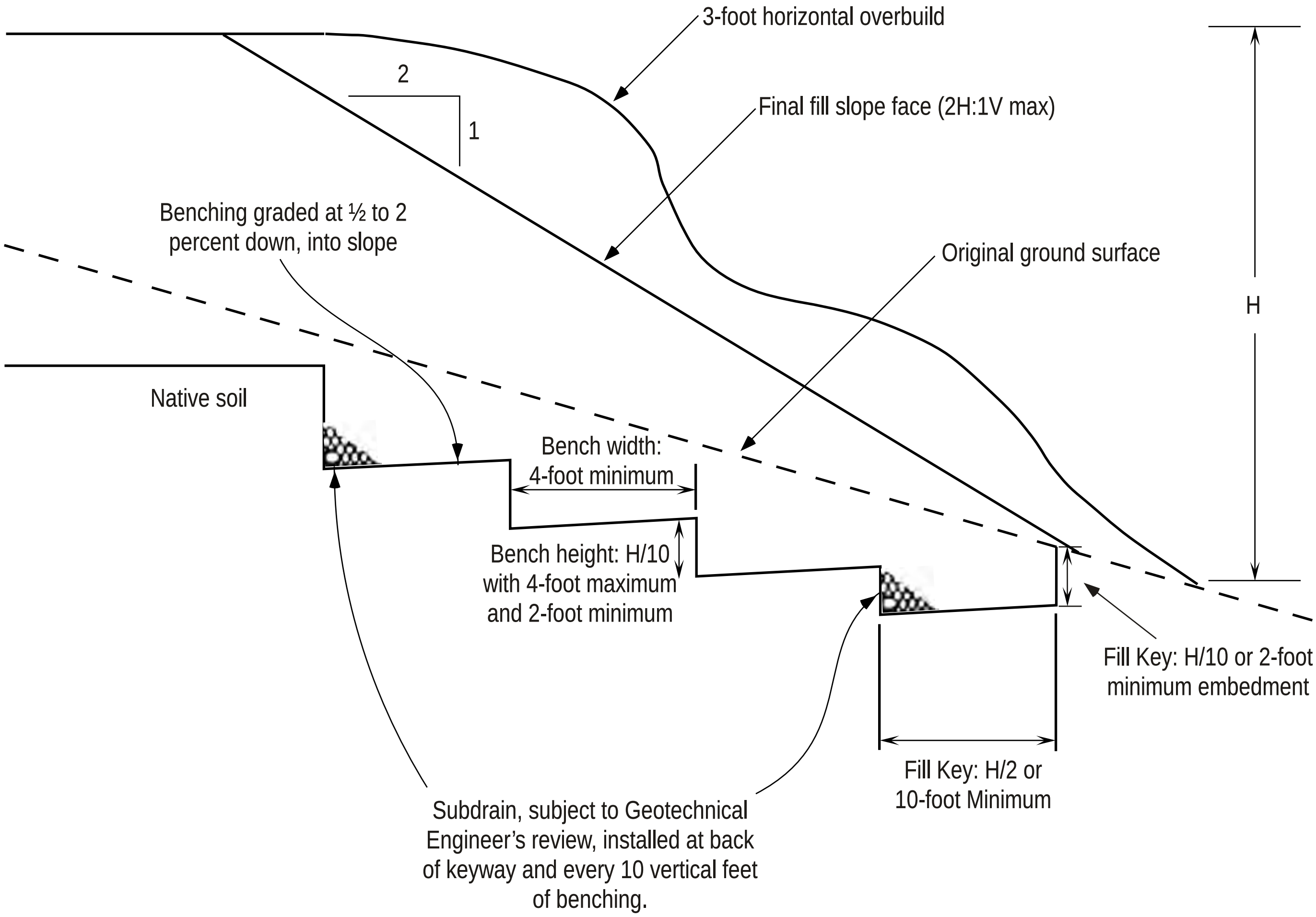
*Project Number G2005344*

**FIGURE 4**

**Fill Slope Detail**

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NOTE: Surfaces to receive fill with slopes steeper than 5H:1V (horizontal:vertical) should be benched and keyed as shown.

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# Appendix A: Subsurface Investigation and Laboratory Testing

## ODOT Skyline Storage Shed
## Tax Lot 2500, NW Cornelius Pass Road
## Portland, Oregon

## CGT Project Number G2005344

September 28, 2020

*Prepared For:*

Mr. Luis Umana, CPM
Oregon Department of Transformation (ODOT)
3700 SE 92nd Avenue
Portland, Oregon, 97266

*Prepared by*
**Carlson Geotechnical**

## A.1.0 SUBSURFACE INVESTIGATION

Our field investigation consisted of six test pits completed on August 28, 2020. The exploration locations are shown on the Site Plan, attached to the geotechnical report as Figure 2. The exploration locations shown therein were determined based on measurements from existing site features (e.g. existing trees, stakes, etc.) and are approximate. The attached figures detail the exploration methods (Figure A1), soil classification criteria (Figure A2), and present detailed logs of the explorations (Figures A3 through A8), as discussed below. Surface elevations indicated on the logs were estimated based on a topographic survey provided by ODOT, dated April 13, 2020.

### A.1.1 Test Pits

CGT observed the excavation of six test pits (TP-1 through TP-6) at the site on August 28, 2020, to depths of up to 10 feet bgs. The test pits were excavated using a John Deere 85G track-mounted excavator provided and operated by ODOT. The test pits were loosely backfilled with the excavated materials upon completion.

### A.1.2 In-Situ Testing – Pocket Penetrometer Tests

Pocket penetrometer readings were generally taken at approximate ½-foot intervals in the upper 4 feet of each test pit. The pocket penetrometer is a hand-held instrument that provides an approximation of the unconfined compressive strength of cohesive, fine-grained soils. The correlation between pocket penetrometer readings and the consistency of cohesive, fine-grained soils is provided on the attached Figure A2.

### A.1.3 Material Classification & Sampling

Representative grab samples of the soils encountered were obtained at select intervals within the test pits, detailed on Figure A1. A qualified member of CGT's geological staff collected the samples and logged the soils in general accordance with the Visual-Manual Procedure (ASTM D2488). An explanation of the classification system is presented on the attached Figure A2. The grab samples were stored in sealable plastic bags and transported along with the tube samples to our soils laboratory for further examination and testing. Our geotechnical staff visually examined all samples in order to refine the initial field classifications.

### A.1.4 Subsurface Conditions

Subsurface conditions are summarized in Section 2.3 of the geotechnical report. Detailed logs of the explorations are presented on the attached exploration logs, Figures A3 through A8.

## A.2.0 LABORATORY TESTING

Laboratory testing was performed on samples collected in the field to refine our initial field classifications and determine in-situ parameters. Laboratory testing on selected samples included the following:

- Eight moisture content determinations (ASTM D2216)
- Two Atterberg limits (plasticity) tests (ASTM D4318)
- One percentage passing the U.S. Standard No. 200 Sieve test (ASTM D1140)

Results of the laboratory tests are shown on the exploration logs.

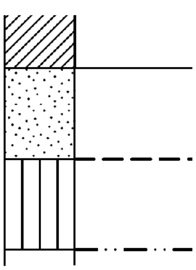
| ODOT SKYLINE STORAGE SHED - PORTLAND, OREGON<br>Project Number G2005344 | FIGURE A1<br>Exploration Key |
|---|---|

| | |
|---|---|
| PL — MC — LL | Atterberg limits (plasticity) test results (ASTM D4318): PL = Plastic Limit, LL = Liquid Limit, and MC= Moisture Content (ASTM D2216) |
| ☐ FINES CONTENT (%) | Percentage passing the U.S. Standard No. 200 Sieve (ASTM D1140) |

## SAMPLING

| | |
|---|---|
| GRAB | Grab sample |
| BULK | Bulk sample |
| SPT | **Standard Penetration Test** (SPT) consists of driving a 2-inch, outside-diameter, split-spoon sampler into the undisturbed formation with repeated blows of a 140-pound, hammer falling a vertical distance of 30 inches (ASTM D1586). The number of blows (N-value) required to drive the sampler the last 12 inches of an 18-inch sample interval is used to characterize the soil consistency or relative density. The drill rig was equipped with an cat-head or automatic hammer to conduct the SPTs. The observed N-values, hammer efficiency, and $N_{60}$ are noted on the boring logs. |
| MC | **Modified California** sampling consists of 3-inch, outside-diameter, split-spoon sampler (ASTM G3550) driven similarly to the SPT sampling method described above. A sampler diameter correction factor of 0.44 is applied to calculate the equivalent SPT $N_{60}$ value per Lacroix and Horn, 1973. |
| CORE | **Rock Coring** interval |
| SH | **Shelby Tube** is a 3-inch, inner-diameter, thin-walled, steel tube push sampler (ASTM D1587) used to collect relatively undisturbed samples of fine-grained soils. |
| WDCP | **Wildcat Dynamic Cone Penetrometer** (WDCP) test consists of driving 1.1-inch diameter, steel rods with a 1.4-inch diameter, cone tip into the ground using a 35-pound drop hammer with a 15-inch free-fall height. The number of blows required to drive the steel rods is recorded for each 10 centimeters (3.94 inches) of penetration. The blow count for each interval is then converted to the corresponding SPT $N_{60}$ values. |
| DCP | **Dynamic Cone Penetrometer** (DCP) test consists of driving a 20-millimeter diameter, hardened steel cone on 16-millimeter diameter steel rods into the ground using a 10-kilogram drop hammer with a 460-millimeter free-fall height. The depth of penetration in millimeters is recorded for each drop of the hammer. |
| POCKET PEN. (tsf) | **Pocket Penetrometer** test is a hand-held instrument that provides an approximation of the unconfined compressive strength in tons per square foot (tsf) of cohesive, fine-grained soils. |

## CONTACTS



Observed (measured) contact between soil or rock units.

Inferred (approximate) contact between soil or rock units.

Transitional (gradational) contact between soil or rock units.

## ADDITIONAL NOTATIONS

| | |
|---|---|
| *Italics* | Notes drilling action or digging effort |
| { Braces } | Interpretation of material origin/geologic formation (e.g. { Base Rock } or { Columbia River Basalt }) |

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*All measurements are approximate.*

# *ODOT SKYLINE STORAGE SHED - PORTLAND, OREGON*
## *Project Number G2005344*

**FIGURE A2**
**Soil Classification**

| Classification of Terms and Content |
|---|
| NAME: Group Name and Symbol<br>Relative Density or Consistency<br>Color<br>Moisture Content<br>Plasticity<br>Other Constituents<br>Other: Grain Shape, Approximate Gradation<br>Organics, Cement, Structure, Odor, etc.<br>Geologic Name or Formation |

| Grain Size | | U.S. Standard Sieve |
|---|---|---|
| Fines | | <#200 (0.075 mm) |
| Sand | Fine<br>Medium<br>Coarse | #200 - #40 (0.425 mm)<br>#40 - #10 (2 mm)<br>#10 - #4 (4.75) |
| Gravel | Fine<br>Coarse | #4 - 0.75 inch<br>0.75 inch - 3 inches |
| Cobbles | | 3 to 12 inches |
| Boulders | | > 12 inches |

## Coarse-Grained (Granular) Soils

**Relative Density**

| SPT $N_{60}$-Value | Density |
|---|---|
| 0 - 4 | Very Loose |
| 4 - 10 | Loose |
| 10 - 30 | Medium Dense |
| 30 - 50 | Dense |
| >50 | Very Dense |

**Minor Constituents**

| Percent by Volume | Descriptor | Example |
|---|---|---|
| 0 - 5% | "Trace" as part of soil description | "trace silt" |
| 5 - 15% | "With" as part of group name | "**POORLY GRADED SAND WITH SILT**" |
| 15 - 49% | Modifier to group name | "**SILTY SAND**" |

## Fine-Grained (Cohesive) Soils

| SPT $N_{60}$-Value | Torvane tsf Shear Strength | Pocket Pen tsf Unconfined | Consistency | Manual Penetration Test |
|---|---|---|---|---|
| <2 | <0.13 | <0.25 | Very Soft | Thumb penetrates more than 1 inch |
| 2 - 4 | 0.13 - 0.25 | 0.25 - 0.50 | Soft | Thumb penetrates about 1 inch |
| 4 - 8 | 0.25 - 0.50 | 0.50 - 1.00 | Medium Stiff | Thumb penetrates about ¼ inch |
| 8 - 15 | 0.50 - 1.00 | 1.00 - 2.00 | Stiff | Thumb penetrates less than ¼ inch |
| 15 - 30 | 1.00 - 2.00 | 2.00 - 4.00 | Very Stiff | Readily indented by thumbnail |
| >30 | >2.00 | >4.00 | Hard | Difficult to indent by thumbnail |

**Minor Constituents**

| Percent by Volume | Descriptor | Example |
|---|---|---|
| 0 - 5% | "Trace" as part of soil description | "trace fine-grained sand" |
| 5 - 15% | "Some" as part of soil description | "some fine-grained sand" |
| 15 - 30% | "With" as part of group name | "**SILT WITH SAND**" |
| 30 - 49% | Modifier to group name | "**SANDY SILT**" |

## Moisture Content

Dry: Absence of moisture, dusty, dry to the touch

Moist: Leaves moisture on hand

Wet: Visible free water, likely from below water table

| | Plasticity | Dry Strength | Dilatancy | Toughness |
|---|---|---|---|---|
| **ML** | Non to Low | Non to Low | Slow to Rapid | Low, can't roll |
| **CL** | Low to Medium | Medium to High | None to Slow | Medium |
| **MH** | Medium to High | Low to Medium | None to Slow | Low to Medium |
| **CH** | Medium to High | High to Very High | None | High |

## Structure

Stratified: Alternating layers of material or color >6 mm thick

Laminated: Alternating layers < 6 mm thick

Fissured: Breaks along definite fracture planes

Slickensided: Striated, polished, or glossy fracture planes

Blocky: Cohesive soil that can be broken down into small angular lumps which resist further breakdown

