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# PRELIMINARY STORMWATER REPORT

**To**
City of Stayton, Oregon

**For**
Stayton Washington, LLC
4020 Kinross Lakes Parkway,
Richfield, OH 44286

**Dated**
March 6, 2024

**Project Number**
2220389.05


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1. Appendix A Stormwater Facility Calculations
2. Appendix B Geotechnical Report

## DESIGNER'S CERTIFICATION AND STATEMENT

I hereby certify that this Stormwater Report for Santiam Industrial Center has been prepared by me or under my supervision and meets minimum standards of the City of Stayton 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. PROJECT OVERVIEW AND DESCRIPTION

The site is in the southwest portion of Stayton on a portion of Marion County tax lot number 091W10CB02400 (Parcel 2 of the partition application approved by Land Use File #4-05/23). The project consists of several loading docks, parking areas and a bioretention pond being built for water quality and detention purposes.

### Vicinity Map

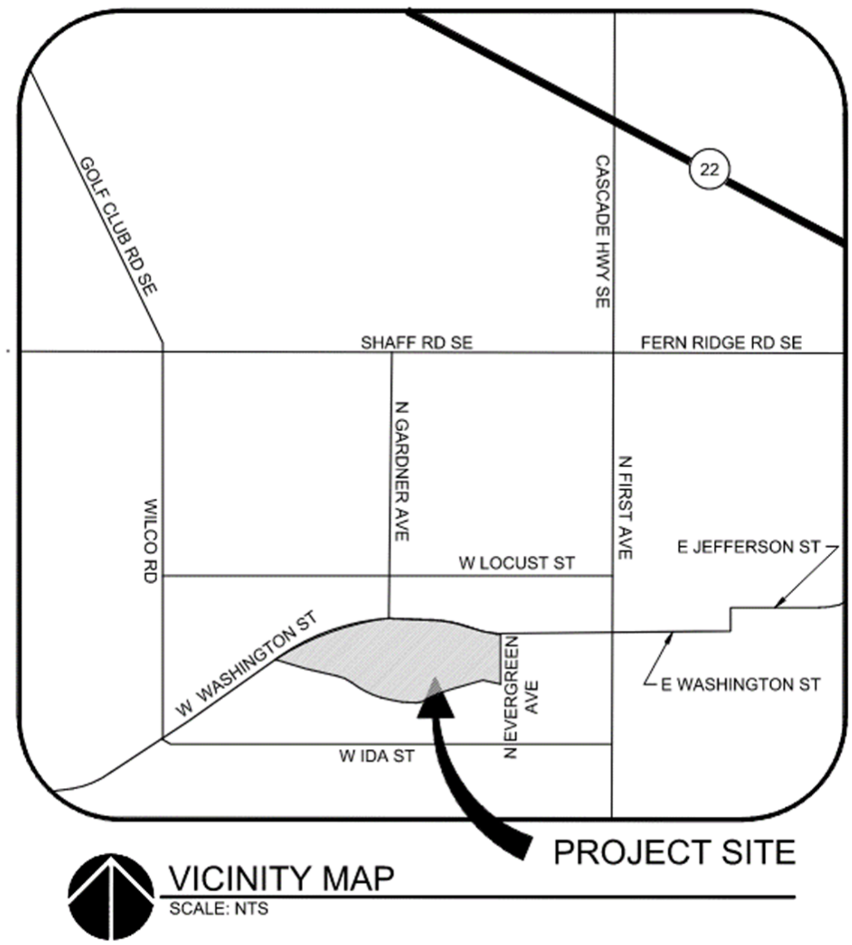



**Figure 1 Vicinity Map**

### Existing Conditions

The existing site currently has three buildings on site (two warehouses and a machine shop, totaling 12.06 acres) that are currently used for industrial and office purposes. Our understanding is that there was also recent work done to remove and remodel some of the existing buildings. There are landscape areas on the south, west and east ends of the site. There is a fence that runs along the north side of the site adjacent to W Washington Street. The Salem Ditch runs along the south side of the site.

The existing land has slopes generally ranging from about 1.5% to 10.0%. The slopes drain to several catch basins located across the south and west areas of the site where it is collected and brought to the Salem Ditch running alongside the south end of the site. The Ditch is about 2' to 4' deep and brings the stormwater west to Mill Creek.

There does not appear to be any stormwater treatment or flow control devices on site. Once the stormwater goes to the Ditch, the stormwater on-site follows the native drainage path and discharges to Mill Creek to the northwest of the site.

The site is bordered to the northwest by commercial properties. Otherwise, it is surrounded by residential areas.


**Figure 2 Existing Conditions Map**

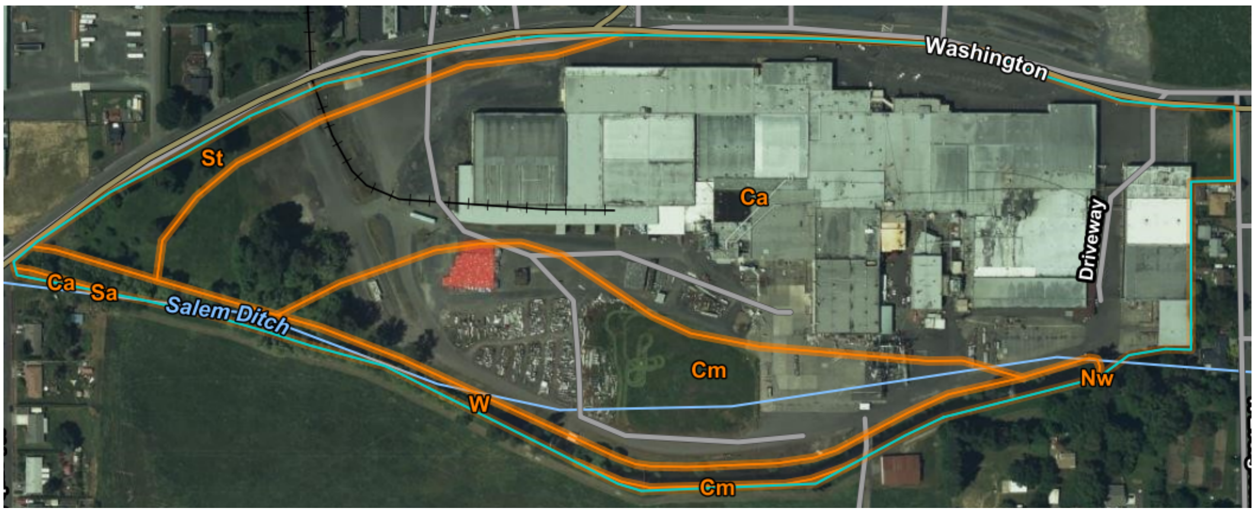
## Soil Conditions

Per the USDA Web Soil Survey, the existing soil consists of several different soils. These soils include Camas gravelly sandy loam (Ca), Cloquato silt loam (Cm), Newberg silt loam (Nw), Salem gravelly silt loam (Sa), Sifton gravelly loam (St), and water (W). See Figure 3 for the location. The hydrologic soil groups have been identified as follows. Ca is group A, Cm is group B, Nw is group A, Sa is group B and St is group B. The first letter is for drained areas and second is for undrained areas.

Group A soils have low runoff potential and high infiltration rates even when thoroughly wetted. They consist chiefly of deep, well to excessively drained sand or gravel and have a high rate of water transmission.

Group B soils have moderate infiltration rates when thoroughly wetted and consist chiefly of moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse textures. These soils have a moderate rate of water transmission.

From the Geotechnical Report, groundwater was observed between depth of 5’ and 6.5’ below ground surface.




| Map Unit Symbol | Map Unit Name | Acres in AOI | Percent of AOI |
|---|---|---|---|
| Ca | Camas gravelly sandy loam | 27.2 | 67.5% |
| Cm | Cloquato silt loam | 8.3 | 20.7% |
| Nw | Newberg silt loam | 0.0 | 0.0% |
| Sa | Salem gravelly silt loam | 0.0 | 0.0% |
| St | Sifton gravelly loam | 2.3 | 5.7% |
| W | Water | 2.5 | 6.1% |
| **Totals for Area of Interest** | | **40.2** | **100.0%** |

**Figure 3 Web Soil Survey Map**

## Proposed Conditions

The proposed improvements will add parking areas and add/repave loading dock areas. In addition, three of the landscape areas will be replaced with gravel and a bioretention planter will be built on the southwest area of the site. Paving for the site will follow geotechnical recommendations with light asphalt paving being used for the car parking areas, concrete paving being used for the dock aprons and heavy asphalt paving being used for the truck areas. Utility lines will be adjusted to accommodate the new dock aprons where needed and landscaping will be planted in the parking islands.

Stormwater runoff will be directed towards the Salem Ditch. Stormwater from five dock aprons along the south side of the building will be rerouted to an existing 36” pipe that outfalls into the ditch. A portion of the pavement area west of the building (including two loading docks) will be collected and routed to the proposed storm facility. The facility will be sized to treat and detain an area equivalent to the disturbed area. A control manhole for the facility will ensure that the appropriate flow is entering the existing pipe and therefore the ditch as well.  Based upon the Geotechnical Report, the subsurface soil and shallow groundwater conditions will not allow infiltration to be effective for stormwater management. As a result, a lined bioretention pond will be used to detain all the stormwater entering the facility before it is routed to the existing storm pipe. Outside of the loading docks the parking areas and landscape area that will be replaced with gravel will be graded to allow the stormwater to sheet flow to an existing catch basin or a dock area.

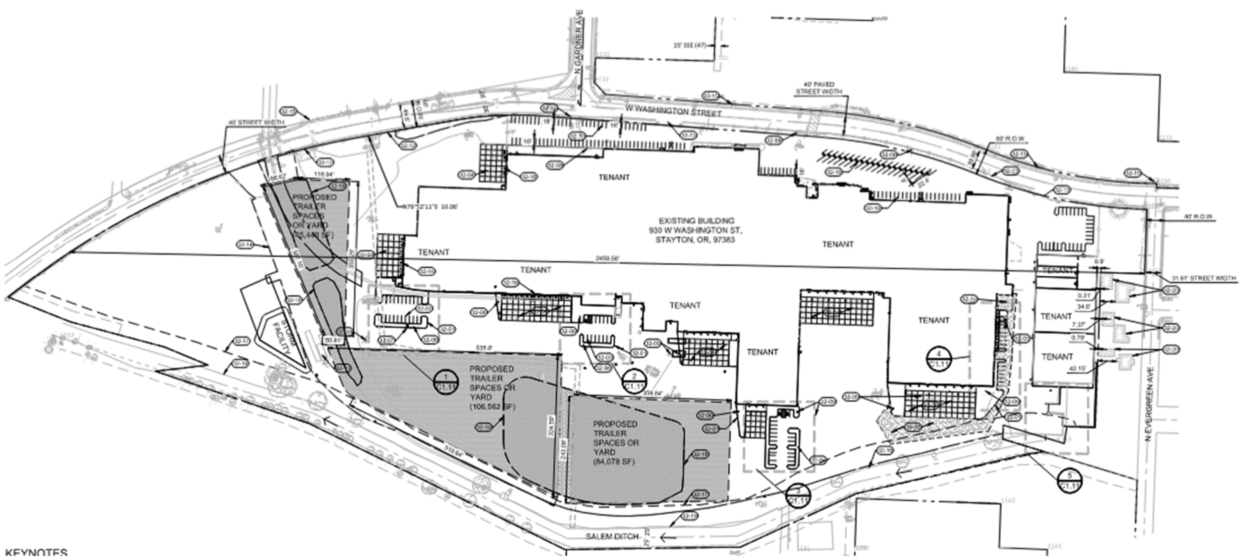



**Figure 4 Site Plan**

The proposed grading mimics the predevelopment grading. Stormwater entering the dock aprons will be collected and routed either to an existing pipe or the proposed storm facility. The stormwater from the storm facility will also connect to an existing pipe. Stormwater from the other paved areas will sheet flow to either an existing catch basin or the dock areas. This will ensure that stormwater on the site will be collected in the existing storm network and brought to the ditch.