Lenses: Has small pockets of different soils, note thickness

Homogeneous: Same color and appearance throughout

## Visual-Manual Classification

| Major Divisions | | | Group Symbols | Typical Names |
|---|---|---|---|---|
| Coarse Grained Soils: More than 50% retained on No. 200 sieve | Gravels: 50% or more *retained* on the No. 4 sieve | Clean Gravels | GW | Well-graded gravels and gravel/sand mixtures, little or no fines |
| | | | GP | Poorly-graded gravels and gravel/sand mixtures, little or no fines |
| | | Gravels with Fines | GM | Silty gravels, gravel/sand/silt mixtures |
| | | | GC | Clayey gravels, gravel/sand/clay mixtures |
| | Sands: More than 50% *passing* the No. 4 sieve | Clean Sands | SW | Well-graded sands and gravelly sands, little or no fines |
| | | | SP | Poorly-graded sands and gravelly sands, little or no fines |
| | | Sands with Fines | SM | Silty sands, sand/silt mixtures |
| | | | SC | Clayey sands, sand/clay mixtures |
| Fine-Grained Soils: 50% or more Passes No. 200 Sieve | Silt and Clays Low Plasticity Fines | | ML | Inorganic silts, rock flour, clayey silts |
| | | | CL | Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, lean clays |
| | | | OL | Organic soil of low plasticity |
| | Silt and Clays High Plasticity Fines | | MH | Inorganic silts, clayey silts |
| | | | CH | Inorganic clays of high plasticity, fat clays |
| | | | OH | Organic soil of medium to high plasticity |
| Highly Organic Soils | | | PT | Peat, muck, and other highly organic soils |

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References:

*ASTM D2487 Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System)*

*ASTM D2488 Standard Practice for Description and Identification of Soils (Visual-Manual Procedure)*

*Terzaghi, K., and Peck, R.B., 1948, Soil Mechanics in Engineering Practice, John Wiley & Sons.*

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# FIGURE A3

## Test Pit TP-01



**CLIENT** ODOT
**PROJECT NAME** ODOT Skyline Storage Shed
**PROJECT NUMBER** G2005344
**PROJECT LOCATION** 2500 NW Cornelius Pass Road, Portland, Oregon
**DATE STARTED** 8/28/20
**GROUND ELEVATION** 588.5 ft
**ELEVATION DATUM** Topographic contours shown on Figure 2
**WEATHER** Sunny, 60°F
**SURFACE** Grassy
**LOGGED BY** BLN
**REVIEWED BY** B. Wilcox
**EXCAVATION CONTRACTOR** CGT
**SEEPAGE** ---
**EQUIPMENT** John Deere 85G
**GROUNDWATER DURING DRILLING** ---
**EXCAVATION METHOD** Test Pit & Infiltration Test
**GROUNDWATER AFTER EXCAVATION** ---

| ELEVATION (ft) | GRAPHIC LOG | GROUP SYMBOL | MATERIAL DESCRIPTION | GROUNDWATER | DEPTH (ft) | SAMPLE TYPE NUMBER | RECOVERY % (RQD) | WDCP $N_{60}$ VALUE | POCKET PEN. (tsf) | DRY UNIT WT. (pcf) | ▲ WDCP $N_{60}$ VALUE ▲ / PL — MC — LL / ☐ FINES CONTENT (%) ☐ (0 20 40 60 80 100) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | ML | **SILT:** Very stiff, light gray-brown, damp to moist, low plasticity, trace subrounded to rounded gravel up to ¼-inch in diameter, and scattered fine roots. Upper 1 foot heavily rooted with fine roots. | | 0 | | | | | | |
| 588 | | | | | | | | | 2.5 | | |
| | | | | | 1 | | | | 3.0 | | |
| 587 | | | | | | | | | 3.0 | | |
| | | CL | **LEAN CLAY:** Very stiff, brown with orange and tan mottling, moist, exhibited low to medium plasticity, trace fine-grained sand. | | 2 | | | | 2.5 | | |
| 586 | | | | | | | | | 3.0 | | |
| | | | | | 3 | | | | 2.25 | | |
| 585 | | | | | | | | | 3.5 | | |
| | | | | | 4 | GRAB 1 | | | | | MC 26; Fines 96 |
| 584 | | | • Test pit terminated at about 4 feet bgs.<br>• Infiltration test performed at 4 feet bgs. See report text for test results.<br>• No groundwater or caving encountered.<br>• Test pit loosely backfilled with excavated materials. | | | | | | | | |

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# FIGURE A4

## Test Pit TP-02



**CLIENT** ODOT
**PROJECT NAME** ODOT Skyline Storage Shed
**PROJECT NUMBER** G2005344
**PROJECT LOCATION** 2500 NW Cornelius Pass Road, Portland, Oregon
**DATE STARTED** 8/28/20
**GROUND ELEVATION** 585.7 ft
**ELEVATION DATUM** Topographic contours shown on Figure 2
**WEATHER** Sunny, 62°F
**SURFACE** Grassy
**LOGGED BY** BLN
**REVIEWED BY** B. Wilcox
**EXCAVATION CONTRACTOR** CGT
**SEEPAGE** ---
**EQUIPMENT** John Deere 85G
**GROUNDWATER DURING DRILLING** ---
**EXCAVATION METHOD** Test Pit
**GROUNDWATER AFTER EXCAVATION** ---

| ELEVATION (ft) | GRAPHIC LOG | GROUP SYMBOL | MATERIAL DESCRIPTION | GROUNDWATER | DEPTH (ft) | SAMPLE TYPE NUMBER | RECOVERY % (RQD) | WDCP $N_{60}$ VALUE | POCKET PEN. (tsf) | DRY UNIT WT. (pcf) | ▲ WDCP $N_{60}$ VALUE ▲ / PL MC LL / □ FINES CONTENT (%) □ (0 20 40 60 80 100) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | ML | **SILT:** Very stiff, light gray-brown, damp to moist, low to medium plasticity, trace fine-grained sand, and some fine roots. Upper 6 inches heavily rooted with fine roots. | | 0 | | | | 3.0 | | |
| | | | | | | | | | 2.0 | | |
| 584 | | | | | | | | | 3.0 | | |
| | | | | | 2 | GRAB 1 | | | 4.0 | | 14 |
| | | CL | **LEAN CLAY:** Very stiff, brown with orange and tan mottling, moist, exhibited low to medium plasticity, trace fine-grained sand. | | | | | | 3.5 | | |
| | | | | | | | | | 4.0 | | |
| 582 | | | | | | | | | 3.75 | | |
| | | | | | 4 | GRAB 2 | | | 4.0 | | 22 |
| 580 | | | | | | | | | | | |
| | | | Black, gray, orange, and tan mottling below about 6 feet bgs. | | 6 | | | | | | |
| 578 | | | | | | GRAB 3 | | | | | |
| | | | | | 8 | | | | | | |
| 576 | | | | | | GRAB 4 | | | | | 27 |
| | | | | | 10 | | | | | | |

- Test pit terminated at about 10 feet bgs.
- No groundwater or caving encountered.
- Test pit loosely backfilled with excavated materials.

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# FIGURE A5

## Test Pit TP-03



**CLIENT** ODOT **PROJECT NAME** ODOT Skyline Storage Shed
**PROJECT NUMBER** G2005344 **PROJECT LOCATION** 2500 NW Cornelius Pass Road, Portland, Oregon
**DATE STARTED** 8/28/20 **GROUND ELEVATION** 590.4 ft **ELEVATION DATUM** Topographic contours shown on Figure 2
**WEATHER** Sunny, 64°F **SURFACE** Grassy **LOGGED BY** BLN **REVIEWED BY** B. Wilcox
**EXCAVATION CONTRACTOR** CGT **SEEPAGE** ---
**EQUIPMENT** John Deere 85G **GROUNDWATER DURING DRILLING** ---
**EXCAVATION METHOD** Test Pit **GROUNDWATER AFTER EXCAVATION** ---

| ELEVATION (ft) | GROUP SYMBOL | MATERIAL DESCRIPTION | DEPTH (ft) | SAMPLE TYPE NUMBER | POCKET PEN. (tsf) | PL / MC / LL; FINES CONTENT (%) |
|---|---|---|---|---|---|---|
| 590 | ML | **SILT:** Stiff to very stiff, light gray-brown, damp to moist, low plasticity, trace fine-grained sand, and scattered fine roots. Upper 6 inches heavily rooted with fine roots. | 0–2 | | 2.0, 2.5, 3.0 | |
| 588 | CL | **LEAN CLAY:** Very stiff, brown with orange, gray, tan, and black mottling, moist, exhibited low to medium plasticity, trace fine-grained sand. | 2–5 | | 2.5, 2.75, 3.0, 2.5, 3.5 | |
| 585 | CL | | 5–5.5 | GRAB 1 | | PL 19, MC 23, LL 49 |
| 582 | CL | | 8–8.5 | GRAB 2 | | MC 22 |

• Test pit terminated at about 8½ feet bgs.
• No groundwater or caving encountered.
• Test pit loosely backfilled with excavated materials.

# FIGURE A6

## Test Pit TP-04

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**CLIENT** ODOT
**PROJECT NAME** ODOT Skyline Storage Shed
**PROJECT NUMBER** G2005344
**PROJECT LOCATION** 2500 NW Cornelius Pass Road, Portland, Oregon
**DATE STARTED** 8/28/20
**GROUND ELEVATION** 588.5 ft
**ELEVATION DATUM** Topographic contours shown on Figure 2
**WEATHER** Sunny, 65°F
**SURFACE** Grassy
**LOGGED BY** BLN
**REVIEWED BY** B. Wilcox
**EXCAVATION CONTRACTOR** CGT
**SEEPAGE** ---
**EQUIPMENT** John Deere 85G
**GROUNDWATER DURING DRILLING** ---
**EXCAVATION METHOD** Test Pit
**GROUNDWATER AFTER EXCAVATION** ---

| ELEVATION (ft) | GRAPHIC LOG | GROUP SYMBOL | MATERIAL DESCRIPTION | GROUNDWATER | DEPTH (ft) | SAMPLE TYPE NUMBER | RECOVERY % (RQD) | WDCP $N_{60}$ VALUE | POCKET PEN. (tsf) | DRY UNIT WT. (pcf) | ▲ WDCP $N_{60}$ VALUE ▲ / PL ● MC LL / ☐ FINES CONTENT (%) ☐ (0 20 40 60 80 100) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 588 | | ML | **SILT:** Stiff to very stiff, light gray-brown, damp to moist, low to medium plasticity, trace fine-grained sand. Upper 1½ inches heavily rooted with fine roots. | | 0 | | | | 1.5 | | |
| | | | | | | | | | 1.75 | | |
| | | | Scattered fine roots below about 1½ feet bgs. | | | | | | 3.0 | | |
| | | | | | 2 | | | | 2.5 | | |
| 586 | | | | | | | | | 3.0 | | |
| | | CL | **LEAN CLAY:** Very stiff, brown with orange and tan mottling, moist, exhibited medium plasticity, trace fine-grained sand. | | | GRAB 1 | | | 3.0 | | |
| | | | | | | | | | 2.75 | | |
| | | | Light gray-brown below about 4 feet bgs. | | 4 | | | | 3.5 | | |
| 584 | | | | | | | | | | | |
| 582 | | | | | 6 | GRAB 2 | | | | | ● 25 |
| 580 | | | | | 8 | | | | | | |
| | | | | | | GRAB 3 | | | | | |
| 578 | | | | | | | | | | | |

• Test pit terminated at about 9½ feet bgs.
• No groundwater or caving encountered.
• Test pit loosely backfilled with excavated materials.

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# FIGURE A7

## Test Pit TP-05



**CLIENT** ODOT
**PROJECT NAME** ODOT Skyline Storage Shed
**PROJECT NUMBER** G2005344
**PROJECT LOCATION** 2500 NW Cornelius Pass Road, Portland, Oregon

**DATE STARTED** 8/28/20 **GROUND ELEVATION** 592.0 ft
**ELEVATION DATUM** Topographic contours shown on Figure 2
**WEATHER** Sunny, 66°F **SURFACE** Grassy
**LOGGED BY** BLN **REVIEWED BY** B. Wilcox
**EXCAVATION CONTRACTOR** CGT
**SEEPAGE** ---
**EQUIPMENT** John Deere 85G
**GROUNDWATER DURING DRILLING** ---
**EXCAVATION METHOD** Test Pit
**GROUNDWATER AFTER EXCAVATION** ---

| ELEVATION (ft) | GRAPHIC LOG | GROUP SYMBOL | MATERIAL DESCRIPTION | GROUNDWATER | DEPTH (ft) | SAMPLE TYPE NUMBER | RECOVERY % (RQD) | WDCP $N_{60}$ VALUE | POCKET PEN. (tsf) | DRY UNIT WT. (pcf) | ▲ WDCP $N_{60}$ VALUE ▲ / PL — MC — LL / ☐ FINES CONTENT (%) ☐ (0 20 40 60 80 100) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | ML | **SILT:** Medium stiff to stiff, light gray-brown, damp to moist, low plasticity, trace fine-grained sand and subrounded to rounded gravel up to ¼-inch in diameter, and abundant fine roots. | | 0 | | | | 1.0 | | |
| 591 | | | | | 1 | | | | 1.25 | | |
| | | CL | **LEAN CLAY:** Very stiff, light gray-brown with orange mottling, moist, exhibited low to medium plasticity, trace fine-grained sand. | | | | | | 1.0 | | |
| 590 | | | | | 2 | | | | 3.5 | | |
| | | | Very stiff below about 2½ feet bgs. | | | GRAB 1 | | | 3.0 | | PL 21, MC 17, LL 34 |
| 589 | | | | | 3 | | | | 4.0 | | |
| | | | | | | | | | 4.0 | | |
| 588 | | | | | 4 | | | | 3.25 | | |
| 587 | | | • Test pit terminated at about 4½ feet bgs.<br>• No groundwater or caving encountered.<br>• Test pit loosely backfilled with excavated materials. | | | | | | | | |

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# FIGURE A8

## Test Pit TP-06



**CLIENT** ODOT
**PROJECT NAME** ODOT Skyline Storage Shed
**PROJECT NUMBER** G2005344
**PROJECT LOCATION** 2500 NW Cornelius Pass Road, Portland, Oregon
**DATE STARTED** 8/28/20 **GROUND ELEVATION** 582.3 ft
**ELEVATION DATUM** Topographic contours shown on Figure 2
**WEATHER** Sunny, 70°F **SURFACE** Grassy
**LOGGED BY** BLN **REVIEWED BY** B. Wilcox
**EXCAVATION CONTRACTOR** CGT
**SEEPAGE** ---
**EQUIPMENT** John Deere 85G
**GROUNDWATER DURING DRILLING** ---
**EXCAVATION METHOD** Test Pit
**GROUNDWATER AFTER EXCAVATION** ---

| ELEVATION (ft) | GRAPHIC LOG | GROUP SYMBOL | MATERIAL DESCRIPTION | GROUNDWATER | DEPTH (ft) | SAMPLE TYPE NUMBER | RECOVERY % (RQD) | WDCP $N_{60}$ VALUE | POCKET PEN. (tsf) | DRY UNIT WT. (pcf) | ▲ WDCP $N_{60}$ VALUE ▲ / PL MC LL / ☐ FINES CONTENT (%) ☐ (0 20 40 60 80 100) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 582 | | ML | **SILT:** Stiff to very stiff, light gray, damp to moist, low plasticity, trace fine-grained sand. Upper 1 foot heavily rooted with fine roots. | | 0 | | | | 1.5 | | |
| 581 | | ML | Very stiff below about 1 foot bgs. | | 1 | | | | 3.25 | | |
| | | ML | Orange and tan mottling below about 1½ feet bgs. | | | GRAB 1 | | | 4.0 | | |
| 580 | | ML | | | 2 | | | | 3.5 | | |
| | | CL | **LEAN CLAY:** Very stiff, brown with orange and tan mottling, moist, exhibited low to medium plasticity, trace fine-grained sand. | | | | | | 2.75 | | |
| 579 | | CL | | | 3 | GRAB 2 | | | 3.5 | | MC 25 |
| | | CL | | | | | | | 3.0 | | |
| 578 | | | | | 4 | | | | 2.5 | | |
| 577 | | | • Test pit terminated at about 4 feet bgs.<br>• No groundwater or caving encountered.<br>• Test pit loosely backfilled with excavated materials. | | | | | | | | |

# Appendix B: Results of Infiltration Testing

## ODOT Skyline Storage Shed
## Tax Lot 2500, NW Cornelius Pass Road
## Portland, Oregon

## CGT Project Number G2005344

September 28, 2020

*Prepared For:*

Mr. Luis Umana
Oregon Department of Transportation (ODOT)
3700 SE 92nd Avenue
Portland, Oregon, 97266

*Prepared by*
**Carlson Geotechnical**

## B.1.0 INTRODUCTION

Mr. Luis Umana of ODOT requested infiltration testing at one location on a site map provided to CGT, at depths of about 4 feet below ground surface (bgs). The test was performed in test pit excavation, designated TP-1 on the Site Plan (IT-1), which is attached to the main report as Figure 2.

## B.2.0 TEST PROCEDURE

The infiltration tests were performed in general accordance with the Encased Falling Head Infiltration Test method as described in Chapter 2.3.6 of the 2016 City of Portland Stormwater Management Manual (PSWMM).

The test pit excavation was advanced to the test depth using a John Deere 85G excavator equipped with a 2-foot wide, toothed bucket. A 6-inch inner-diameter PVC pipe was inserted 6 inches into the exposed soil at the base of the test pit to achieve an adequate seal. The lower 2 inches of the PVC pipe were filled with open-graded gravel fill up to about ¾-inch in diameter to prevent scouring.

The test pipe was filled with about 12 inches of water, and the subsurface soils were allowed to soak for 4 hours in accordance with the referenced test method. After the soaking period, about 12 inches of water remained in the pipe. The test was discontinued due to no change in the water level during the 4 hour soaking period.

### B.2.1 Infiltration Test Results

The following table presents the raw data and calculated rates of infiltration that we observed from the infiltration tests. Please note the calculated infiltration rates do <u>not</u> include any safety or correction factors.

| | | | | | |
|---|---|---|---|---|---|
| **Location:** | See Site Plan (Figure 2) | **Date:** | June 10, 2020 | **Infiltration Test:** | IT-1 |
| **Test Method:** | 2016 PSWMM Encased Falling Head | **Inner Diameter of Pipe:** | 6 inches | **Infiltration Test Depth:** | 4 feet |
| **Soil at infiltration test depth:** | Lean Clay (CL) | | see exploration log for detail | | |
| **Presaturation Start Time:** | 8:45 a.m. | **Presaturation Notes:** | No change in water level during soaking period | | |
| **Presaturation End Time:** | 12:45 p.m. | | | | |
| Test Terminated due to no drop in water level during 4 hours soaking period (infiltration rate of zero) | | | | | |

## B.3.0 DISCUSSION

As indicated in the preceding section, no change in water level was observed during the soaking period, resulting in an infiltration rate of zero at the test location. Due to the zero infiltration rates observed during testing, we anticipate infiltrating stormwater at this project site will be ineffective. We recommend stormwater collected from new impervious areas of the site be routed to the public stormwater system or other suitable discharge point, if available.

If alternative infiltration locations and/or greater bottom depths are considered at the site, CGT recommends supplemental field investigation and testing be performed. CGT would be pleased to perform supplemental field investigation and testing for an additional fee, upon request.

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# Appendix C: Engineering Geologic Report

## ODOT Skyline Storage Shed
## Tax Lot 2500, NW Cornelius Pass Road
## Multnomah County, Oregon

## CGT Project Number G2005344

September 28, 2020

*Prepared For:*

Luis Umana
Oregon Department of Transportation (ODOT)
3700 SE 92nd Avenue
Portland, Oregon, 97266

*Prepared by*
**Carlson Geotechnical**

CERTIFIED
OREGON
RYAN T. HOUSER
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## TABLE OF CONTENTS



## ATTACHMENTS



Multnomah County Geologic Hazards Permit (GHP) Form 1

Doc ID: G:\GEOTECH\PROJECTS\2020 Projects\G2005344 - ODOT Skyline Storage Shed\G2005344 - GEO\008 - Deliverables\Appendix C - Engineering Geology Report\Appendix C - Engineering Geologic Report.docx

## C.1.0 INTRODUCTION

The southern portion of the site is located within the Multnomah County Slope Hazard Overlay, which requires a Geologic Hazards Permit (GHP) be completed. To complete the GHP, an engineering geologic report is required for the proposed project. A portion of the Slope Hazard Overlay is shown on the attached Figure C1, which also shows the approximate location of the proposed ODOT Skyline Storage Shed project. Our specific scope of services included the following:

- Review available literature for geologic hazards in the vicinity of the site. Specific hazards addressed by this study include:
    - Erosion potential
    - Landslide potential / Slope stability
    - Seismic potential
    - Flood potential
    - Volcanic hazards potential
- Review available topographic, geologic, and geologic hazard maps for the area.
- Perform a surface reconnaissance of the site.
- Review subsurface explorations performed as part of the geotechnical report.
- Detail geologic hazards that may affect the proposed land use.
- Provide an opinion regarding the geologic feasibility of the site for the proposed development, including a **qualitative** conclusion regarding the effects of the geologic conditions on the proposed land use, the effects of the proposed land use on future geologic processes, and the effects of the geologic conditions and proposed land use on surrounding properties.
- Provide recommendations for hazard mitigation.
- Provide a written report summarizing the results of our engineering geologic reconnaissance in general accordance with Multnomah County Code Sections 39.5052(C)(3)(c) and 38.5515(C)(3)(c) and the 2014 State of Oregon Guideline for Preparing Engineering Geologic Reports.
- Complete the Multnomah County Geologic Hazards Permit (GHP) Form 1, which is attached at the end of this appendix.

## C.2.0 GEOLOGY

Surficial geology is mapped as windblown sediments, consisting primarily of silt and clay[1]. The windblown sediments are generally about 20 to 30 feet thick in this portion of the Tualatin Mountains (Portland West Hills). These soils are susceptible to soil creep on moderate to steep slopes, and are susceptible to loss of strength, landslides, and flow failures, particularly when subjected to heavy rainfall or concentrated runoff. Bedrock in the area is mapped as the Sentinel Bluffs member of the Miocene Columbia River Basalt Group[2], which extends to hundreds of feet below ground surface.

---

[1] Ma, Lina, Madin, Ian P., et al., 2012. Lidar-based Surficial Geologic Map and Database of the Greater Portland, Oregon, Area. Oregon Department of Geology and Mineral Industries Open-File Report O-12-02.

[2] DOGAMI, 2006. Preliminary Geologic Map of the Linnton 7.5' Quadrangle, Multnomah and Washington Counties, Oregon. Oregon Department of Geology and Mineral Industries Open-File Report O-08-06.

A portion of the surficial geologic map is attached as Figure C2. A portion of the bedrock geologic map is attached as Figure C3.

## C.3.0 SEISMICITY

The site is located in a tectonically and seismically active area that may be affected by earthquakes generated by crustal and subduction zone sources.

### C.3.1 Earthquake Sources

C.3.1.1 <u>Crustal Sources</u>

Crustal earthquakes typically occur at depths ranging from 15 to 40 kilometers bgs[3]. According to the United States Geological Survey Quaternary fault and fold database[4], nearby seismic sources capable of producing damaging earthquakes in this region include Oatfield fault, Portland Hills fault, East Bank fault, Helvetia fault, Beaverton fault zone, Canby-Molalla fault, Gales Creek fault zone, Grant Butte fault, Damascus-Tickle Creek fault, and the Lacamas Lake fault. Quaternary faults in the vicinity of the site are shown on the attached Figure C4, and are summarized in the following table.

**Table C1 Known Active or Potentially Active Crustal Faults in the Vicinity of the Site**

| USGS Fault No. | Fault Name | Distance and Direction from Site | USGS Fault Class[1] |
|---|---|---|---|
| 875 | Oatfield fault | 1.75 km E | A |
| 877 | Portland Hills fault | 4 km ENE | A |
| 876 | East Bank fault | 5.5 km ENE | A |
| 714 | Helvetia fault | 7 km WSW | A |
| 715 | Beaverton fault zone | 15 km S | A |
| 716 | Canby-Molalla fault | 20 km SSE | A |
| 718 / OR1 | Gales Creek fault zone | 24 km WSW | A |
| 878 | Grant Butte fault | 28 km SE | A |
| 879 | Damascus-Tickle Creek fault | 28 km SE | A |
| 880 | Lacamas Lake fault | 28 km ENE | A |

1 USGS Fault Classes from USGS Earthquake Hazards Program, 2008 National Seismic Hazard Maps
Class A: Fault with convincing evidence of Quaternary activity (ACTIVE)
Class B: Fault that requires further study in order to confidently define their potential as possible sources of earthquake-induced ground motion (POTENTIALLY ACTIVE)
Class C: Fault with insufficient evidence for Quaternary activity (LOW POTENTIAL FOR ACTIVITY)

*C.3.1.1.1* *<u>Helvetia fault (USGS 714)</u>*

The Helvetia fault is a north-northwest trending structure located on the northeastern margin of the Tualatin Basin. There is no evidence for displacement of late Quaternary deposits along the fault; however, the most

[3] Geomatrix Consultants, 1995. Seismic Design Mapping, State of Oregon: unpublished report prepared for Oregon Department of Transportation, Personal Services Contract 11688, January 1995.

[4] U.S. Geological Survey, 2020. Quaternary fault and fold database for the United States, *accessed September 2020*, from USGS web site: http://earthquakes.usgs.gov/regional/qfaults/.

recent age of displacement is poorly constrained[5]. Therefore, the fault is considered active, but with a long recurrence interval.

*C.3.1.1.2* *Beaverton fault zone (USGS 715)*

The Beaverton fault zone consists of an east-west striking normal fault that forms the southern margin of the Tualatin basin. This fault offsets Miocene Columbia River Basalt, but is covered by thick sequences of Pliocene to Pleistocene Missoula flood deposits. As a result, no fault scarp is present at the surface, and the Beaverton fault zone is not present on most geologic maps of the area. Yeats and others[6] indicate that the Beaverton Faults displace post-Columbia River Basalt sediments; however, the age and nature of deformation is not known. The Beaverton fault is considered active, but with a long recurrence interval.

*C.3.1.1.3* *Canby-Molalla fault (USGS 716)*

The Canby-Molalla fault is a right-lateral strike-slip fault located within the Willamette Valley. The Canby-Molalla fault appears to offset Missoula flood deposits, and seismic reflection surveys suggest Holocene deformation of sediments. The fault has little geomorphologic expression, but is considered active, with a slip rate of less than 0.2 mm per year.

*C.3.1.1.4* *Gales Creek fault zone (USGS 718)*

The Gales Creek fault zone is a 73-kilometer-long zone of northwest-trending right-lateral strike-slip faults located on the western margin of the Willamette Valley. The fault zone offsets Miocene Columbia River basalts, but no unequivocal evidence for Quaternary displacement has been identified. However, the majority of the faults are covered with very recent alluvium, which may have buried evidence of recent deformation. Estimates for the latest movements along the Gales Creek fault zone typically predate the late Pleistocene; in other words, the fault has not had activity within the last approximately 30,000 years. The recurrence interval for the Gales Creek fault zone is likely greater than 50,000 years, based on the information available.