## II. BASIS OF DESIGN

In 2010 the City of Stayton adapted the City of Portland's Stormwater Management Plan for the City's stormwater design standards, and all new developments are required to meet the current 2020 Portland Stormwater Management Manual. The Basis of Design for Stormwater Quality and Flow Control, as determined by the City of Portland's Bureau of Environmental Services (BES)'s Stormwater Management Manual (SWMM), is as follows:

- **Discharge Hierarchy** is Level 2B: Offsite flow, either directly or via a storm-only system to a waterbody other than the Willamette River, Columbia River, or Columbia Slough.
- **Detention** is required as the site outfall is not into a waterbody and infiltration is not being used for the storm facility.
- **Water quality** requires an achieved 70% Total Suspended Solids (TSS) removal from the runoff resulting from 90% of the average annual rainfall using the BES online Presumptive Approach Calculator (PAC) and a rectangular basin (flat).
- **Conveyance** will be designed for a **10-year storm** frequency using the Rational Method per BES's Sewer and Drainage Facilities Design Manual.

## III. ANALYSIS

### Methodology

The stormwater management strategy matches the predeveloped flows by routing the stormwater to the existing outfall. It also treats and detains the equivalent disturbed area. A summary of the areas can be found in Table 1 and precipitation rates for the City of Stayton can be found in Table 2.

| Table 1: Disturbed Area Summary | | | | | | |
|---|---|---|---|---|---|---|
| Cover Type | Pre-Development Conditions | | | Post Development Conditions | | |
| | Area ($ft^2$) | Hydrologic Soil Group | CN | Area ($ft^2$) | Hydrologic Soil Group | CN |
| Open Space – Good Condition | 99,592 | A/B | 79 | 33,782 | A/B | 90 |
| Paved Streets, Parking Lots | 156,885 | A/B | 98 | 222,695 | A/B | 98 |

| Table 2: Precipitation Rates | |
|---|---|
| Storm Event | 24-HR Precipitation (inches) |
| Water Quality | 1.61 |
| 2-year | 2.5 |
| 5-year | 3.0 |
| 10-year | 3.5 |
| 25-year | 4.0 |

## Water Quality

The stormwater facility used the performance approach to determine if the pollution reduction requirement for water quality has been met. Under this approach, volume-based facilities must treat a 1.61 inch storm over 24 hrs for a 1 year event. See Appendix A and Table 3 for storm facility details.

| Table 3: Storm Facility Details | |
|---|---|
| **Field** | **Selected Option** |
| Category | Basin (Flat) |
| Shape | Rectangle |
| Location | Parcel |
| Configuration | D: No Infiltration |

## Water Quantity & Flow Control

The stormwater facility used the performance approach to determine if the flow control requirement has been met. See Appendix A and Table 4 for peak flow details.

| Table 4: Peak Flow Events | | |
|---|---|---|
| **Storm event** | **Existing CFS** | **Proposed CFS** |
| 2 yr | 1.736 | 0.849 |
| 5 yr | 2.376 | 0.973 |
| 10 yr | 3.039 | 1.084 |
| 25 yr | 3.721 | 1.281 |

## Conveyance

Detailed conveyance calculations will be designed during the final build using the Rational Method for a 10-year design storm.

## IV. ENGINEERING CONCLUSIONS

Based on compliance with the Stormwater Management Manual:

- **Detention** is required since infiltration is not recommended per the Geotechnical Report.
- **Water quality** treats the 1 year storm event (1.61 inch storm over 24 hrs).
- **Conveyance** was designed for a **10-year storm** frequency using the Rational Method per BES's Standards.

Therefore, the design for Santiam Industrial Center adheres to the City of Portland's Stormwater Management Manual requirements which then meets the City of Stayton stormwater design standards.

APPENDIX A
## STORMWATER FACILITY CALCULATIONS

# Hydrograph Summary Report

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

| Hyd. No. | Hydrograph type (origin) | Peak flow (cfs) | Time interval (min) | Time to Peak (min) | Hyd. volume (cuft) | Inflow hyd(s) | Maximum elevation (ft) | Total strge used (cuft) | Hydrograph Description |
|---|---|---|---|---|---|---|---|---|---|
| 1 | SBUH Runoff | 1.436 | 2 | 474 | 20,263 | ------ | ------ | ------ | WQ FLOW TO POND |

# Hydrograph Summary Report

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

| Hyd. No. | Hydrograph type (origin) | Peak flow (cfs) | Time interval (min) | Time to Peak (min) | Hyd. volume (cuft) | Inflow hyd(s) | Maximum elevation (ft) | Total strge used (cuft) | Hydrograph Description |
|---|---|---|---|---|---|---|---|---|---|
| 2 | SBUH Runoff | 3.104 | 2 | 474 | 43,826 | ------ | ------ | ------ | DET FLOW TO POND |
| 3 | SBUH Runoff | 1.736 | 2 | 478 | 26,557 | ------ | ------ | ------ | EX FLOW TO POND |
| 6 | Reservoir | 0.849 | 2 | 554 | 43,705 | 2 | 434.06 | 12,869 | Det Flow |

# Hydrograph Report

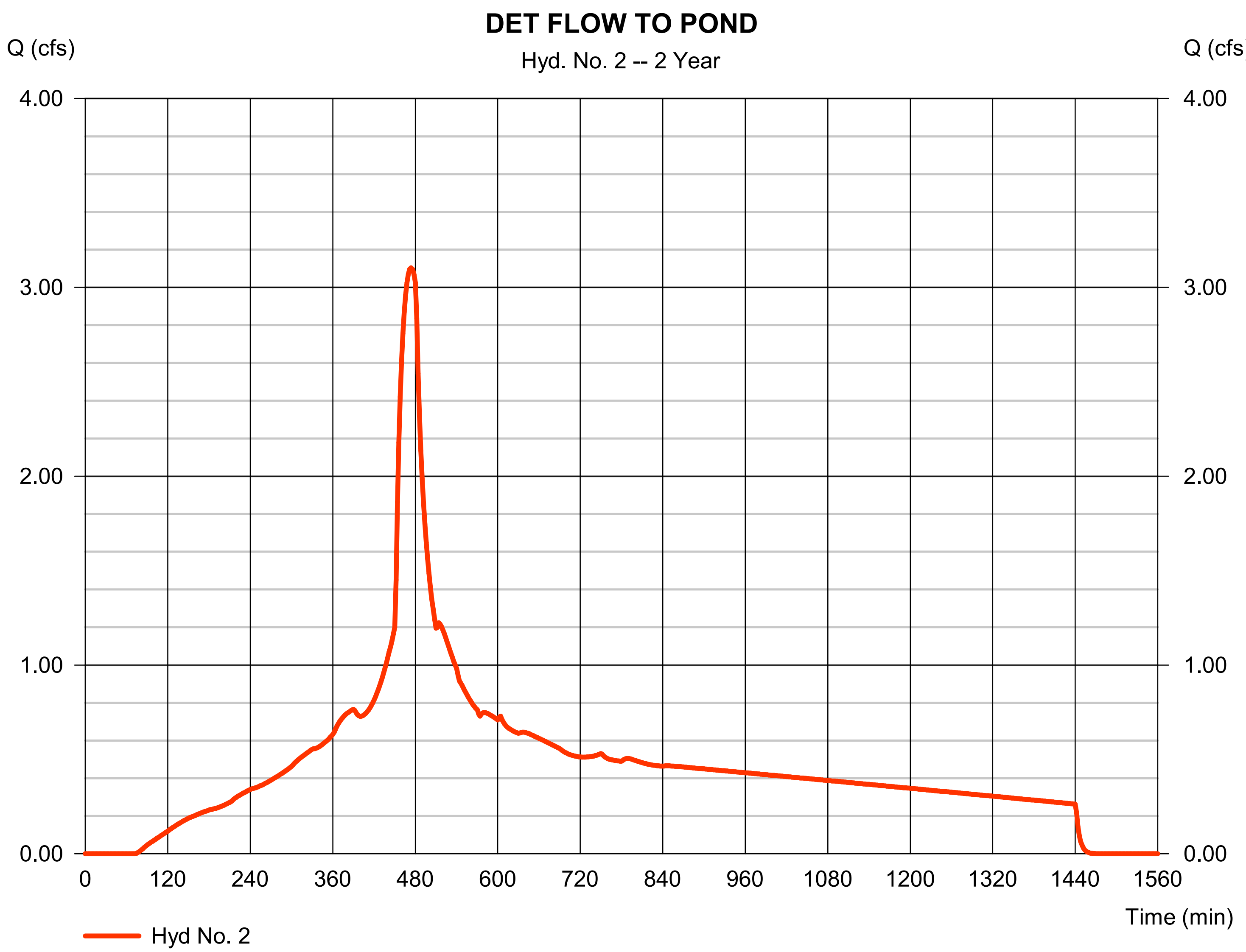
Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2021 Friday, 03 / 1 / 2024

## Hyd. No. 2

DET FLOW TO POND

| | | | |
|---|---|---|---|
| Hydrograph type | = SBUH Runoff | Peak discharge | = 3.104 cfs |
| Storm frequency | = 2 yrs | Time to peak | = 474 min |
| Time interval | = 2 min | Hyd. volume | = 43,826 cuft |
| Drainage area | = 5.580 ac | Curve number | = 97* |
| Basin Slope | = 0.0 % | Hydraulic length | = 0 ft |
| Tc method | = User | Time of conc. (Tc) | = 5.00 min |
| Total precip. | = 2.50 in | Distribution | = Type IA |
| Storm duration | = 24 hrs | Shape factor | = n/a |

* Composite (Area/CN) = [(4.910 x 98) + (0.670 x 90)] / 5.580



# Hydrograph Report

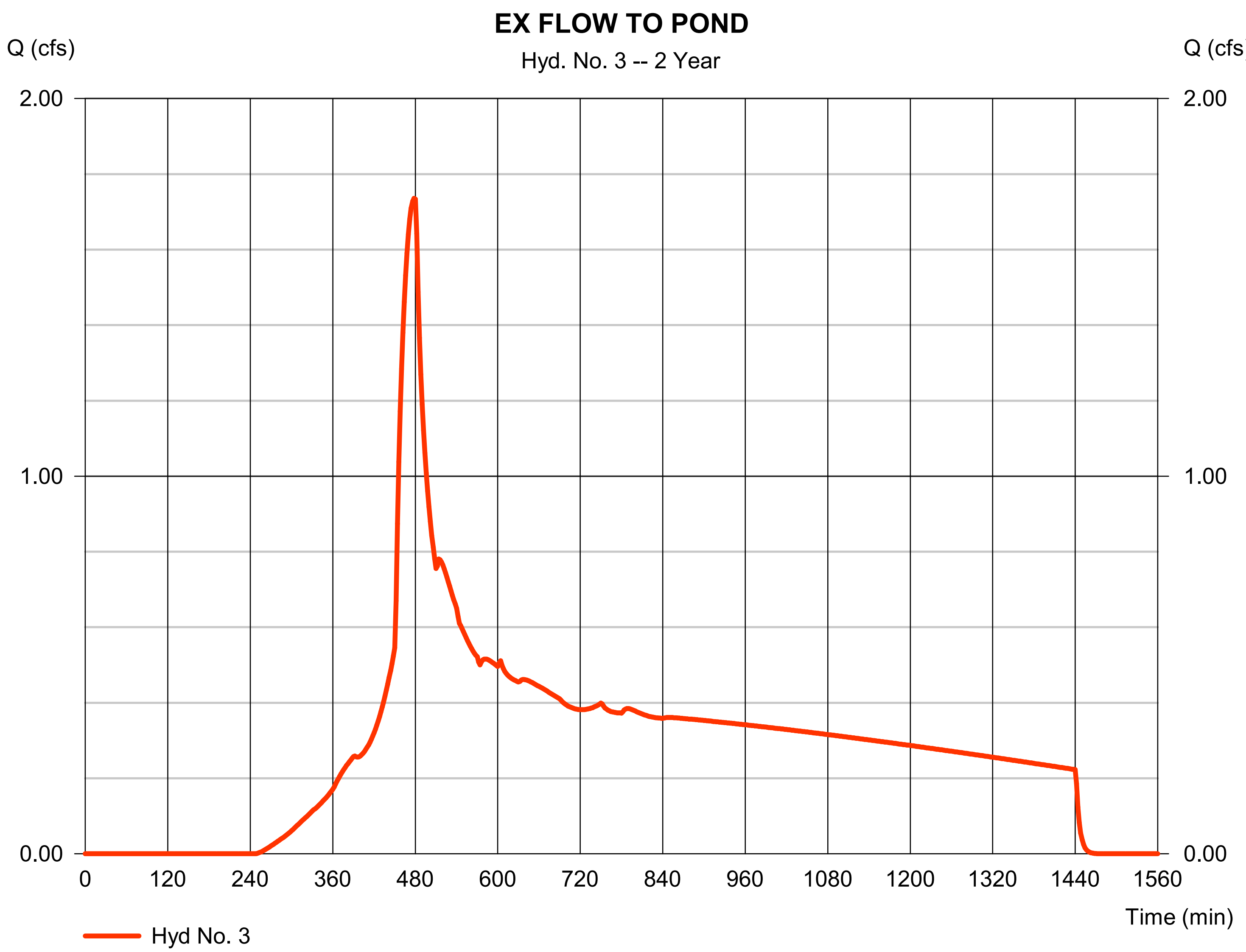
Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2021 Friday, 03 / 1 / 2024

## Hyd. No. 3

EX FLOW TO POND

| | | | |
|---|---|---|---|
| Hydrograph type | = SBUH Runoff | Peak discharge | = 1.736 cfs |
| Storm frequency | = 2 yrs | Time to peak | = 478 min |
| Time interval | = 2 min | Hyd. volume | = 26,557 cuft |
| Drainage area | = 5.580 ac | Curve number | = 87* |
| Basin Slope | = 0.0 % | Hydraulic length | = 0 ft |
| Tc method | = User | Time of conc. (Tc) | = 5.00 min |
| Total precip. | = 2.50 in | Distribution | = Type IA |
| Storm duration | = 24 hrs | Shape factor | = n/a |

* Composite (Area/CN) = [(3.290 x 98) + (2.290 x 72)] / 5.580



# Hydrograph Report

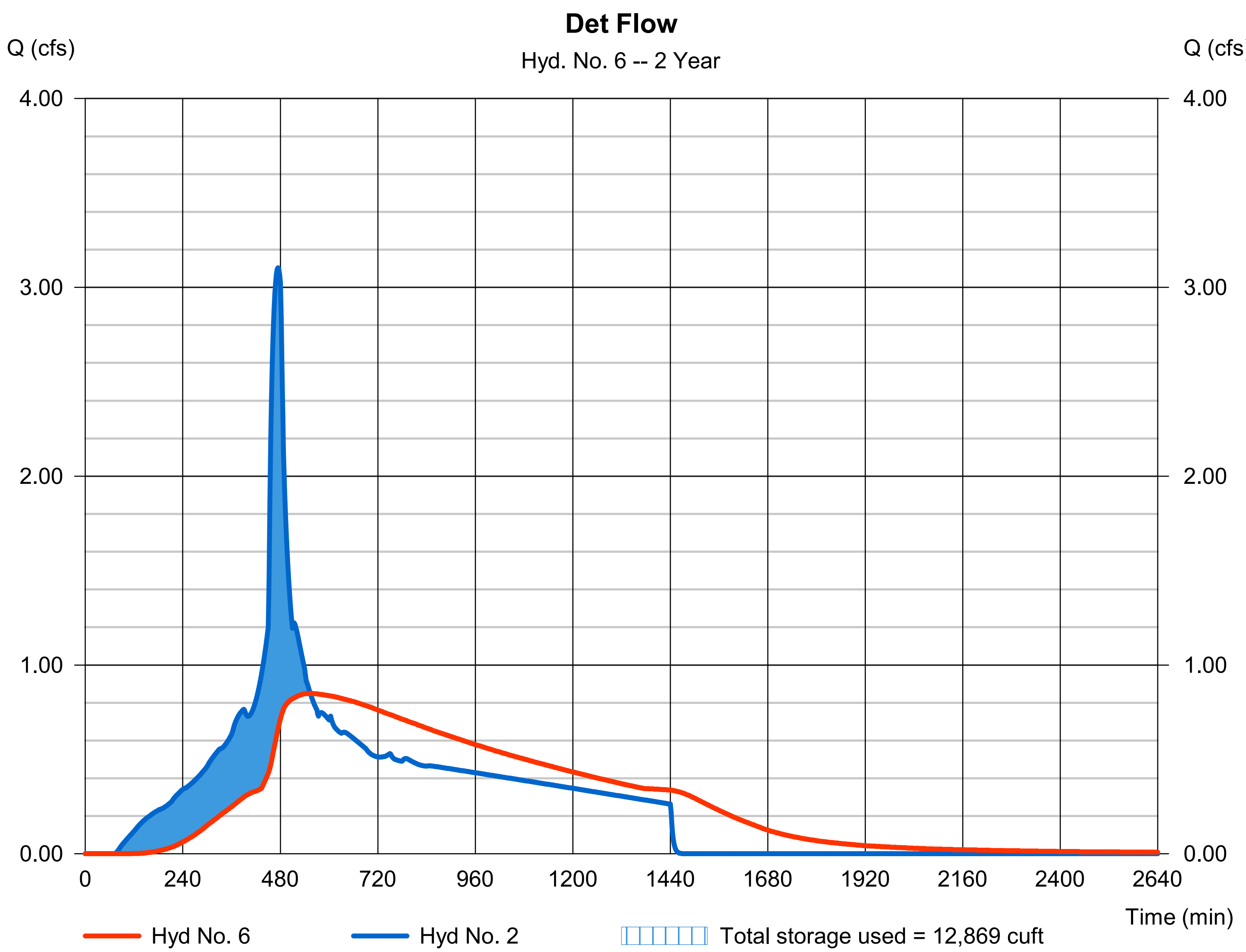
Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2021 Friday, 03 / 1 / 2024

## Hyd. No. 6

Det Flow

| | | | |
|---|---|---|---|
| Hydrograph type | = Reservoir | Peak discharge | = 0.849 cfs |
| Storm frequency | = 2 yrs | Time to peak | = 554 min |
| Time interval | = 2 min | Hyd. volume | = 43,705 cuft |
| Inflow hyd. No. | = 2 - DET FLOW TO POND | Max. Elevation | = 434.06 ft |
| Reservoir name | = Water Quanitity | Max. Storage | = 12,869 cuft |

Storage Indication method used.



# Hydrograph Summary Report

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

| Hyd. No. | Hydrograph type (origin) | Peak flow (cfs) | Time interval (min) | Time to Peak (min) | Hyd. volume (cuft) | Inflow hyd(s) | Maximum elevation (ft) | Total strge used (cuft) | Hydrograph Description |
|---|---|---|---|---|---|---|---|---|---|
| 2 | SBUH Runoff | 3.794 | 2 | 474 | 53,845 | ------ | ------ | ------ | DET FLOW TO POND |
| 3 | SBUH Runoff | 2.376 | 2 | 478 | 35,226 | ------ | ------ | ------ | EX FLOW TO POND |
| 6 | Reservoir | 0.973 | 2 | 562 | 53,724 | 2 | 434.31 | 16,090 | Det Flow |

# Hydrograph Report

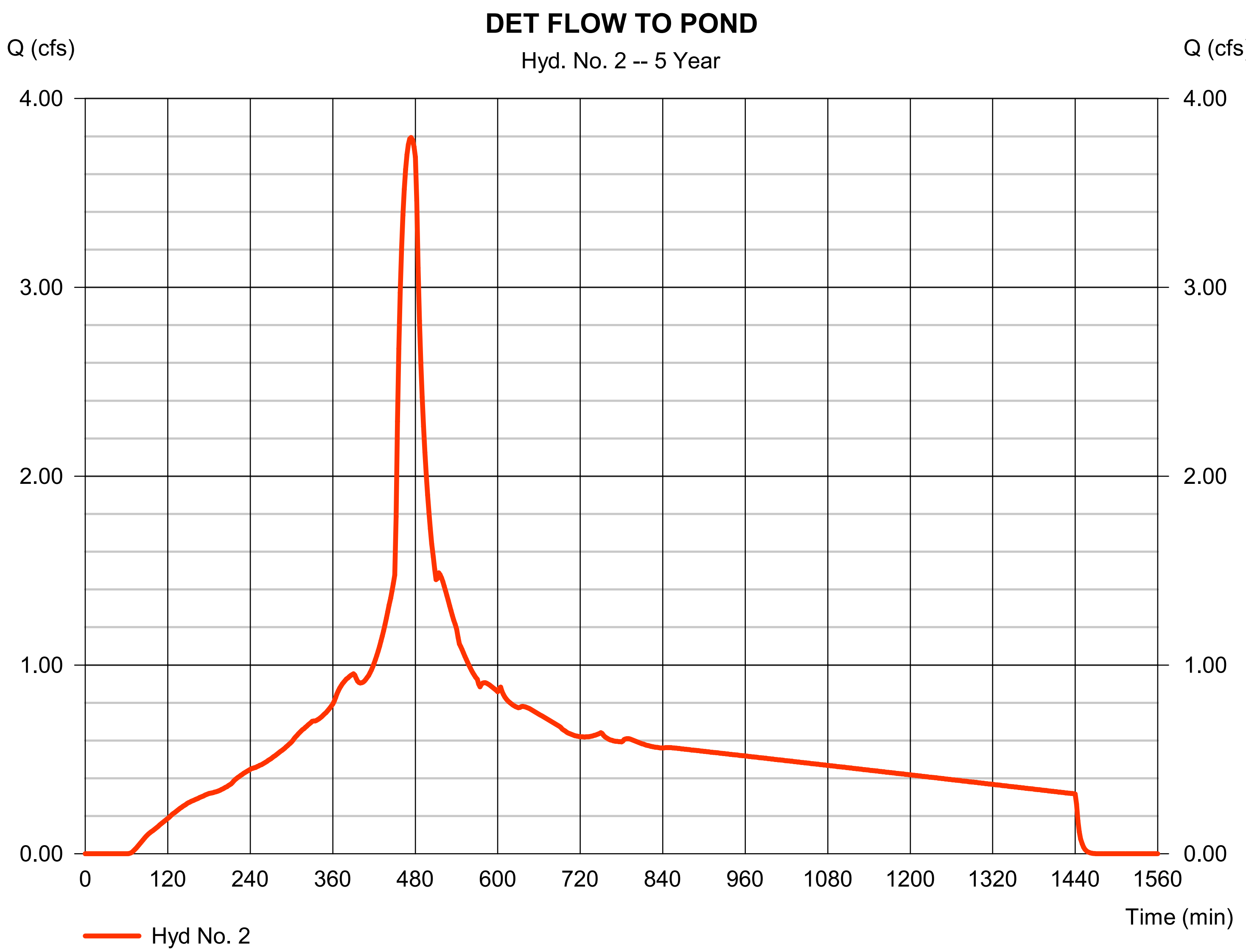


## Hyd. No. 2

DET FLOW TO POND

| | | | |
|---|---|---|---|
| Hydrograph type | = SBUH Runoff | Peak discharge | = 3.794 cfs |
| Storm frequency | = 5 yrs | Time to peak | = 474 min |
| Time interval | = 2 min | Hyd. volume | = 53,845 cuft |
| Drainage area | = 5.580 ac | Curve number | = 97* |
| Basin Slope | = 0.0 % | Hydraulic length | = 0 ft |
| Tc method | = User | Time of conc. (Tc) | = 5.00 min |
| Total precip. | = 3.00 in | Distribution | = Type IA |
| Storm duration | = 24 hrs | Shape factor | = n/a |