*C.3.1.1.5* *Oatfield fault (USGS 875)*

The Oatfield fault consists of a 29-kilometer-long steeply dipping reverse fault that forms escarpments in Miocene Columbia River Basalt in the Tualatin Mountains. No fault scarps or displacement of surficial deposits have been described, but exposures within tunnels show offset of Boring Lava, indicating Quaternary activity. The slip rate for the Oatfield fault has been calculated to be about 0.1 mm per year based on the tunnel exposures. Given the very low slip rate and lack of displacement of surficial deposits, this fault is considered to have a very long recurrence interval.

*C.3.1.1.6* *East Bank fault (USGS 876)*

The East Bank fault consists of a 29-kilometer-long steeply dipping reverse fault that parallels the Portland Hills fault. No Quaternary surficial fault scarps have been identified, and the fault is largely buried by thick sequences of Pleistocene Missoula flood deposits. Recent shallow seismic reflection data suggest subsurface displacement of the older Missoula flood deposits (before 12,700 years ago). Therefore, the fault is considered to be active, with a low slip rate and a very long recurrence interval.

---

[5] Geomatrix Consultants, 1995. Seismic Design Mapping, State of Oregon: Final Report to Oregon Department of Transportation, Project No. 2442.

[6] Yeats, R.S., *et al*., 1996. Tectonics of the Willamette Valley Oregon: in Assessing earthquake hazards and reducing risk in the Pacific Northwest, v. 1: U.S. Geological Survey Professional Paper 1560, p. 183-222, 5 plates, scale 1:100,000.

#### *C.3.1.1.7 <u>Portland Hills fault (USGS 877)</u>*

The Portland Hills fault zone is a series of northwest-trending faults forming the northeastern margin of the Tualatin Mountains. The faults associated with this structural zone vertically displace the Columbia River Basalt Group by 1,130 feet, and appear to control thickness changes in late Pleistocene sediment[7]. Geomorphic lineaments suggestive of Pleistocene deformation have been identified within the fault zone, but none of the fault segments has been shown to cut Holocene deposits[8,9]. The fact that the faults do not cut Holocene sediments is most likely a result of the faulting being related to a time of intense uplift of the Oregon Coast Range during the Miocene, and little to no movement along the faults during the Holocene.

Recent studies of this fault[10] concluded that the Portland Hills fault is active, based on contemporary seismicity in the vicinity of the fault, and seismic reflection data suggesting that the fault cuts late Pleistocene layered strata. Additionally, in May of 2000, while taking magnetic readings to map the fault, an Oregon Department of Geology and Mineral Industries (DOGAMI) geologist observed folded sediment in a retaining wall cut in North Clackamas Park south of Portland. The folded sediments consisted of sand and silt deposited by Pleistocene floods derived from glacial Lake Missoula approximately 12,800 to 15,000 years ago. An investigation of the folded strata by DOGAMI geologists and engineering consultants showed that the entire sequence of sediment layers is folded and they concluded that this folding is evidence for an active fault beneath the site, and the fault is either the Portland Hills fault, or a closely related structure[11].

#### *C.3.1.1.8 <u>Grant Butte fault (USGS 878)</u>*

The Grant Butte fault forms the southern margin of the Portland basin, and consists of a 10-kilometer-long normal fault. The Grant Butte fault offsets Pliocene-Pleistocene Springwater Formation and Boring Lava. No Quaternary surficial fault scarps have been identified, but the fault is largely buried by thick sequences of Pliocene to Pleistocene Missoula flood deposits. Based on radiometric age dating techniques, the fault has been active within the late Quaternary. Therefore, the Grant Butte fault is considered active with a long recurrence interval.

#### *C.3.1.1.9 <u>Damascus-Tickle Creek fault zone (USGS 879)</u>*

The Damascus-Tickle Creek fault zone consists of numerous relatively short northeast- and northwest-trending faults forming a broad fault zone along the southern edge of the Portland basin. The location of several eruptive vents of the Boring Lava suggest a direct relationship with the Damascus-Tickle Creek fault zone. The majority of the faults within the zone are buried by Pliocene to Pleistocene Missoula flood deposits, however, at least one fault strand may offset the flood deposits.

---

7 Mabey, M.A., Madin, I.P., Youd, T.L., Jones, C.F., 1993, Earthquake hazard maps of the Portland quadrangle, Multnomah and Washington Counties, Oregon, and Clark County, Washington: Oregon Department of Geology and Mineral Industries Geological Map Series GMS-79, Plate 2, 1:24,000.

8 Conforth and Geomatrix Consultants, 1992. Seismic hazard evaluation, Bull Run dam sites near Sandy, Oregon: unpublished report to City of Portland Bureau of Water Works.

9 Balsillie, J.J. and Benson, G.T., 1971. Evidence for the Portland Hills fault: The Ore Bin, Oregon Dept. of Geology and Mineral Industries, v. 33, p. 109-118.

10 Wong *et al.*, 2001. The Portland Hills Fault: An Earthquake Generator or Just Another Old Fault? Published by Oregon Geology, V63, number 2, Spring 2001.

11 Madin and Hemphill-Haley, 2001: The Portland Hills Fault at Rowe Middle School. Oregon Geology V63 p47.

*C.3.1.1.10* *Lacamas Lake fault (USGS 880).*

The Lacamas Lake fault is a northwest-trending structure located in the vicinity of Lacamas Lake, near Camas, Washington, at the northeastern margin of the Portland basin. This fault was originally identified by well-expressed lineaments defined by the relatively steep linear valley margins along both sides of Lacamas Lake[12]. Although recent activity on the Lacamas Lake fault is uncertain, the fault is considered active based on possible displacement of Troutdale sediments, prominent topographic lineaments associated with the fault, and possible associated seismicity. The fault is buried by Pleistocene Missoula flood deposits, suggesting a long recurrence interval.

C.3.1.2 Cascadia Subduction Zone Seismic Sources

The Cascadia Subduction Zone (CSZ) is a 1,100-kilometer-long zone of active tectonic convergence where oceanic crust of the Juan de Fuca Plate is subducting beneath the North American continental plate at a rate of about 3 to 4 centimeters per year[13]. The fault trace is located off of the coast of southern British Columbia, Washington, Oregon, and northern California; approximately 200 kilometers west of the site (see attached Figure C5).

Two primary sources of seismicity are associated with the CSZ: relatively shallow earthquakes that occur on the interface between the two plates (Subduction Zone earthquakes), and deep earthquakes that occur along faults within the subducting Juan de Fuca plate (intraplate earthquakes).

*C.3.1.2.1* *Subduction Zone Earthquakes*

Large subduction zone (megathrust) earthquakes occur within the upper approximate 30 kilometers of the contact between the two plates[14]. As the Juan de Fuca Plate subducts beneath the North American Plate through this zone, the plates are locked together by friction[15]. Stress slowly builds as the plates converge until the frictional resistance is exceeded, and the plates rapidly slip past each other resulting in a "megathrust" earthquake. The United States Geologic Survey estimates megathrust earthquakes on the CSZ may have magnitudes up to M9.2.

Geologic evidence indicates a recurrence interval for major subduction zone earthquakes of 250 to 650 years, with the last major event occurring in 1700[16,17]. The eastern margin of the seismogenic portion of the Cascadia Subduction zone is located approximately 68 kilometers west of the site, as shown on Figure C5.

12 Madin and Hemphill-Haley, 2001: The Portland Hills Fault at Rowe Middle School. Oregon Geology V63 p47.
13 DeMets, C., Gordon, R.G., Argus, D.F., Stein, S., 1990. Current plate motions: Geophysical Journal International, v. 101, p. 425-478.
14 Pacific Northwest Seismic Network, 2020. Pacific Northwest Earthquake Sources Overview, *accessed September 2020,* from PNSN web site, http://pnsn.org/outreach/earthquakesources/.
15 Pacific Northwest Seismic Network, 2020. Pacific Northwest Earthquake Sources Overview, *accessed September 2020,* from PNSN web site, http://pnsn.org/outreach/earthquakesources/.
16 Atwater, B.F., 1992. Geologic evidence for earthquakes during the past 2,000 years along the Copalis River, southern coastal Washington: Journal of Geophysical Research, v. 97, p. 1901-1919.
17 Peterson, C.D., Darienzo, M.E., Burns, S.F., and Burris, W.K., 1993. Field trip guide to Cascadia paleoseismic evidence along the northern California coast: evidence of subduction zone seismicity in the central Cascadia margin. Oregon Department of Geology and Mineral Industries, Oregon Geology, Vol. 55, p. 99-144.

*C.3.1.2.2 Intraplate Earthquakes*

Below about 30 kilometers, the plate interface does not appear to be locked by friction, and the plates slowly slide past each other. The curvature of the subducted plate increases as the advancing edge moves east, creating extensional forces within the plate. Normal faulting occurs in response to these extensional forces. This region of maximum curvature and faulting of the subducting plate is where large intraplate earthquakes are expected to occur, and is located at depths ranging from 30 to 60 kilometers[18,19,20]. Intraplate earthquakes within the Juan de Fuca plate generally have magnitudes less than M7.5[21].

The 2001 M6.8 Nisqually earthquake near Olympia, Washington, occurred within this seismogenic zone at a depth of 52 kilometers. The site is located within the intraplate seismogenic zone, as shown on Figure C5.

### C.3.2 Historic Seismicity

The Pacific Northwest is a seismically active area. Epicenters for historic earthquakes[22] in western Oregon from 1924 to 2002 are shown on Figure C6. The majority of these earthquakes are shallow (crustal) in nature, with a lesser amount of intraplate sources. No large-scale subduction-zone earthquakes occurred during this period.

## C.4.0 LOCAL TOPOGRAPHY

Topography in the vicinity of the site is shown on Figure 1 attached to the geotechnical report. We also reviewed topographic data available at DOGAMI's lidar data viewer website[23] and Metro's RLIS data available from their website[24]. The site is located on a southwest-facing slope just south of Cornelius Pass through the Tualatin Mountains northwest of Portland, Oregon. The site is located at an elevation of approximately 590 feet above mean sea level (MSL). Slope morphology in the vicinity of the site is generally characterized by rounded, convex slopes with incised, dendritic drainages. Slope gradients in the vicinity of the site generally range from about 6 horizontal to 1 vertical (6H:1V) along NW Skyline Boulevard northeast of the site, to about 4H:1V along the drainages south of the site. Site topography observed during our reconnaissance is discussed in detail in Section C.6.1 below.

[18] Geomatrix Consultants, 1995. Seismic Design Mapping, State of Oregon: unpublished report prepared for Oregon Department of Transportation, Personal Services Contract 11688, January 1995.

[19] Geomatrix Consultants, 1993. Seismic margin Earthquake For the Trojan Site: Final Unpublished Report For Portland General Electric Trojan Nuclear Plant, Rainier, Oregon, May 1993.

[20] Kirby, Stephen H., Wang, Kelin, Dunlop, Susan, 2002, The Cascadia Subduction Zone and Related Subduction Systems—Seismic Structure, Intraslab Earthquakes and Processes, and Earthquake Hazards: U.S. Geological Survey Open-File Report 02-328, 182 pp.

[21] Cascadia Region Earthquake Workshop, 2008. Cascadia Deep Earthquakes. Washington Division of Geology and Earth Resources, Open File Report 2008-1.

[22] U.S. Geological Survey, 2020. Earthquake Catalog, *accessed September 2020*, from USGS web site: https://earthquake.usgs.gov/earthquakes/.

[23] Oregon Department of Geology and Mineral Industries, 2020. 2007 Aerial Lidar Survey Data, *accessed September 2020*, from National Oceanic and Atmospheric Administration (NOAA) Data Access Viewer, https://coast.noaa.gov/dataviewer/#/.

[24] Metro Regional Government, 2020. MetroMap Regional Land Information System (RLIS) data, *accessed September 2020,* from Metro website: http://gis.oregonmetro.gov/metromap/.

## C.5.0 HAZARDS

### C.5.1 Landslides

Landsliding is a common hazard in the Pacific Northwest that can be initiated on marginally stable slopes by human disturbances such as grading and deforestation, and by natural processes including earthquake shaking, volcanism, heavy rainfalls, and rapid snow melt. Recent studies indicate that the most common causes for slope failures are intense rainfall and human alteration, including the placement of building loads on slopes, excavating or over-steepening slopes, and the infiltration or diversion of storm water runoff[25]. For example, excavation into the base of marginally stable slopes may reduce forces resisting failure on those slopes, thus causing movement. Adding fill and/or a structure to the top or mid portion of a slope increases the driving forces on a slope and may contribute to failure. Redirecting water onto or into slopes may exploit existing planes of weakness within those slopes, causing failure.

The Statewide Landslide Information Database for Oregon (SLIDO)[26] and Landslide Inventory Maps of the Linnton Quadrangle[27] show that no landslides have been mapped on the site. The closest mapped landslide is located approximately 700 feet southeast of the site on a steep slope on the opposite side of a south-trending drainage. This pre-historic (greater than 150 years old) landslide is mapped as a shallow earthflow with a failure depth of about 7 feet. The geomorphology of the slide suggests it was triggered by erosion acting at the base of the slope within the drainage. A portion of the landslide inventory map is attached as Figure C7.

Much of the SLIDO mapping is based on Light Detection and Ranging (lidar) data and imagery available from the Oregon Department of Geology and Mineral Industries (DOGAMI). We also reviewed the lidar imagery available on the DOGAMI lidar data viewer website[28]. DOGAMI provides contours and bare earth imagery, which has been filtered to remove foliage and buildings. The lidar data portray the topography at a much greater level of detail than traditional mapping methods, and can reveal features that are otherwise difficult to ascertain. In areas where human activity has modified the topography extensively, such as through road-building and general grading, the resulting “background noise” can mask features that might otherwise be apparent. Based on our review of the lidar data, we did not observe any obvious signs of previous landslides at or in the immediate vicinity of the site, with the exception of the mapped landslide southeast of the site.