* Composite (Area/CN) = [(4.910 x 98) + (0.670 x 90)] / 5.580



# Hydrograph Report

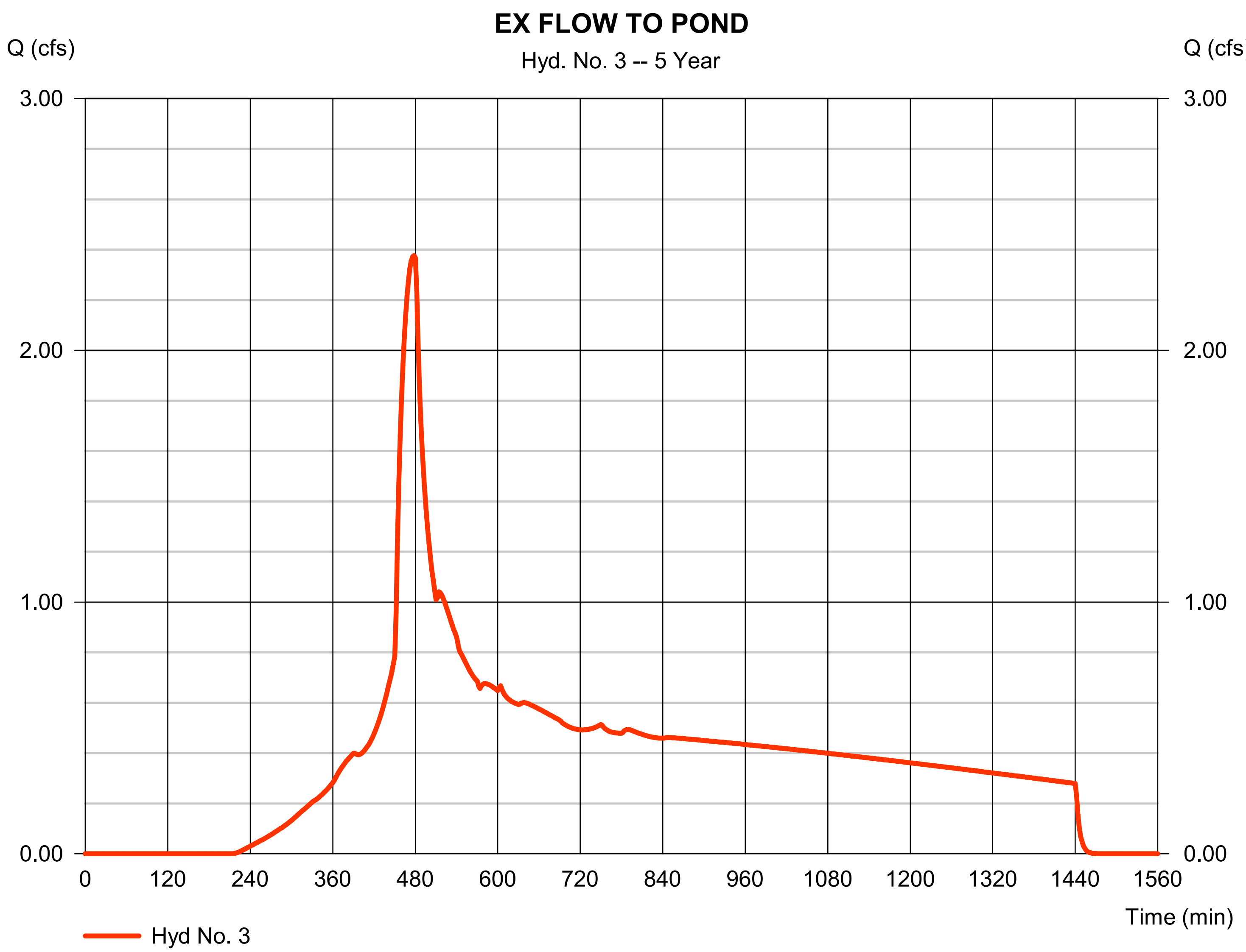
Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2021 Friday, 03 / 1 / 2024

## Hyd. No. 3

EX FLOW TO POND

| | | | |
|---|---|---|---|
| Hydrograph type | = SBUH Runoff | Peak discharge | = 2.376 cfs |
| Storm frequency | = 5 yrs | Time to peak | = 478 min |
| Time interval | = 2 min | Hyd. volume | = 35,226 cuft |
| Drainage area | = 5.580 ac | Curve number | = 87* |
| Basin Slope | = 0.0 % | Hydraulic length | = 0 ft |
| Tc method | = User | Time of conc. (Tc) | = 5.00 min |
| Total precip. | = 3.00 in | Distribution | = Type IA |
| Storm duration | = 24 hrs | Shape factor | = n/a |

* Composite (Area/CN) = [(3.290 x 98) + (2.290 x 72)] / 5.580



# Hydrograph Report

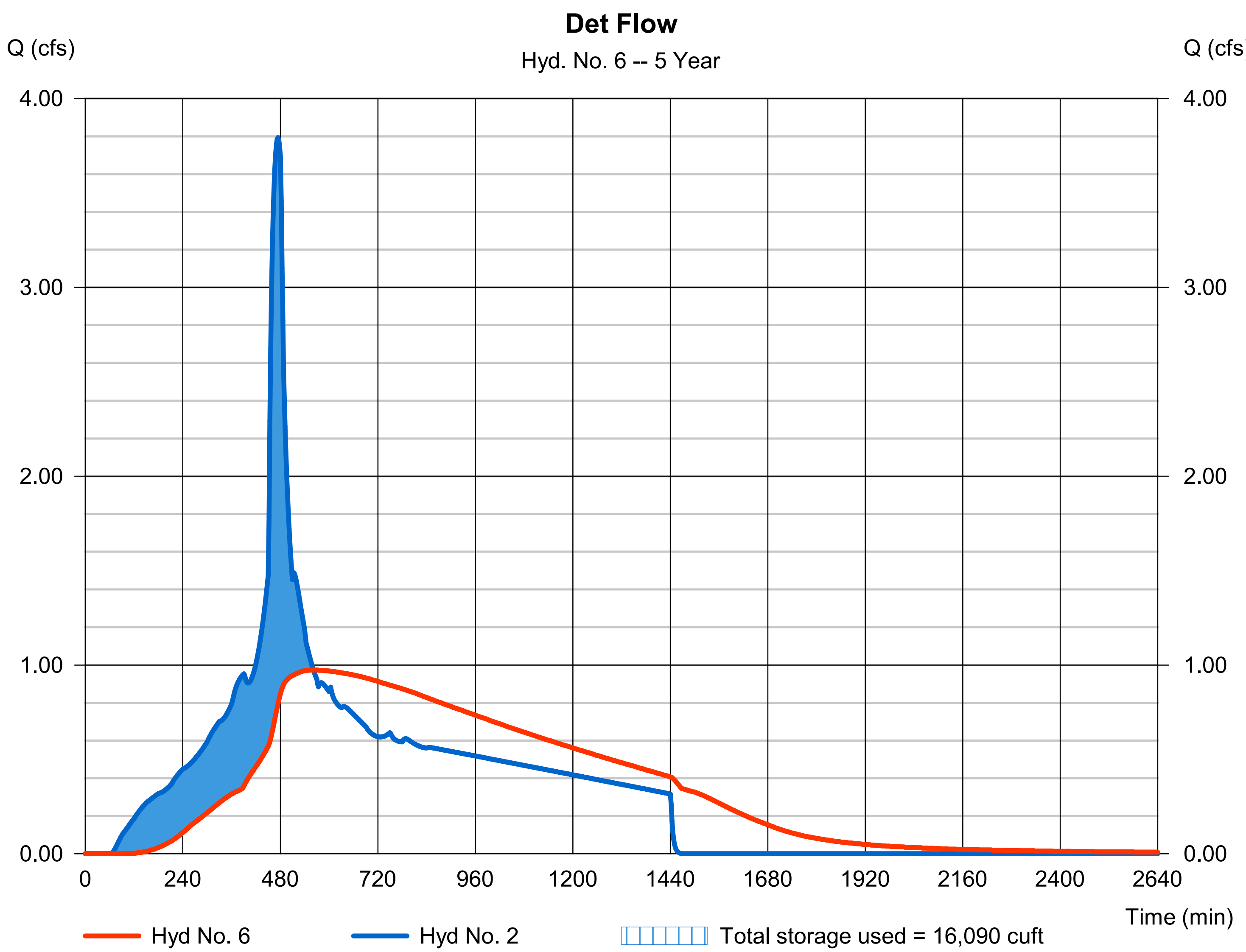
Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2021 | Friday, 03 / 1 / 2024

## Hyd. No. 6

Det Flow

| | | | |
|---|---|---|---|
| Hydrograph type | = Reservoir | Peak discharge | = 0.973 cfs |
| Storm frequency | = 5 yrs | Time to peak | = 562 min |
| Time interval | = 2 min | Hyd. volume | = 53,724 cuft |
| Inflow hyd. No. | = 2 - DET FLOW TO POND | Max. Elevation | = 434.31 ft |
| Reservoir name | = Water Quanitity | Max. Storage | = 16,090 cuft |

Storage Indication method used.



# Hydrograph Summary Report

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

| Hyd. No. | Hydrograph type (origin) | Peak flow (cfs) | Time interval (min) | Time to Peak (min) | Hyd. volume (cuft) | Inflow hyd(s) | Maximum elevation (ft) | Total strge used (cuft) | Hydrograph Description |
|---|---|---|---|---|---|---|---|---|---|
| 2 | SBUH Runoff | 4.479 | 2 | 474 | 63,893 | ------ | ------ | ------ | DET FLOW TO POND |
| 3 | SBUH Runoff | 3.039 | 2 | 476 | 44,206 | ------ | ------ | ------ | EX FLOW TO POND |
| 6 | Reservoir | 1.084 | 2 | 570 | 63,772 | 2 | 434.56 | 19,461 | Det Flow |

# Hydrograph Report

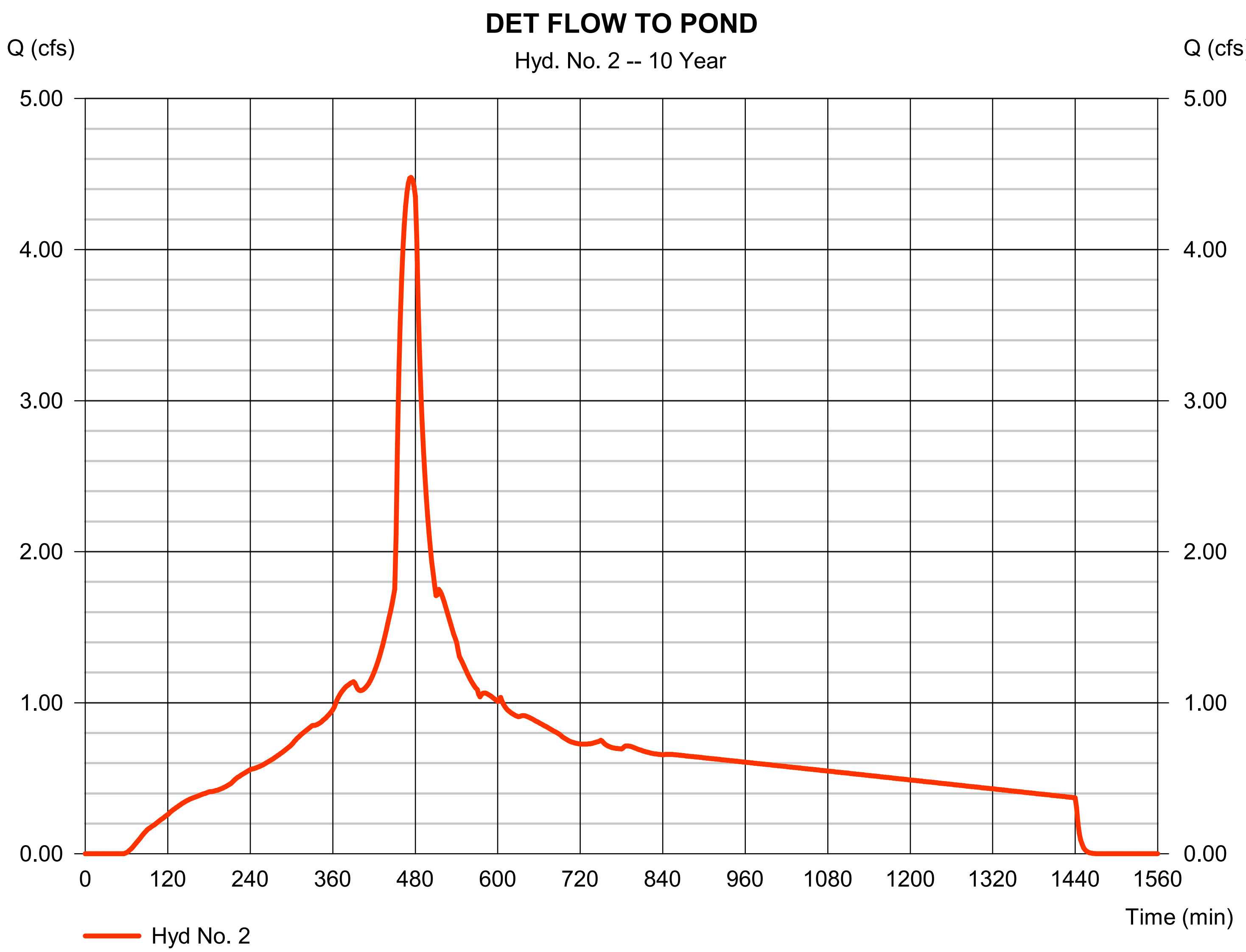
Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2021 Friday, 03 / 1 / 2024