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[25] Hofmeister, R., Madin, I., Wang, Y., and Hasenberg, C. 2003, Earthquake and Landslide Hazards Maps and Future Earthquake Damage Estimates, Clackamas County, Oregon: Oregon Department of Geology and Mineral Industries, Open File Report OFR 0-03-10.

[26] Oregon Department of Geology and Mineral Industries, 2020. Statewide Landslide Information Database for Oregon (SLIDO), *accessed September 2020* , from DOGAMI web site: http://www.oregongeology.org/sub/slido/index.htm.

[27] Burns, William J., Duplantis, Serin, and Mickelson, Katherine A., 2010. Landslide Inventory Maps of the Sauvie Island Quadrangle, Columbia and Multnomah Counties, Oregon, and Clark County, Washington, Oregon Department of Geology and Mineral Industries IMS-40.

[28] Oregon Department of Geology and Mineral Industries, 2020. Oregon Lidar Data Viewer, *accessed September 2020*, from DOGAMI web site: http://www.oregongeology.org/sub/LiDARdataviewer/index.htm.

DOGAMI developed a statewide landslide susceptibility map[29] using the lidar data, USGS topography, SLIDO historical landslide information, and the state geologic map. The landslide susceptibility hazard mapping available on HAZVU indicates a "low" (landsliding unlikely) potential for landsliding for the area of the proposed building, and a "moderate" (landsliding possible) potential for landsliding for the remainder of the site. The existing steep cut slope along the northeast side of NW Cornelius Pass Road (southwest of the site) is mapped as having a "high" (landsliding likely) potential for landsliding.

### C.5.2 Seismic Hazards

C.5.2.1 Liquefaction

A wide variety of slope and ground failures can occur in response to intense seismic shaking during large magnitude earthquakes. These failures are often related to the phenomenon of liquefaction, the process by which water-saturated sediment changes from a solid to a liquid state. Since liquefied sediment may not support the overlying ground, or any structure built thereon, a variety of failures may occur, including lateral spreading, landslides, ground settlement and cracking, sand boils, oscillation lurching, etc. The conditions necessary for liquefaction to occur are: (1) the presence of poorly consolidated, generally cohesionless sediment; (2) saturation of the sediment by groundwater; and (3) an earthquake that produces intense seismic shaking (generally a moment magnitude greater than M5.0). In general, older, more consolidated sediment, and sediment above the water table will not liquefy[30]. Field performance data and laboratory tests indicate that liquefaction occurs predominantly in well-sorted, loose to medium dense sand or silty sand, but can also occur in lean clays and silts[31].

The Oregon Department of Geology and Mineral Industries' Oregon Statewide Geohazards Viewer[32] shows a high hazard for liquefaction for the site and immediate vicinity[33].

The depth to groundwater map for the Portland area[34] indicates groundwater is present at depths of 273 feet bgs in the vicinity of the site. Based on the depth to groundwater, site soils are considered non-liquefiable.

C.5.2.2 Expected Ground Shaking

The HAZVU[35] website includes a layer indicating the expected earthquake shaking felt at a site for a magnitude 9.0 Cascadia Subduction Zone earthquake (as discussed in Section C.3.1.2.1). The mapping is

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29 Burns, William J, Mickelson, Katherine A., and Madin, Ian P, 2016. Landslide susceptibility overview map of Oregon. Oregon Department of Geology and Mineral Industries, Open-File Report O-16-02. Available on Oregon Statewide Geohazards Viewer, *accessed September 2020*, from DOGAMI web site: http://www.oregongeology.org/sub/hazvu/index.htm.

30 Youd, T.L. and Hoose, S.N. 1978. Historic ground failures in Northern California triggered by earthquakes: U.S. Geological Survey Professional Paper 993, p.117.

31 Seed, R.B., et al. 2003. Recent Advances In Soil Liquefaction Engineering: A Unified And Consistent Framework. Earthquake Engineering Research Center College Of Engineering University Of California, Berkeley.

32 Oregon Department of Geology and Mineral Industries, 2020. Oregon Statewide Geohazards Viewer, *accessed September 2020*, from DOGAMI web site: http://www.oregongeology.org/sub/hazvu/index.htm.

33 The liquefaction hazard mapping shown on the DOGAMI Geohazard Viewer is based on material types from the Oregon Geologic Data Compilation (OGDC) map and does not take into account groundwater conditions or the relative consistency or density of the materials present. DOGAMI assigned relative hazard (low, moderate, high) levels based on the potential of a particular soil type to experience liquefaction-induced settlement from a design-level earthquake. Specific limitations of the mapping are presented at https://www.oregongeology.org/hazvu/hazards-assets.htm.

34 Snyder, D.T., 2008, Estimated depth to ground water and configuration of the water table in the Portland, Oregon area: U.S. Geological Survey, Scientific Investigations Report SIR-2008-5059, scale 1:60,000.

based on six categories of ground shaking ranging from “light” (category 1) to “violent” (category 6). The map indicates a “very strong” (category 4) level of ground shaking anticipated at the site during a design level earthquake.

C.5.2.3 <u>Surface Rupture</u>

*C.5.2.3.1* *<u>Faulting</u>*

As discussed above, the site is situated in a region of the country characterized by extensive faulting and known for seismic activity. However, no known faults are mapped on or immediately adjacent to the site, the risk of surface rupture impacting the proposed development at the site due to faulting is considered very low.

*C.5.2.3.2* *<u>Lateral Spread</u>*

Surface rupture due to lateral spread can occur on sites underlain by liquefiable soils that are located on or immediately adjacent to slopes steeper than about 3 degrees (20H:1V), and/or adjacent to a free face, such as a stream bank or the shore of an open body of water. During lateral spread, the materials overlying the liquefied soils are subject to lateral movement downslope or toward the free face. Recognizing the lack of liquefiable soils, we characterize the risk of lateral spread to be negligible.

## C.6.0 SITE RECONNAISSANCE

CGT Senior Engineering Geologist Ryan Houser, RG, CEG, performed a reconnaissance of the site on September 17, 2020.

### C.6.1 Surface Conditions

The proposed site layout is shown on the Site Plan, attached to the geotechnical report as Figure 2. The site was bordered to the north by NW Skyline Boulevard, to the west and south by NW Cornelius Pass Road, and to the east by an agricultural field.

Topography observed at the site was consistent with that shown on Figure C8 and the topographic profiles presented on Figure C9.

An 8-foot-tall cut slope ascended to the south above NW Skyline Boulevard at a gradient of about 2H:1V. This slope was vegetated with pine trees. No signs of instability were noted on this cut slope.

South of the cut slope, the site generally descended gently to the west at gradients up to about 9H:1V. The majority of the site was vegetated with grasses (Figure C10, Photograph 1). No signs of slope instability or erosion were noted on the site.

South of the southern property boundary, the area descended to the south at gradients averaging approximately 5H:1V, with a localized area near an offsite drainage exhibiting a slope gradient up to about 2H:1V. This area was vegetated with coniferous trees, grasses, blackberry bushes and ivy (Figure C10, Photograph 2). No signs of instability were noted on this slope.

---

[35] Oregon Department of Geology and Mineral Industries, 2020. Oregon Statewide Geohazards Viewer, *accessed September 2020*, from DOGAMI web site: http://www.oregongeology.org/sub/hazvu/index.htm.

West of the western property boundary, the topography descended to the west at a generally even gradient of 2H:1V. This slope (Figure C10, Photograph 3) was up to about 25 feet in height, and consisted of a cut slope associated with grading for NW Cornelius Pass Road. This slope was vegetated with grasses and scattered coniferous trees. Basalt bedrock was exposed at the base of the cut slope, as indicated on Figure C8. No signs of instability were noted on this cut slope.

### C.6.2 Review of Site Subsurface Conditions

As indicated in Section 2.3 of the main report, subsurface conditions encountered in our explorations consisted of native lean clay. This material is consistent with the windblown sediments described in Section C.2.0 above. The weathered basalt observed at the base of the cut slope along NW Cornelius Pass Road is consistent with the basalt bedrock described in Section C.2.0 above.

## C.7.0 FINDINGS & RECOMMENDATIONS

The primary geologic hazards that may affect the site are potential slope instability and seismic shaking. We anticipate that with proper construction control, the geology and topography of the site and the surrounding area will not adversely affect the proposed project, and the project will have no geologic impact on adjacent properties or the risk of slope instability. It is our opinion that, with the use of generally accepted construction techniques and by strictly following the recommendations contained in the geotechnical report and in the building code, the site is geologically suitable for the proposed development.

### C.7.1 Slope Instability

As described above, the site is located on a gentle west-facing slope. No signs of recent instability were observed during the reconnaissance. Steep slopes in the vicinity of the site consisted of constructed offsite cut slopes associated with offsite road construction. Cut slopes observed as part of this study did not show signs of instability or erosion.

The proposed storage shed will be located on a relatively level portion of the site in an area away from steep slopes or areas that are considered potentially unstable. We understand proposed grading is minimal.

In order to minimize impact to the stability of the site and surrounding properties, the recommendations contained in the geotechnical report should be incorporated into the project plans and specifications. Provided the geotechnical recommendations are incorporated into the project plans, the proposed development will have little to no impact on the stability of the site or surrounding properties.

Any construction within hillside areas inherently bears greater risk of slope instability. The on-site and off-site slopes may be susceptible to slope instability resulting from factors beyond the owner’s control, such as off-site grading, erosion and other ground disturbance, a major earthquake, or heavy precipitation. The owners must recognize and accept the risk of potential slope instability from causes beyond their control or as yet unrecognized.

### C.7.2 Seismic Shaking

To minimize the risk that this hazard will adversely impact the proposed development, the structure should be designed and constructed in accordance recommendations provided in the geotechnical report. The proposed development will have no impact on this hazard.

### C.7.3 Other Hazards

Other geologic hazards identified in the State of Oregon Engineering Geology Report guidelines include:

- Shallow Groundwater
- Subsidence
- Coastal Erosion
- Coastal Flooding
- Tsunami / Seiche
- Expansive Soils
- Volcanic Hazards

Based on our research, field reconnaissance, and previous experience in the area, none of these hazards are present at the site.

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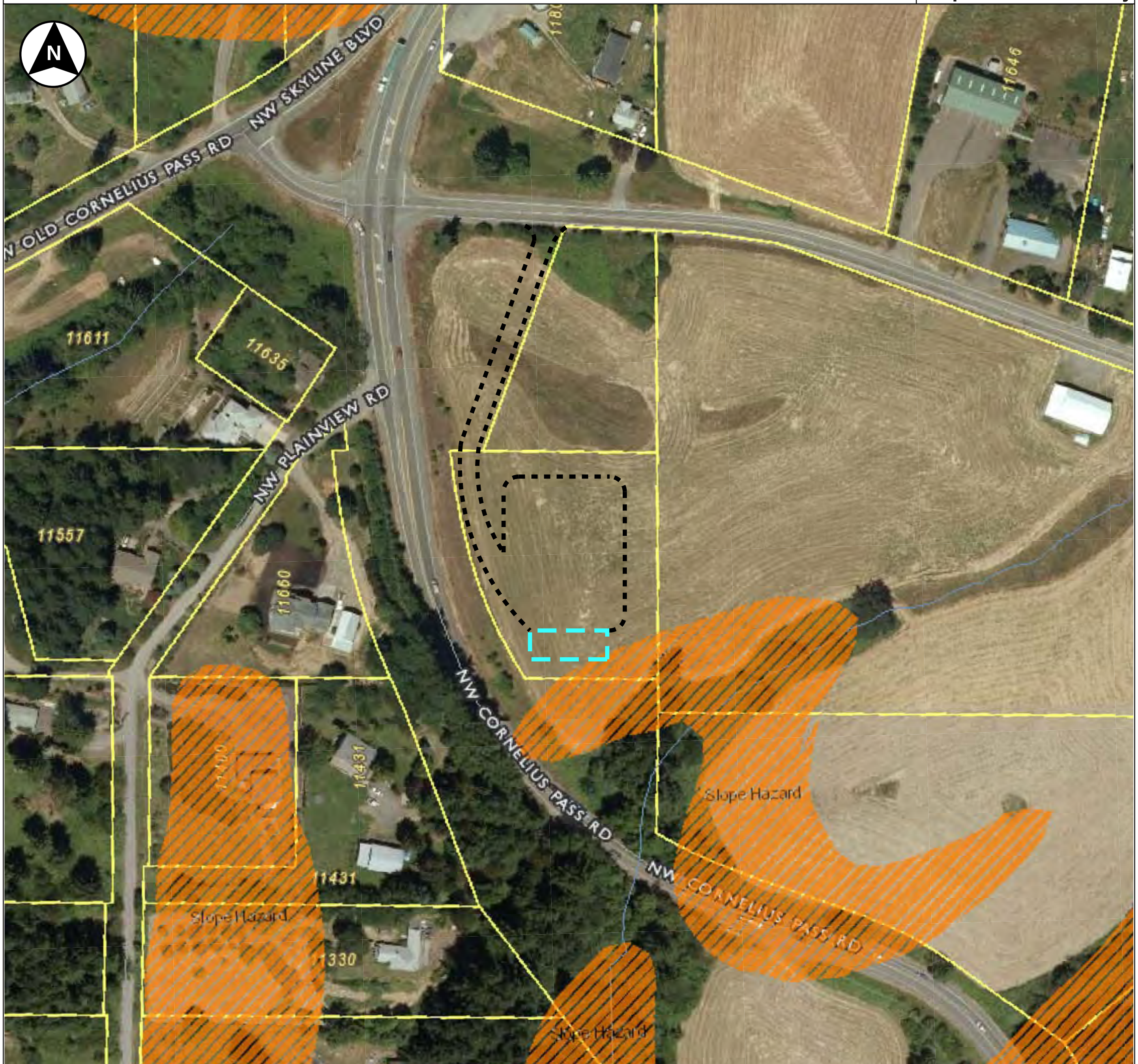
*Project Number G2005344*

**FIGURE C1**
**Slope Hazard Overlay**




LEGEND

Slope Hazard Overlay

Proposed Storage Building

Proposed Access Road and Materials Storage Area

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NOTES: 2010 aerial photograph and slope hazard overlay obtained from Multnomah County Land Use Planning Interactive Zoning Map online application accessed September 2020, from Multnomah County website: http://www4.multco.us/lup/. All locations should be considered approximate.

1 Inch = 200 Feet

0 200 400

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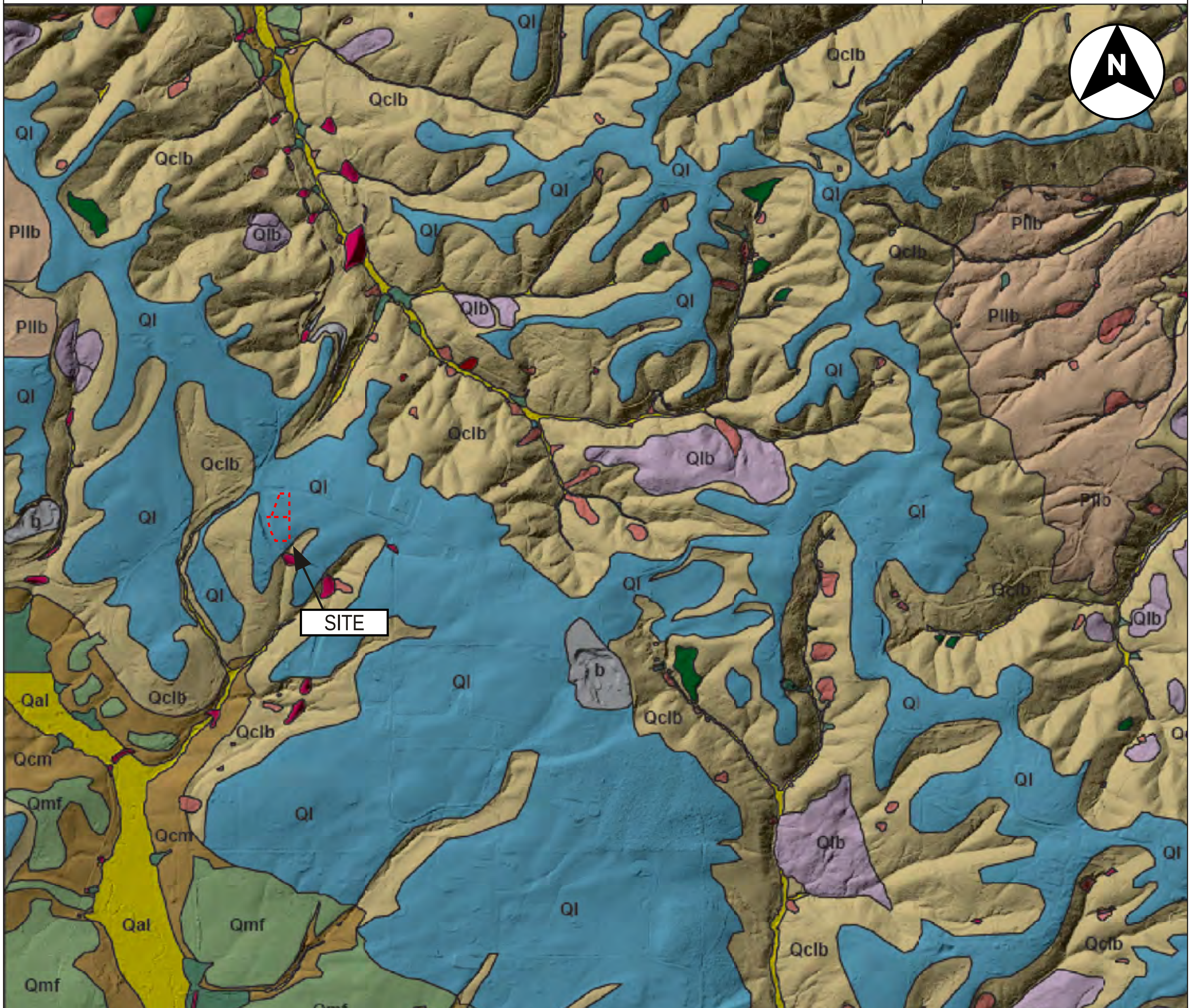
**FIGURE C2**

**Surficial Geologic Map**




**DEPOSITS FORMED BY MOVING WATER AND WIND**

- Qal — Alluvium (Holocene)
- Qt — Terrace deposits (late Pleistocene-Holocene)
- Qmf — Missoula (Bretz) flood deposits (late Pleistocene)
- Ql — Primary loess (Pleistocene)

**DEPOSITS FORMED BY MASS TRANSPORT**

**Colluvium Deposits**

- Qclb — Loess-basalt fragment colluvium (Quaternary)
- Qcm — Missoula flood silt colluvium (Quaternary)
- Qcg — Loess and conglomerate colluvium (Quaternary)

**Landslide Deposits**

- Qf — Flow and fan deposits (latest Pleistocene-Holocene)
- Qls1 — Surficial landslides (Pleistocene-Holocene)
- Qls2 — Surficial landslides (Pleistocene-Holocene)
- Hls — Surficial landslides (Holocene)
- Qlb — Bedrock landslides (Pleistocene-Holocene)
- Pllb — Bedrock landslides (Pleistocene-Holocene)

**OTHER MATERIALS**

- af — Artificial fill (Anthropocene)
- b — Bedrock exposures

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Map adapted from Madin, Ma, and Niewendorp, 2008. Preliminary Geologic Map of the Linnton 7.5' Quadrangle, Multnomah and Washington Counties, Oregon. Oregon Department of Geology and Mineral Industries Open File Report O-08-06.

1 Inch = 2,000 Feet

0 2,000 4,000

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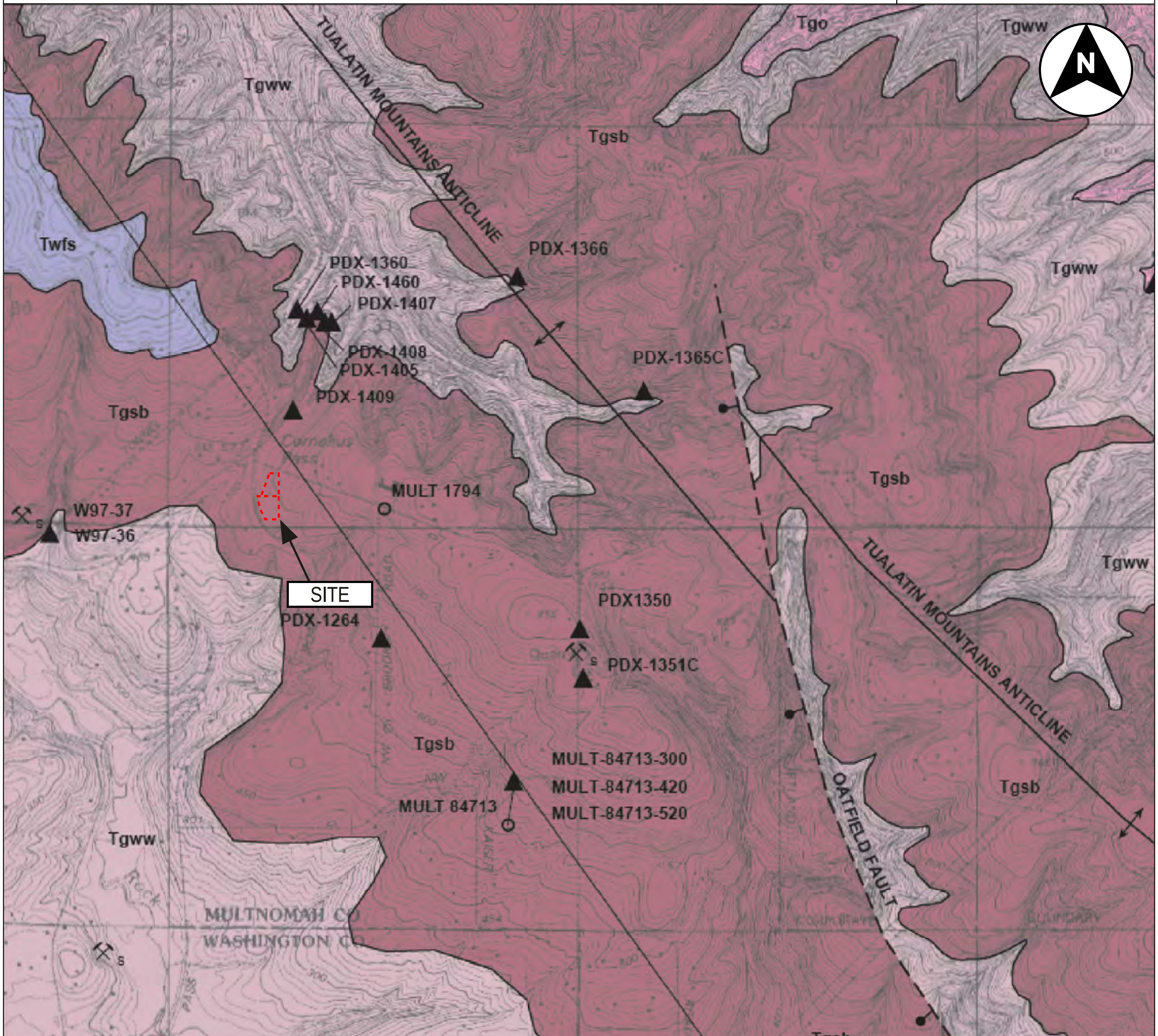
Project Number G2005344

FIGURE C3
Bedrock Geologic Map




**Boring Volcanic Field Rocks (Pliocene and Pleistocene)**

| | |
|---|---|
| Qbab | Basaltic andesite of Barnes Road (Pleistocene) |
| Qbae | Basaltic andesite of Elk Point (Pleistocene) |
| Qbk | Basalt of Kaiser Road (Pleistocene) |
| QTbb | Basaltic andesite of Bonny Slope (Pliocene to Pleistocene) |

**Hillsboro Formation (Miocene to Pleistocene)**

| | |
|---|---|
| Th | |

**Troutdale Formation (Miocene and Pliocene)**

| | |
|---|---|
| Ttg | Conglomerate and sandstone unit (Miocene? and Pliocene) |

**Columbia River Basalt Group (middle and lower Miocene)**

**Wanapum Basalt (middle Miocene)**

**Frenchman Springs Member (middle Miocene)**

| | |
|---|---|
| Twfs | Basalt of Sand Hollow (middle Miocene) |

**Grande Ronde Basalt (middle and lower Miocene)**

| | |
|---|---|
| Tgsb | Member of Sentinel Bluffs (middle Miocene) |

**Member of Winter Water (middle Miocene)**

| | |
|---|---|
| Tgww | Basalt of Winter Water (middle Miocene) |

**Member of Ortley (middle Miocene)**

| | |
|---|---|
| Tgo | Basalt of Ortley (middle Miocene) |

**Scappoose Formation (lower Miocene and upper Oligocene)**

| | |
|---|---|
| Ts | Marine sandstone unit (Miocene?) |

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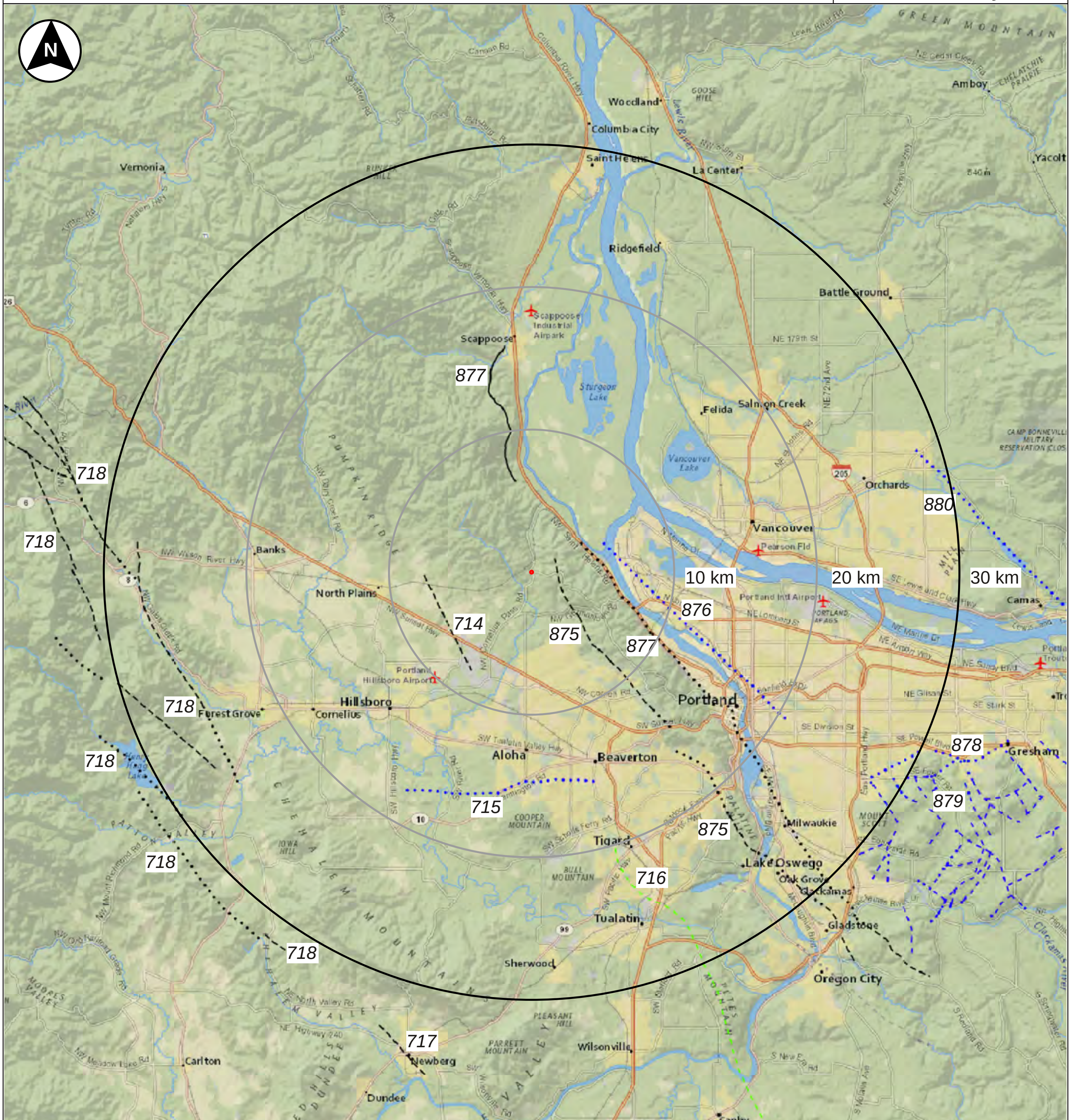
Map adapted from Madin, Ma, and Niewendorp, 2008. Preliminary Geologic Map of the Linnton 7.5' Quadrangle, Multnomah and Washington Counties, Oregon. Oregon Department of Geology and Mineral Industries Open File Report O-08-06.