## Hyd. No. 2

DET FLOW TO POND

| | | | |
|---|---|---|---|
| Hydrograph type | = SBUH Runoff | Peak discharge | = 4.479 cfs |
| Storm frequency | = 10 yrs | Time to peak | = 474 min |
| Time interval | = 2 min | Hyd. volume | = 63,893 cuft |
| Drainage area | = 5.580 ac | Curve number | = 97* |
| Basin Slope | = 0.0 % | Hydraulic length | = 0 ft |
| Tc method | = User | Time of conc. (Tc) | = 5.00 min |
| Total precip. | = 3.50 in | Distribution | = Type IA |
| Storm duration | = 24 hrs | Shape factor | = n/a |

* Composite (Area/CN) = [(4.910 x 98) + (0.670 x 90)] / 5.580



# Hydrograph Report

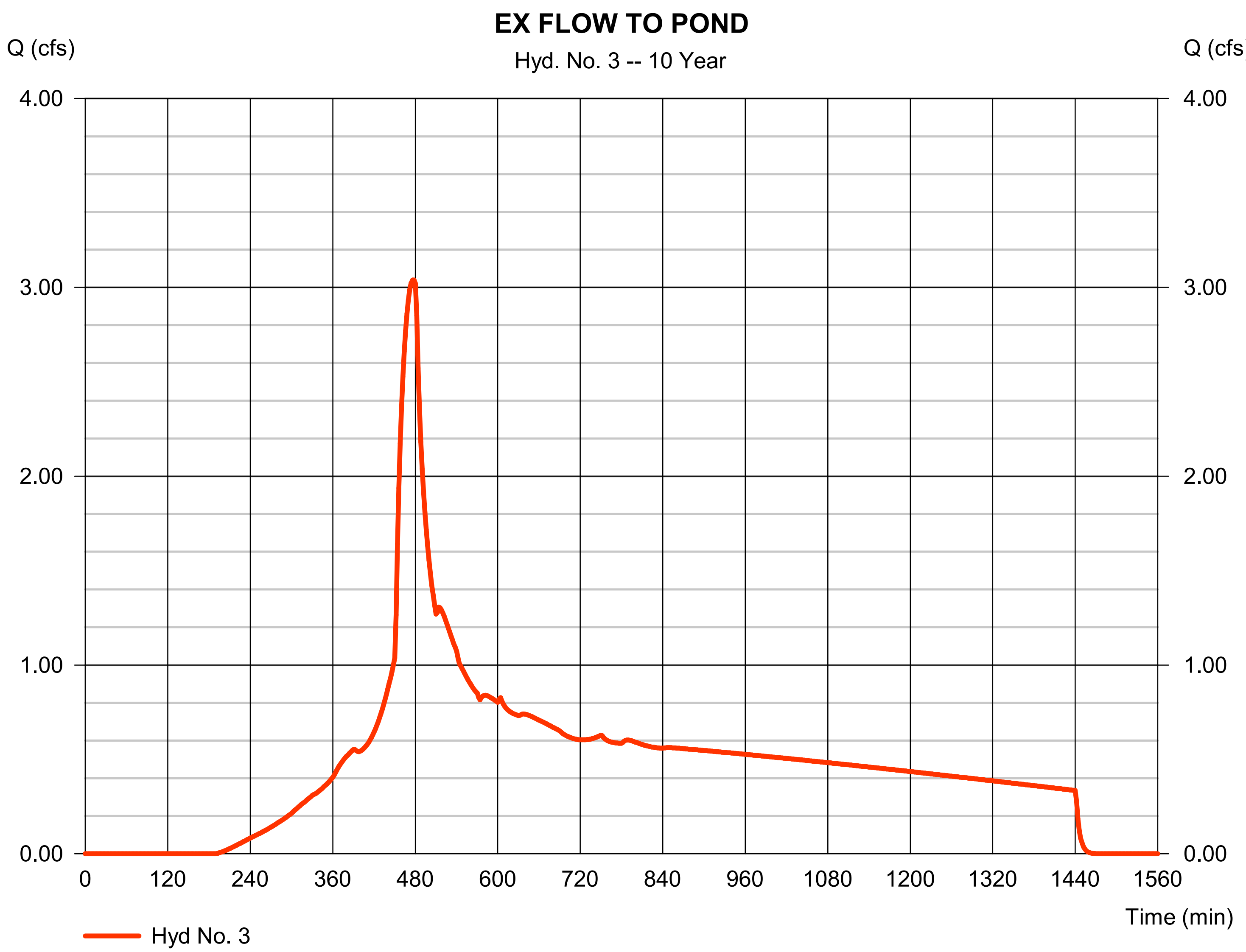
Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2021 | Friday, 03 / 1 / 2024

## Hyd. No. 3

EX FLOW TO POND

| | | | |
|---|---|---|---|
| Hydrograph type | = SBUH Runoff | Peak discharge | = 3.039 cfs |
| Storm frequency | = 10 yrs | Time to peak | = 476 min |
| Time interval | = 2 min | Hyd. volume | = 44,206 cuft |
| Drainage area | = 5.580 ac | Curve number | = 87* |
| Basin Slope | = 0.0 % | Hydraulic length | = 0 ft |
| Tc method | = User | Time of conc. (Tc) | = 5.00 min |
| Total precip. | = 3.50 in | Distribution | = Type IA |
| Storm duration | = 24 hrs | Shape factor | = n/a |

* Composite (Area/CN) = [(3.290 x 98) + (2.290 x 72)] / 5.580



# Hydrograph Report

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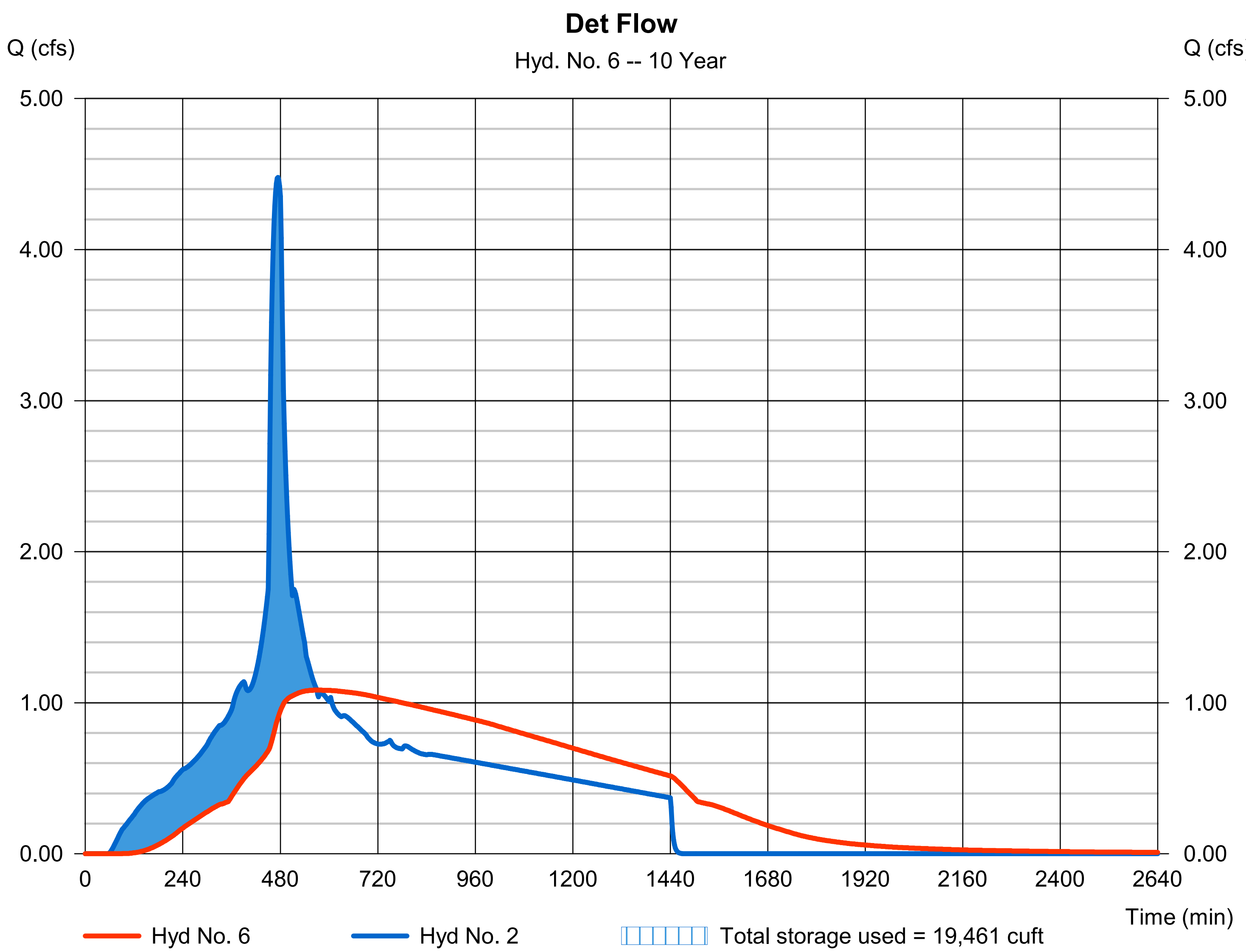
## Hyd. No. 6

Det Flow

| | | | |
|---|---|---|---|
| Hydrograph type | = Reservoir | Peak discharge | = 1.084 cfs |
| Storm frequency | = 10 yrs | Time to peak | = 570 min |
| Time interval | = 2 min | Hyd. volume | = 63,772 cuft |
| Inflow hyd. No. | = 2 - DET FLOW TO POND | Max. Elevation | = 434.56 ft |
| Reservoir name | = Water Quanitity | Max. Storage | = 19,461 cuft |

Storage Indication method used.