1 Inch = 2,000 Feet

0 2,000 4,000

ODOT SKYLINE STORAGE SHED - MULTNOMAH COUNTY, OREGON
Project Number G2005344

FIGURE C4
USGS Quaternary Faults




LEGEND

| | |
|---|---|
| Historic (< 150 years) | Well constrained location (solid line) |
| Latest Quaternary (< 15,000 years) | Moderately constrained location (dashed line) |
| Late Quaternary (< 130,000 years) | Inferred location (dotted line) |
| Middle and late Quaternary (< 750,000 years) | |
| Undifferentiated Quaternary (< 1.6 million years) | 716 USGS Fault Number |
| Unspecified Age | |
| Class B (age varies) | |

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NOTES: Data from USGS Quaternary Fault and Fold Database, *accessed July 2020*, at website: https://earthquake.usgs.gov/cfusion/qfault/.

1 Inch = 10 Kilometers

0 10 20

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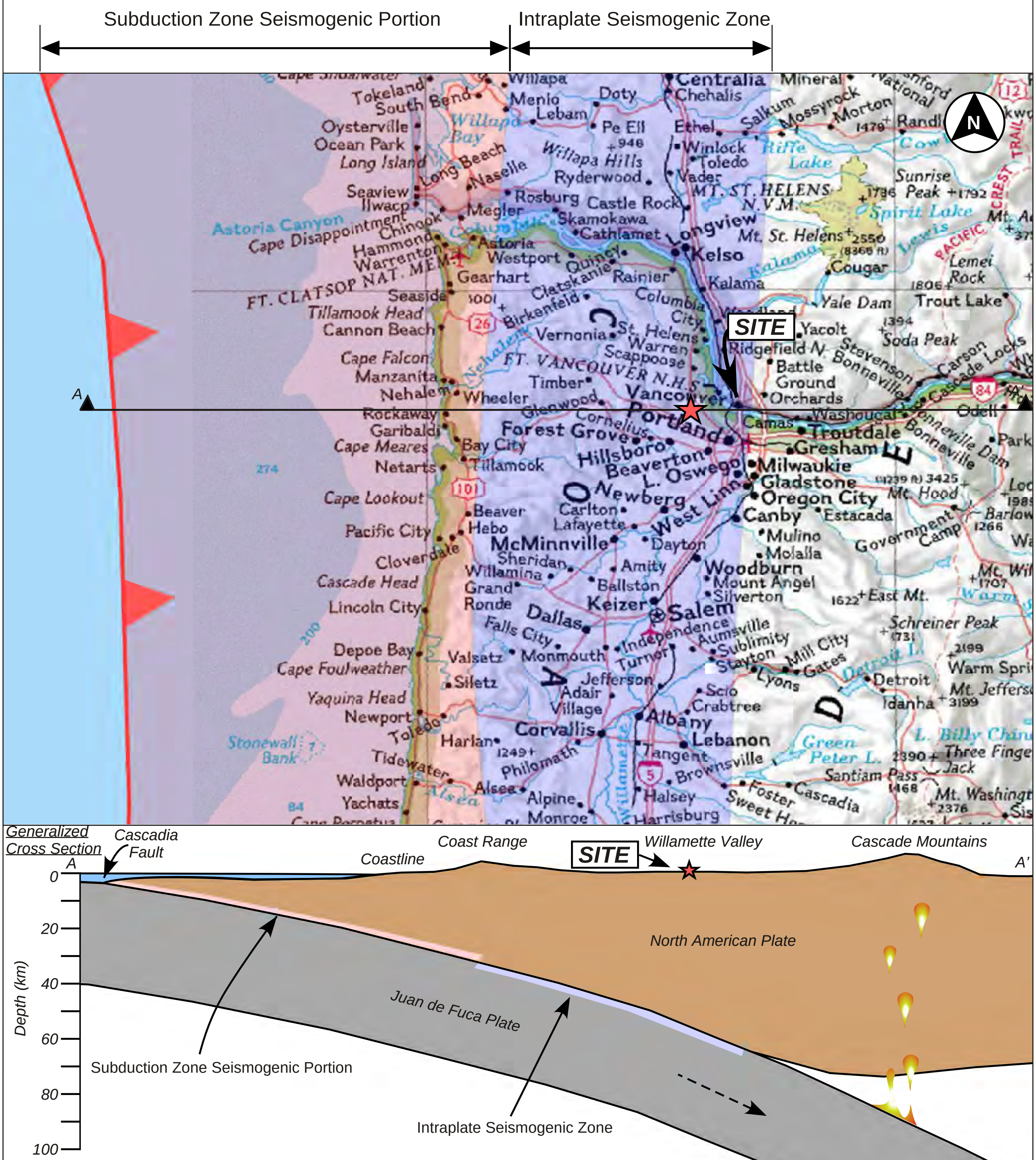
**Project Number G2005344**

**FIGURE C5**

**Cascadia Subduction Zone**

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GEOTECHNICAL
503-601-8250

McCrory, Blair, Oppenheimer, and Walter, 2004. Depth to the Juan de Fuca slab beneath the Cascadia subduction margin - A 3-D model for storing earthquakes: U.S. Geological Survey Data Series 91.

Scale 1 Inch = 50 kilometers

0 50 100

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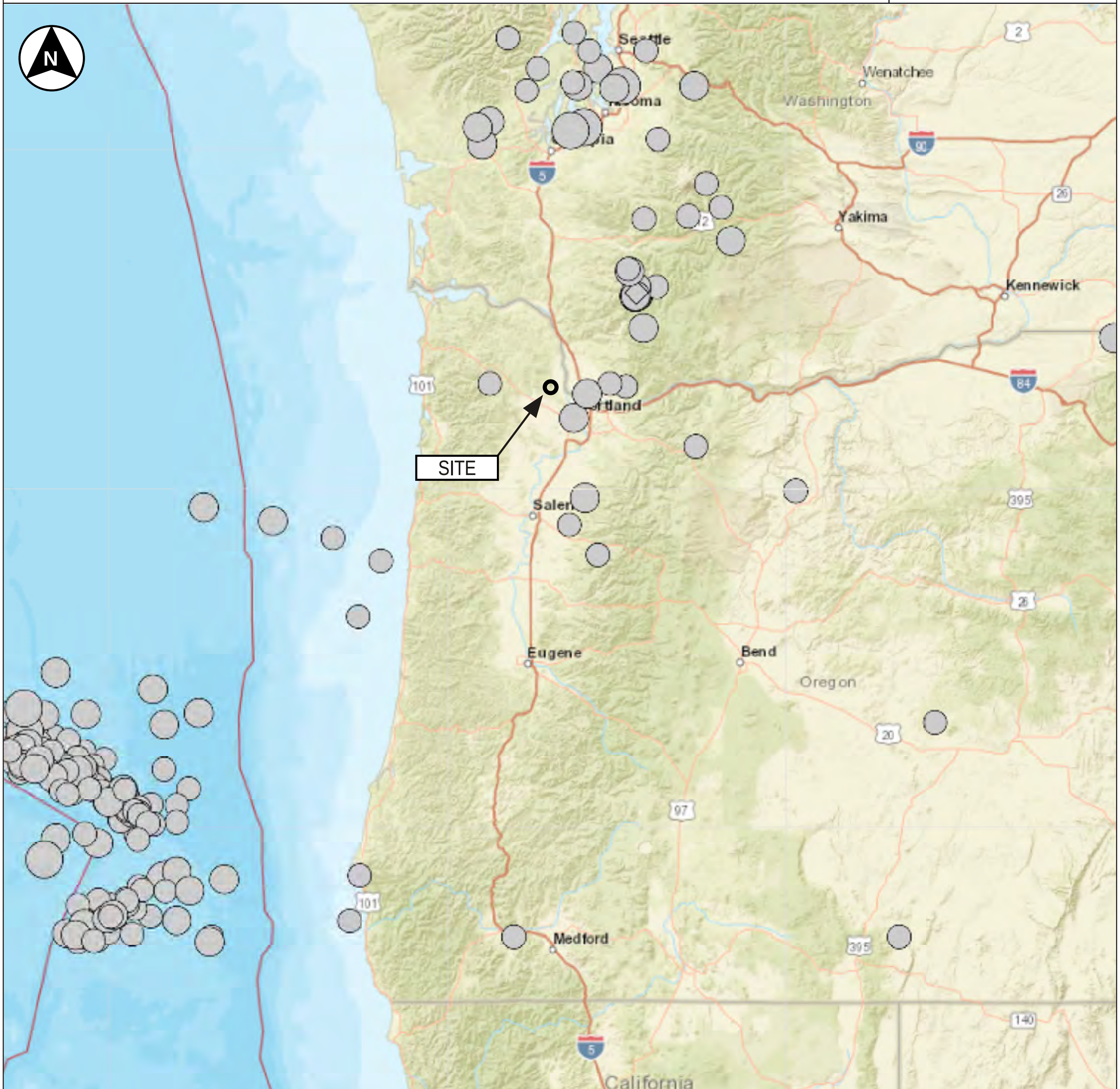
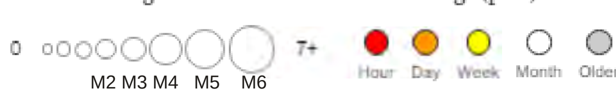
Project Number G2005344

FIGURE C6
Historical Earthquakes




## 1924 - 2020 Earthquakes with Magnitude above M4.5

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Map created from USGS Earthquake Catalog at https://earthquake.usgs.gov/earthquakes/, *accessed September 2020.*

Latitude: 45.870877° North
Longitude: 122.688732° West

1 Inch = 100 kilometers

0 100 200

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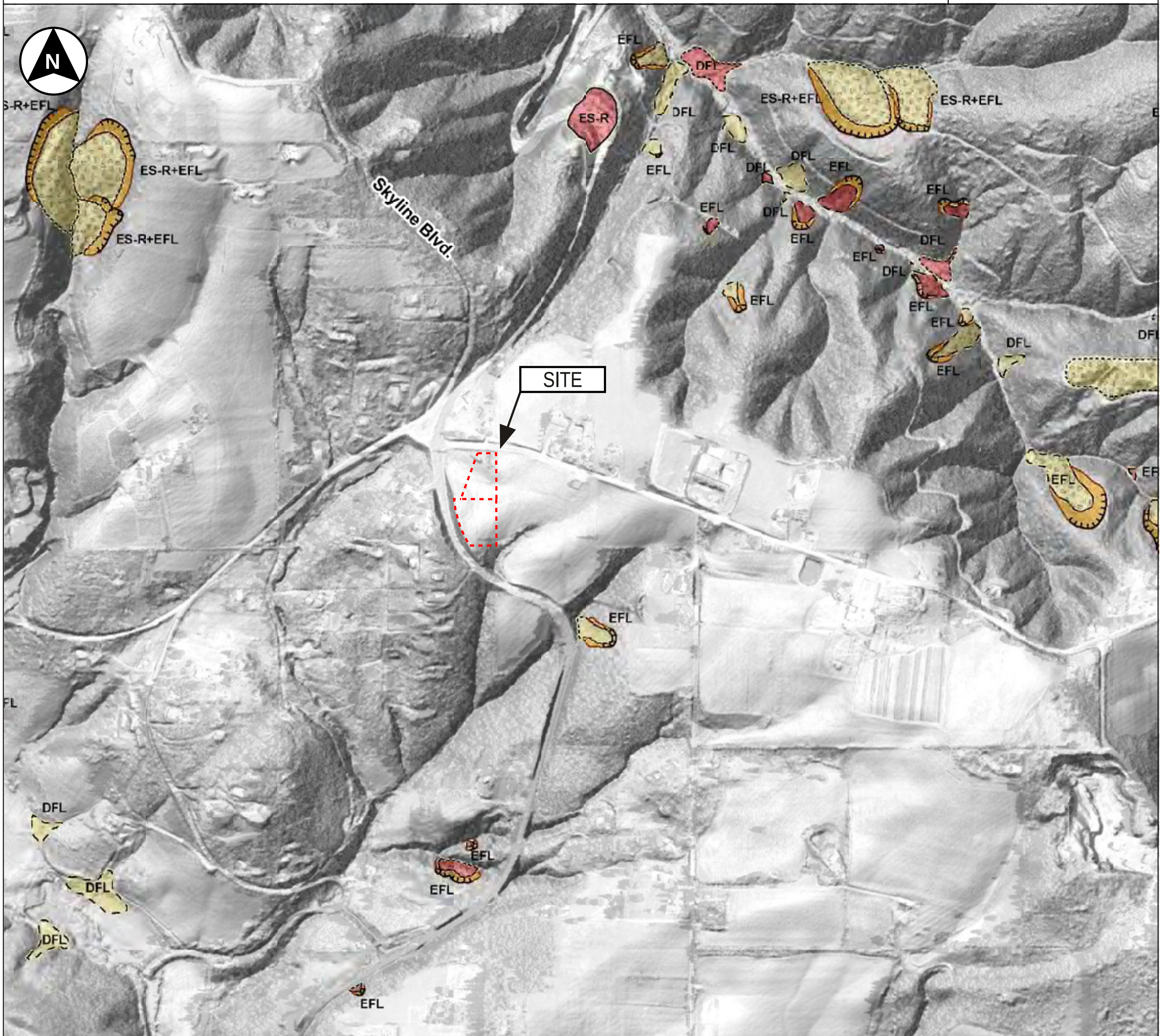
FIGURE C7

Landslide Inventory




Historic and/or Active Landslide (movement less than 150 years ago)
Prehistoric or Ancient Landslide (movement greater than 150 years ago)
Head Scarp Zone and Flank Zones
Head Scarp
Shallow-Seated Landslide
Deep-Seated Landslide

CARLSON CGT GEOTECHNICAL 503-601-8250

Map adapted from Burns and Duplantis, 2011, Landslide Inventory Map of the Linnton Quadrangle, Multnomah and Washington Counties, Oregon. Oregon Department of Geology and Mineral Industries IMS-35.