# Hydrograph Summary Report

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| 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 |
|---|---|---|---|---|---|---|---|---|---|
| 2 | SBUH Runoff | 5.160 | 2 | 474 | 73,960 | ------ | ------ | ------ | DET FLOW TO POND |
| 3 | SBUH Runoff | 3.721 | 2 | 476 | 53,407 | ------ | ------ | ------ | EX FLOW TO POND |
| 6 | Reservoir | 1.185 | 2 | 594 | 73,839 | 2 | 434.82 | 22,988 | Det Flow |

# Hydrograph Report

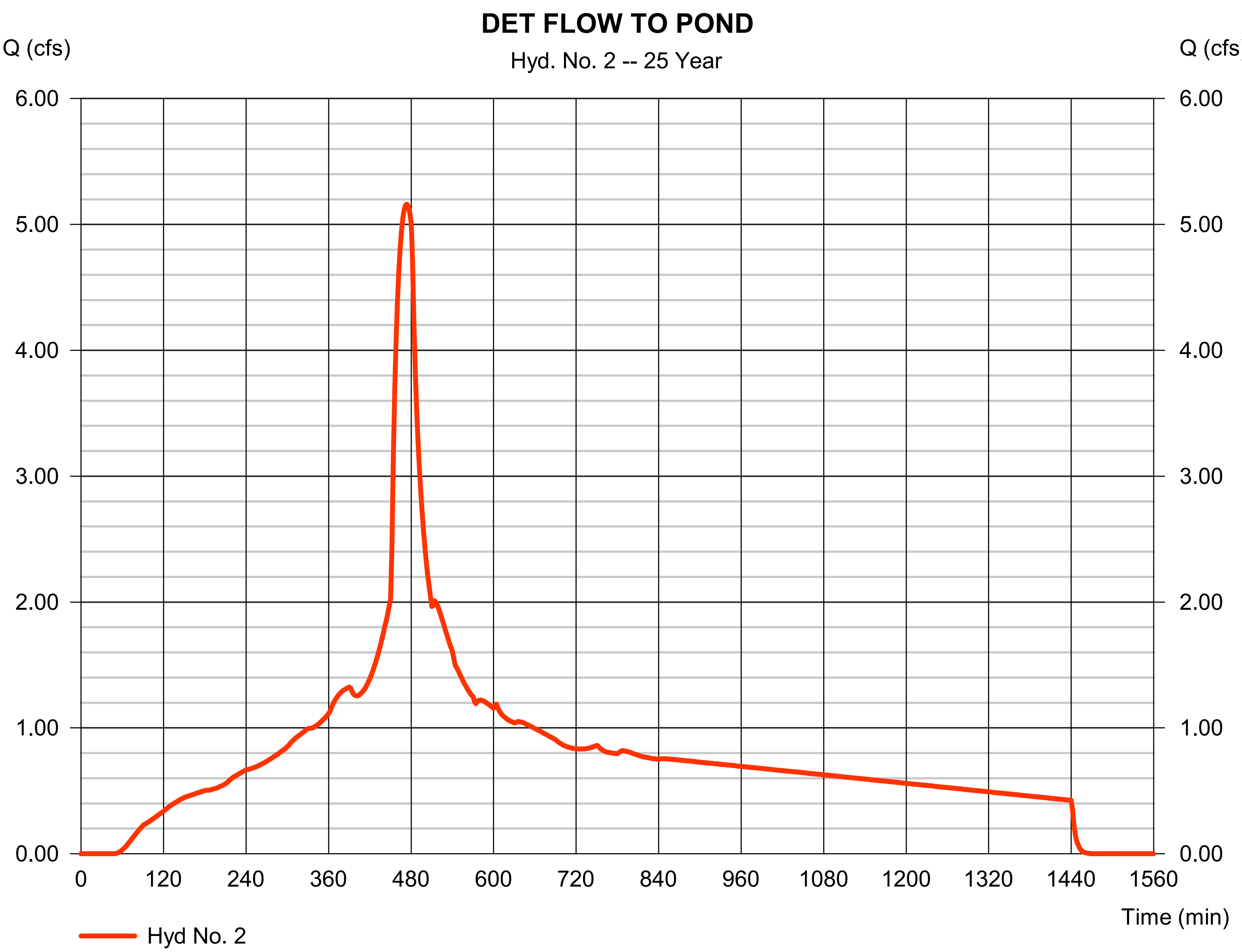
Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2021 Friday, 03 / 1 / 2024

## Hyd. No. 2

DET FLOW TO POND

| | | | |
|---|---|---|---|
| Hydrograph type | = SBUH Runoff | Peak discharge | = 5.160 cfs |
| Storm frequency | = 25 yrs | Time to peak | = 474 min |
| Time interval | = 2 min | Hyd. volume | = 73,960 cuft |
| Drainage area | = 5.580 ac | Curve number | = 97* |
| Basin Slope | = 0.0 % | Hydraulic length | = 0 ft |
| Tc method | = User | Time of conc. (Tc) | = 5.00 min |
| Total precip. | = 4.00 in | Distribution | = Type IA |
| Storm duration | = 24 hrs | Shape factor | = n/a |

* Composite (Area/CN) = [(4.910 x 98) + (0.670 x 90)] / 5.580



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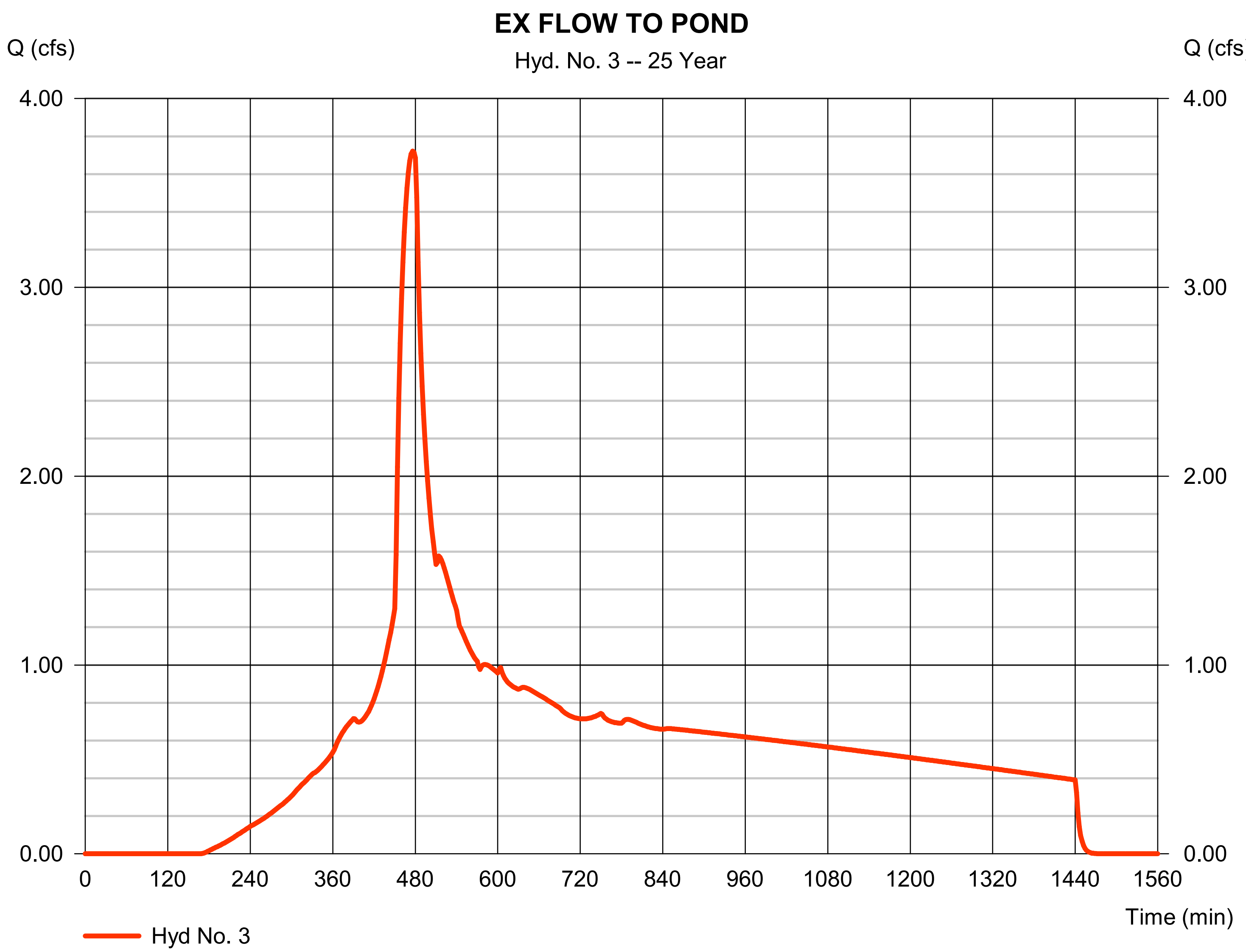
Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2021 Friday, 03 / 1 / 2024

## Hyd. No. 3

EX FLOW TO POND

| | | | |
|---|---|---|---|
| Hydrograph type | = SBUH Runoff | Peak discharge | = 3.721 cfs |
| Storm frequency | = 25 yrs | Time to peak | = 476 min |
| Time interval | = 2 min | Hyd. volume | = 53,407 cuft |
| Drainage area | = 5.580 ac | Curve number | = 87* |
| Basin Slope | = 0.0 % | Hydraulic length | = 0 ft |
| Tc method | = User | Time of conc. (Tc) | = 5.00 min |
| Total precip. | = 4.00 in | Distribution | = Type IA |
| Storm duration | = 24 hrs | Shape factor | = n/a |

* Composite (Area/CN) = [(3.290 x 98) + (2.290 x 72)] / 5.580



# Hydrograph Report

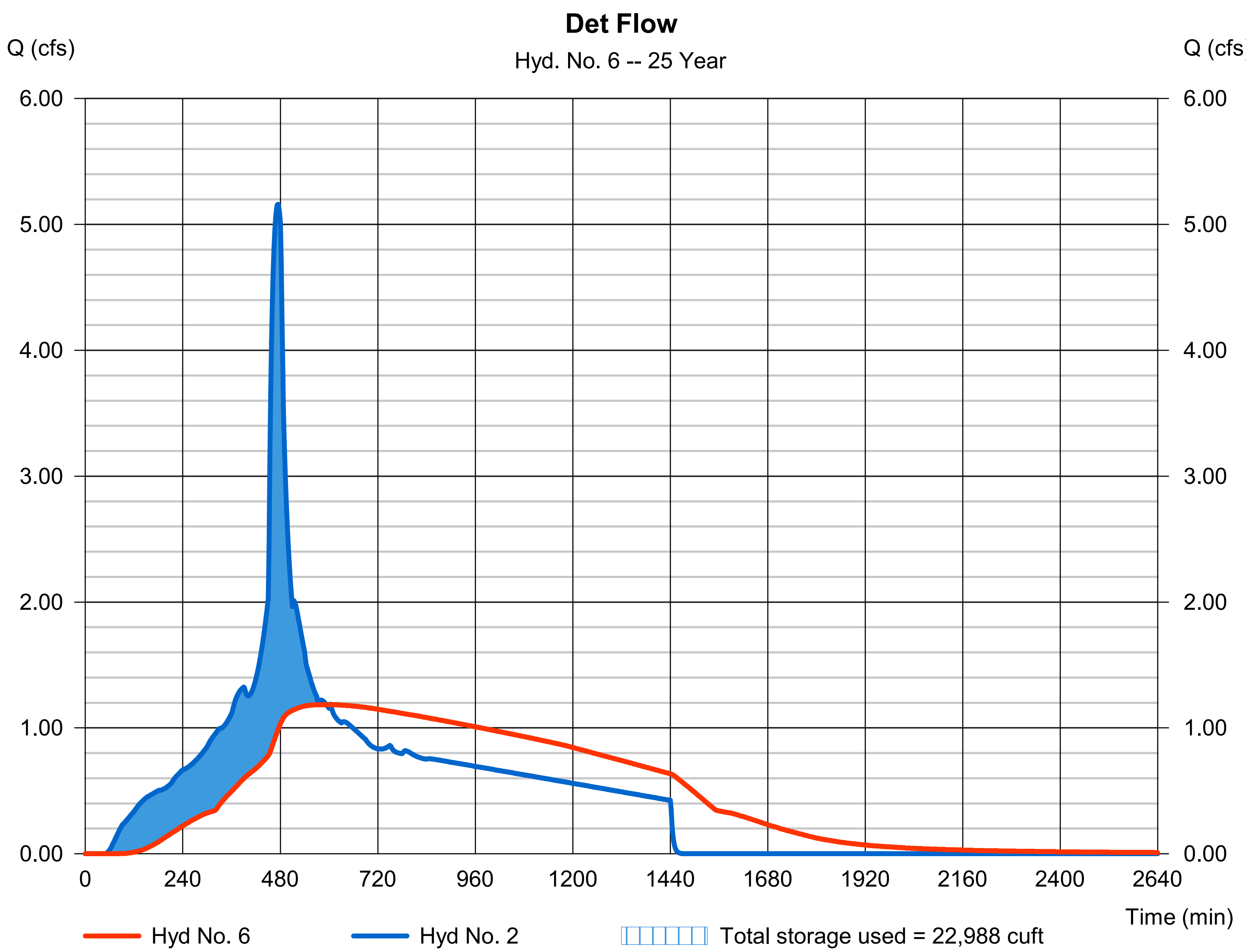
Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2021 Friday, 03 / 1 / 2024

## Hyd. No. 6

Det Flow

| | | | |
|---|---|---|---|
| Hydrograph type | = Reservoir | Peak discharge | = 1.185 cfs |
| Storm frequency | = 25 yrs | Time to peak | = 594 min |
| Time interval | = 2 min | Hyd. volume | = 73,839 cuft |
| Inflow hyd. No. | = 2 - DET FLOW TO POND | Max. Elevation | = 434.82 ft |
| Reservoir name | = Water Quanitity | Max. Storage | = 22,988 cuft |

Storage Indication method used.