1 Inch = 1000 feet
0 1000 2000

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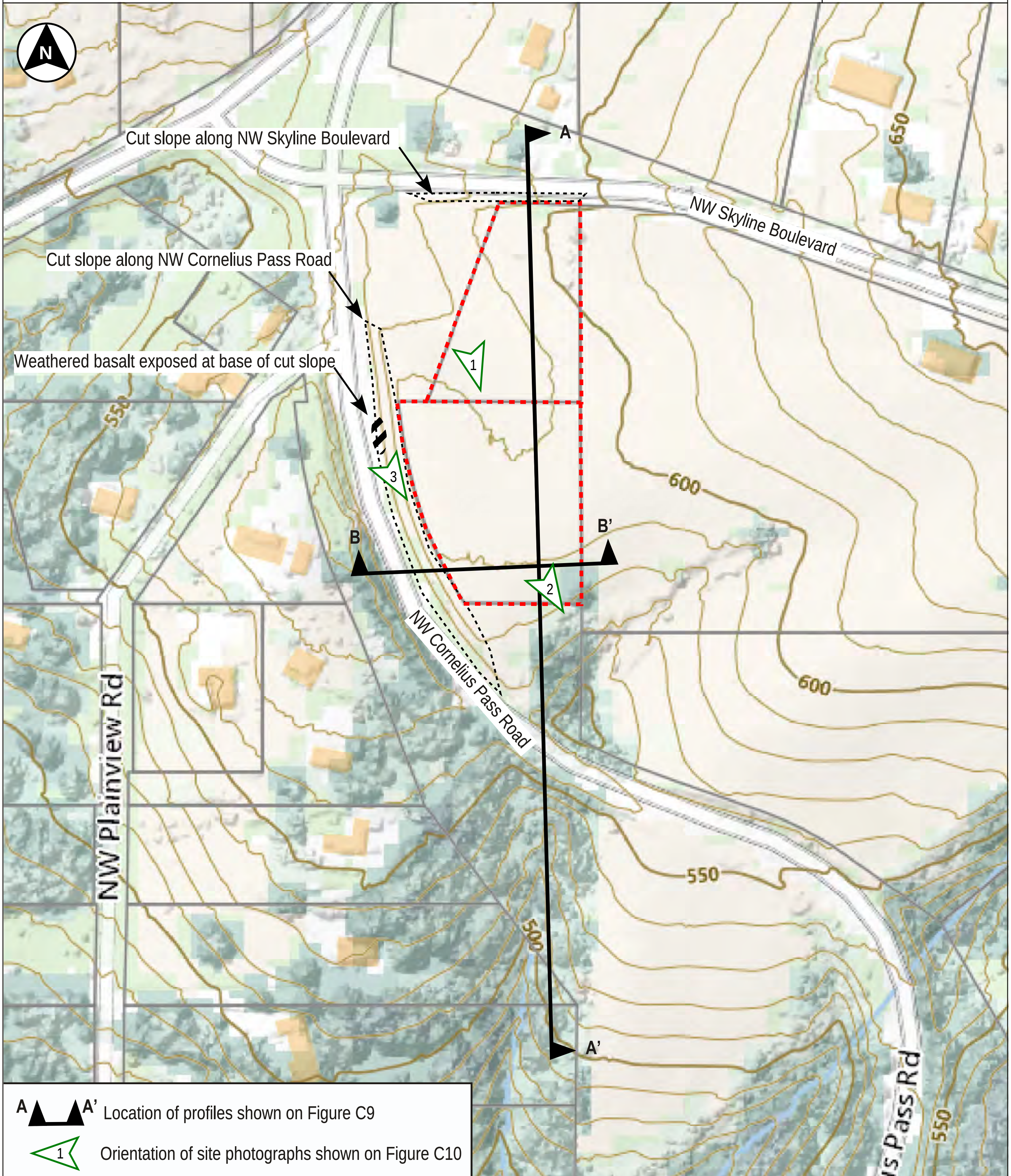
*Project Number G2005344*

**FIGURE C8**

**Vicinity Topography**

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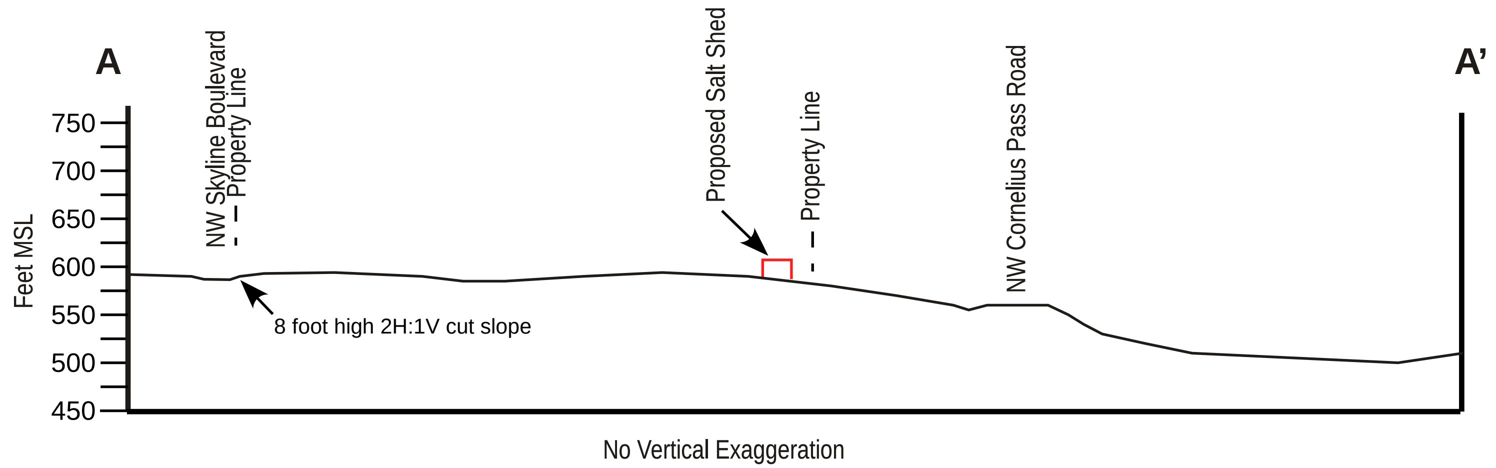
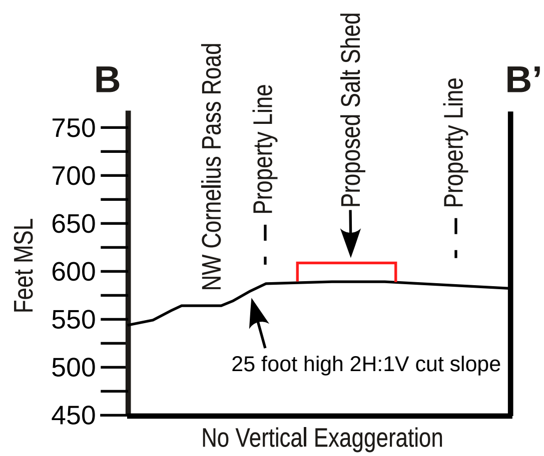
NOTES: 2009 topographic contours, and property lines from MetroMap Regional Land Information System (RLIS) data, accessed September 2020, from Metro website: http://gis.oregonmetro.gov/metromap/.

1 Inch = 200 feet

0 200 400




1 Inch = 200 Feet

0 200 400

Note: Existing topography from Figure C8

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Tigard, Oregon 97281

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**FIGURE C9**

**Topographic Profiles**

Photograph 1

Photograph 2

Photograph 3



*See Figure C8 for approximate photograph locations and directions. Photographs were taken at the time of the site reconnaissance.*

**Land Use Planning Division**
1600 SE 190th Ave, Ste 116
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multco.us/landuse

# GEOLOGIC HAZARDS PERMIT (GHP) Form 1:
## *GEOTECHNICAL RECONNAISSANCE AND STABILITY PRELIMINARY STUDY*

*Note: Response to each question below must be completed or verified by a Certified Engineering Geologist or Geotechnical Engineer, including a State of Oregon Registration Stamp and Number in the space provided on page four. The GHP Form 1 addresses Multnomah County Code Section 39.5085(C)(3)(c); 38.5515(C)(3)(c), Geologic Hazards Permits.*

**Site Address:** Tax Lot 2500, NW Cornelius Pass Road, Multnomah County, Oregon

**Legal Description:** 2N1W31C-02500

**Property Owner's Name:** Oregon Department of Transportation

**Firm Preparing Report:** Carlson Geotechnical

**Address:** 18270 SW Boones Ferry Road, Suite 6

**City:** Durham **State:** Oregon **Zip:** 97224

**Preparer's Name:** Ryan T. Houser, CEG

**Phone Number:** 503-601-8250 x1107

## GENERAL PROPERTY INFORMATION

1. a. Maximum Slope on Property: 2H:1V Area in which it is located: Road cut at north end of site
   Average Slope of Property: 8H:1V

   b. Are there any wetlands or streambeds on the property? (Please Circle) Yes No
   If yes, please show on topographical survey or sketch.

   c. Volume of soil or earth material disturbed, stored, disposed of or used as fill: none at present

   d. Total area of proposed ground disturbance:
   ____________ (square feet) ____________ (acres)

New roadway and open pile materials storage areas will encompass a footprint of approximately 1 acre.
New storage shed has a plan footprint of approximately 3,060 square feet.

Were building plans considered when completing this form? (Please Circle) Yes No

If yes, please note the author and date the plans were prepared.

ODOT Conceptual plans dated 04/13/20

2. What is the general topography of the property? Please attach a topographic survey or sketch with pertinent notes.

Site topography described in Section C.6.1 of engineering geology report (Appendix C), and shown on Figures 2 and C8.

3. Are there any visible signs of instability or other potentially adverse site features (Landslides, slumps, mud flow, creep, ravines, fills, cuts, seeps, springs, ponds, etc.) within the surrounding area for a minimum distance of 100 feet beyond the subject property boundaries? Describe and indicate on attached topographic survey or sketch.

No signs of instability or adverse features were observed within 100 feet of the subject property. See Sections C.6 and C.7 of engineering geology report (Appendix C) for discussion.

4. Is any earthwork proposed in connection with site development?

(Please Circle) Yes No

If yes, please indicate depth and extent of cuts/fills; describe fill types.

Cuts and fills to achieve finished grades are anticipated to be less than 3 feet, as described in Section 1.1 of the geotechnical report.

5. In your opinion, will the proposed earthwork cause potential stability problems for the subject and/or adjacent properties?

(Please Circle) Yes No

IF YES, EXPRESS PROBABILITY:

(Please Circle) Very Probable Possibly Possible, but remote

If Very Probable or Possibly, please explain.

6. In your opinion, will the proposed development (structures, foundations, parking area, streets, etc.) create potential stability problems for the subject and/or adjacent properties?

(Please Circle) Yes No

IF YES, EXPRESS PROBABILITY:

(Please Circle) Very Probable Possibly Possible, but remote

If Very Probable or Possibly, please explain.

7. In your opinion would the subsurface disposal of sewage effluent on the site (i.e., drain fields) have an adverse affect on stability of the site or adjacent area?

(Please Circle) Yes No Not applicable

IF YES, EXPRESS PROBABILITY:

(Please Circle) Very Probable Possibly Possible, but remote

If Very Probable or Possibly, please explain.

8. If answer is Very Probable or Possibly to questions 4 or 5, is it your opinion, on the basis of a visual evaluation, that adequate stability might be achieved by preferred siting of the development, alternative foundation support, earthwork, drainage, etc.?

(Please Circle) Yes No Not applicable

If yes, please explain.

9. Do you recommend additional geotechnical studies (i.e., mapping, testing pits or borings, stability analysis, etc.) prior to site development?

(Please Circle) Yes No

If yes, please explain.

A geotechnical investigation report was completed by Carlson Geotechnical for the proposed project dated September 23, 2020. The geotechnical recommendations contained therein should be incorporated into the design and development of the proposed project.

*By signing and affixing the required stamp below, the Certifying Engineering Geologist or Geotechnical Engineer certifies that the site is suitable for the proposed development.*

Signature ______________________

Date 9/28/20

Affix Seal Here

CERTIFIED
OREGON
RYAN T. HOUSER
E1914
ENGINEERING GEOLOGIST

EXPIRES 12/1/20