# Hydrograph Summary Report

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

| Hyd. No. | Hydrograph type (origin) | Peak flow (cfs) | Time interval (min) | Time to Peak (min) | Hyd. volume (cuft) | Inflow hyd(s) | Maximum elevation (ft) | Total strge used (cuft) | Hydrograph Description |
|---|---|---|---|---|---|---|---|---|---|
| 2 | SBUH Runoff | 5.840 | 2 | 474 | 84,040 | ------ | ------ | ------ | DET FLOW TO POND |
| 3 | SBUH Runoff | 4.412 | 2 | 476 | 62,770 | ------ | ------ | ------ | EX FLOW TO POND |
| 6 | Reservoir | 1.281 | 2 | 608 | 83,918 | 2 | 435.09 | 26,703 | Det Flow |

| | | |
|---|---|---|
| 389-pond sizing.gpw | Return Period: 100 Year | Friday, 03 / 1 / 2024 |

# Hydrograph Report

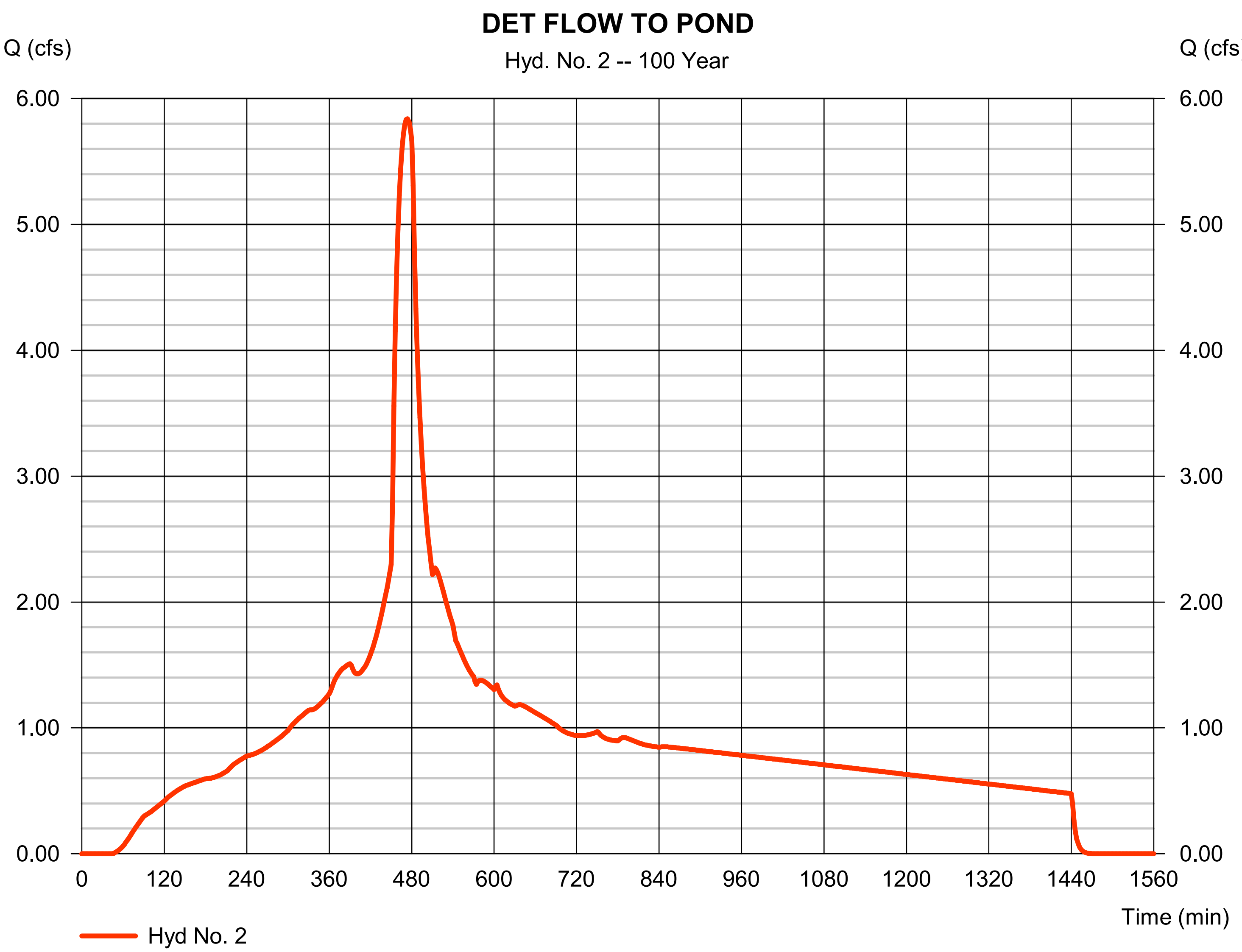
Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2021 — Friday, 03 / 1 / 2024

## Hyd. No. 2

DET FLOW TO POND

| | | | |
|---|---|---|---|
| Hydrograph type | = SBUH Runoff | Peak discharge | = 5.840 cfs |
| Storm frequency | = 100 yrs | Time to peak | = 474 min |
| Time interval | = 2 min | Hyd. volume | = 84,040 cuft |
| Drainage area | = 5.580 ac | Curve number | = 97* |
| Basin Slope | = 0.0 % | Hydraulic length | = 0 ft |
| Tc method | = User | Time of conc. (Tc) | = 5.00 min |
| Total precip. | = 4.50 in | Distribution | = Type IA |
| Storm duration | = 24 hrs | Shape factor | = n/a |

* Composite (Area/CN) = [(4.910 x 98) + (0.670 x 90)] / 5.580



# Hydrograph Report

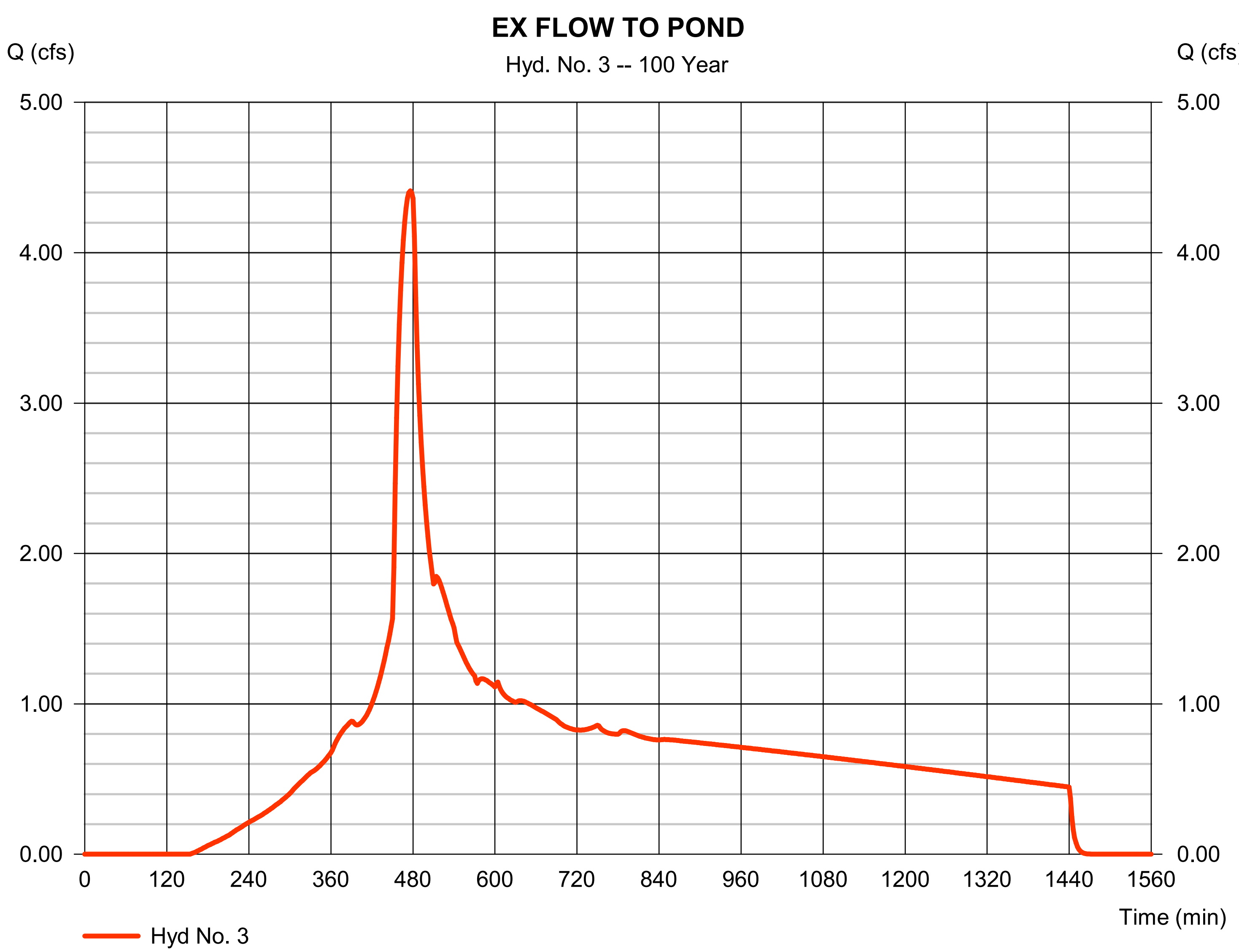
Hydraflow Hydrographs Extension for Autodesk® Civil 3D® by Autodesk, Inc. v2021 Friday, 03 / 1 / 2024

## Hyd. No. 3

EX FLOW TO POND

| | | | |
|---|---|---|---|
| Hydrograph type | = SBUH Runoff | Peak discharge | = 4.412 cfs |
| Storm frequency | = 100 yrs | Time to peak | = 476 min |
| Time interval | = 2 min | Hyd. volume | = 62,770 cuft |
| Drainage area | = 5.580 ac | Curve number | = 87* |
| Basin Slope | = 0.0 % | Hydraulic length | = 0 ft |
| Tc method | = User | Time of conc. (Tc) | = 5.00 min |
| Total precip. | = 4.50 in | Distribution | = Type IA |
| Storm duration | = 24 hrs | Shape factor | = n/a |

* Composite (Area/CN) = [(3.290 x 98) + (2.290 x 72)] / 5.580



# Hydrograph Report

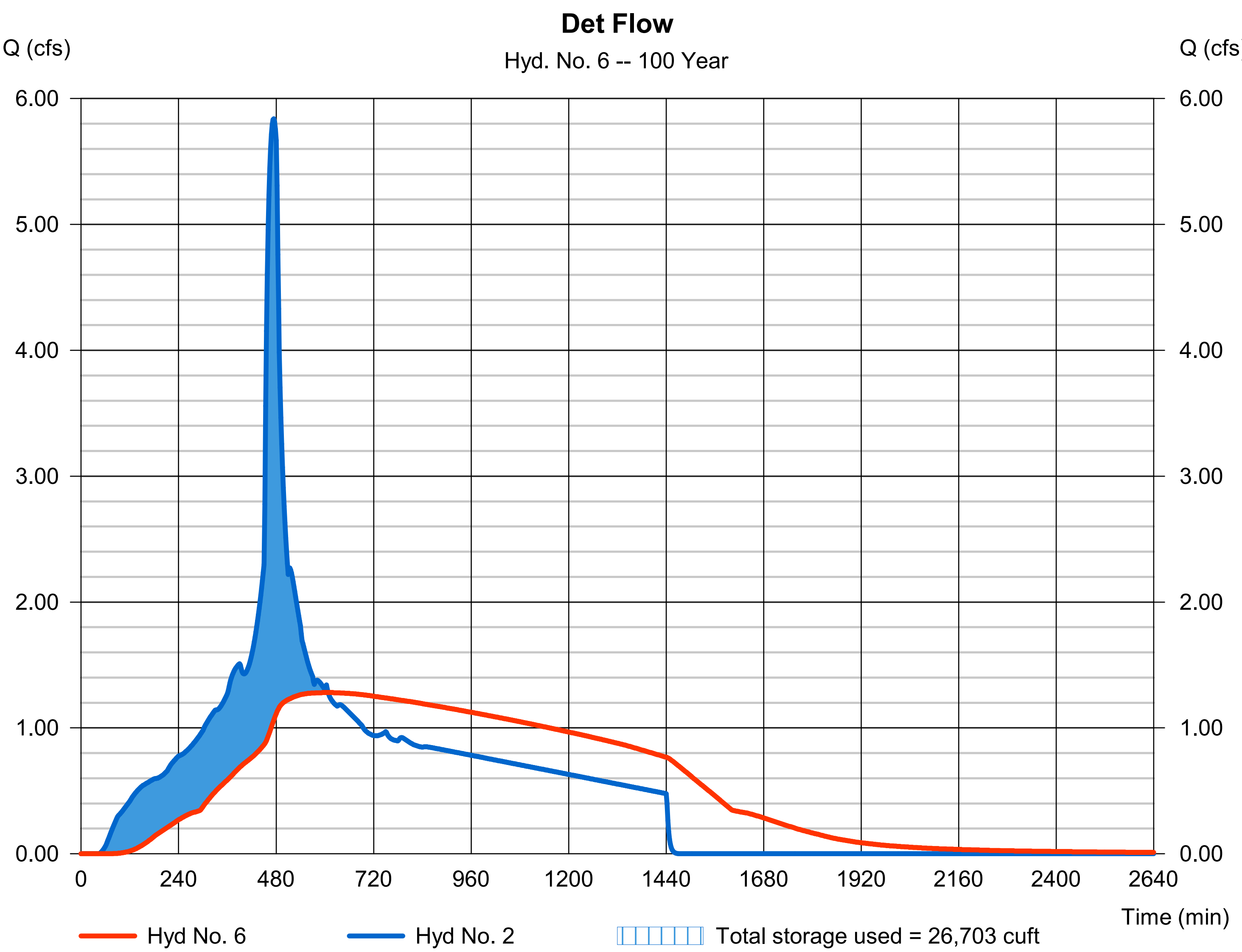


## Hyd. No. 6

Det Flow

| | | | |
|---|---|---|---|
| Hydrograph type | = Reservoir | Peak discharge | = 1.281 cfs |
| Storm frequency | = 100 yrs | Time to peak | = 608 min |
| Time interval | = 2 min | Hyd. volume | = 83,918 cuft |
| Inflow hyd. No. | = 2 - DET FLOW TO POND | Max. Elevation | = 435.09 ft |
| Reservoir name | = Water Quanitity | Max. Storage | = 26,703 cuft |

Storage Indication method used.



APPENDIX B
**GEOTECHNICAL REPORT**

# Report of Geotechnical Engineering Services

## Santiam Industrial Center

## Stayton, Oregon

December 6, 2023

Geotechnical ▪ Environmental ▪ Special Inspections

Columbia West Engineering, Inc




www.columbiawestengineering.com

Geotechnical ▪ Environmental ▪ Special Inspections
Columbia West
Engineering, Inc

December 6, 2023

Santiam Industrial Center
c/o IRG Realty Advisors, LLC
4020 Kinross Lakes Parkway, Suite 200
Richfield, OH 44286
Attention: Coby Holley

**Report of Geotechnical Engineering Services**
Santiam Industrial Center
930 W Washington Street
Stayton, Oregon
Columbia West Project: IRG-1-01-1

Columbia West is pleased to present this geotechnical report for the new building at the Santiam Industrial Center in Stayton, Oregon. Our services were conducted in accordance with our proposal dated September 1, 2023.

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

Sincerely,

Columbia West

REGISTERED PROFESSIONAL ENGINEER
86690
OREGON
May 8, 2012
NICHOLAS N. PAVEGLIO
EXPIRES: 12/31/24

Nick Paveglio, PE
Principal Engineer

Cc: Collin Wong, NBS Real Estate Consulting

NNP:glw

Attachments
Document ID: IRG-1-01-1-120623-geor

## EXECUTIVE SUMMARY

This section provides a summary of the geotechnical considerations associated with the proposed new building at the Santiam Industrial Center in Stayton, Oregon. This summary is an overview and the report should be referenced for a thorough discussion of subsurface conditions and geotechnical recommendations for the project.

- Up to 3.5 feet of undocumented fill is present at the site. The undocumented fill may not be suitable to support foundations. All proposed structural improvements should be supported on conventional spread footings on firm native soil or structural fill on firm native soil. Alternatively, spread footings can be founded on undocumented fill if it is evaluated by Columbia West if the owner is willing to accept a slightly higher risk of settlement.

- Floor slabs and pavements can be constructed on undocumented fill, provided they are evaluated as described in the "Construction" section and the owner is willing to accept a small risk of poor slab and pavement performance.

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

- Cobbles and possibly boulders are present at the site. Cobbles and especially boulders will result in difficult excavations.

- Groundwater was observed between depths of 5 and 6.5 feet BGS during explorations at the site. Contractors should be prepared to dewater excavations that extend more than a few feet BGS.

- When exposed, the fine-grained soil beneath the existing pavement sections is sensitive to disturbance when at a moisture content that is above optimum. As discussed in the report, the fine-grained subgrade should be protected from disturbance and damage by construction traffic when exposed.

- Moisture conditioning will be likely be required to use the onsite material as structural fill. Accordingly, extended dry weather will be required to adequately condition and place the soil as structural fill. It will be difficult, if not impossible, to adequately compact on-site soil during the rainy season or during prolonged periods of rainfall.

- Based on the subsurface soil and shallow groundwater conditions, we do not anticipate infiltration will be effective for stormwater management.

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# REPORT OF GEOTECHNICAL ENGINEERING SERVICES
# SANTIAM INDUSTRIAL CENTER
# STAYTON, OREGON

## 1.0 INTRODUCTION

Columbia West is pleased to submit this geotechnical report for improvements at the Santiam Industrial Center in Stayton, Oregon. The approximately 33-acre site is located at 930 W Washington Street. The site is currently occupied by a large, irregular-shaped building with a footprint of approximately 500,000 square feet. An ancillary building with a footprint of approximately 40,000 is located east of the larger building and the remainder of the site is predominately covered by gravel, asphalt concrete, and cement parking areas and drive aisles. The site is shown relative to surrounding physical features on Figure 1. Figure 2 shows existing conditions at the site.

Improvements at the site include the following:

- Demolition and resurfacing in the truck court near the southeast portion of the building
- Construction of new dock doors
- Select slab replacement
- New utilities

Structural loading was unknown at the time of this report, however, we anticipate maximum column and wall loads will be less than 100 kips and 4 kips per foot, respectively. We anticipate cuts and fills will be less than a few feet. Stormwater will be collected and potentially transported to a facility in the southern portion of the site.

## 2.0 PURPOSE AND SCOPE

The purpose of our services was to provide geotechnical engineering recommendations for the planned improvements. The specific scope of our services included the following:

- Reviewed information available in Columbia West's files in the vicinity of the site.
- Coordinated and managed the field investigation, including utility locates and Columbia West staff.
- Drilled four borings to depths between approximately 14 and 17 feet below ground surface (BGS).
- Collected geotechnical soil samples from the explorations for laboratory testing and maintained a log of encountered soil and groundwater conditions in the explorations.
- Completed a laboratory testing program, including the following tests:
  - Seven moisture content determinations in general accordance with ASTM D2216
  - Six particle-size analyses in general accordance with ASTM C117 or ASTM D1140
  - Three organic content determinations in accordance with ASTM D2974
- Prepared this geotechnical report that summarizes our explorations, laboratory testing, and analyses and provides geotechnical design criteria and construction recommendations for the development, including information relating to the following:

- Soil and groundwater conditions
- Seismic hazards
- Consolidation potential
- Foundation support
- Seismic design criteria
- Earthwork recommendations
- Drainage recommendations
- Design groundwater elevations
- Pavement recommendations
- Infiltration systems

## 3.0 SITE CONDITIONS

### *3.1 GEOLOGY*

The near surface geology at the site is mapped as Quaternary-aged alluvial deposits, consisting of silt, clay, sand and gravel, in floodplains of the Willamette River and major tributaries (O'Connor et al. 2001).

Based on review of the NRCS Web Soil Survey, the near-surface soil conditions in the central and north portion of the property consist of Camas gravelly sandy loam. The upper five feet of the soil profile is described as approximately 1-foot of sandy loam underlain by 4 feet of very gravelly sand. The southern portion of the property near the Salem ditch is mapped as Cloquato silt loam. The soil profile in the upper 5 feet is described as silt with variable proportions of sand.

### *3.2 SURFACE CONDITIONS*

The approximately 33-acre site is located at 930 W Washington Street. The site is bound by W Washington Street to the north, residential properties to the east, the Salem Ditch to the south, and W Washington Street and the Salem Ditch to the west.

The site currently occupied by a large, irregular-shaped building with a footprint of approximately 500,000 square feet. An ancillary building with a footprint of approximately 40,000 square feet is located east of the larger building and the remainder of the site is predominately covered by gravel, asphalt concrete, and cement parking areas and drive aisles.

### *3.3 SUBSURFACE CONDITIONS*

#### 3.3.1 General

Subsurface conditions at the site were explored by completing four drilled borings (B-1 through B-4) to depths between approximately 14 and 17 feet BGS. In addition to Columbia West's explorations, we obtained well logs from recent environmental borings completed at the site. Locations of Columbia West's explorations and well logs are shown on Figure 2. Descriptions of Columbia West's explorations and the boring logs are presented in Appendix A. Laboratory testing results from samples collected in the explorations are included in Appendix B. Water well logs are included in Appendix C. A summary of the subsurface conditions are presented below.

### 3.3.2 Soil Conditions

#### *3.3.2.1 Pavement Section*

A pavement section consisting of six inches of asphalt concrete over 12 to 24 inches of crushed aggregate was encountered in borings B-2 and B-3.

#### *3.3.2.2 Undocumented Fill*

Undocumented fill consisting of 30 inches of crushed aggregate and medium dense to dense silty gravel were encountered in borings B-1 and B-4, respectively. The fill is moist and brown to grey. The fill in boring B-4 contained cobbles up to 6 inches in diameter.

#### *3.3.2.3 Native Silt*

Native silt is present below the fill or pavement section. The silt is generally soft to medium stiff, brown to black, and moist. It has low to medium plasticity and trace to minor organics. The silt ranged in thickness from 2.5 to 14.5 feet in Columbia West's explorations.

Laboratory testing indicates the organic content of the upper silt ranges from 4 to 6 percent. This is a higher than typical organic content for Willamette Valley silt. Based on historical imagery and mapping, the Salem Ditch previously traversed the southern portion of the site. We anticipate that higher than typical organics is a result of the proximity to the former ditch where low lying areas adjacent to channels have the propensity for higher organic contents. Due to the presence of organics, the silt is likely slightly more compressible than similar soil and primary settlement times may be longer than normal.

#### *3.3.2.4 Native Sand*

Loose to very loose silty sand to sand with silt was present below the silt in borings B-2 and B-3. The sand is brown to grey and wet and fine to medium textured. It ranges in thickness between 2.5 and 5 feet.

#### *3.3.2.5 Gravel*

Gravel is present below the silt or sand to the maximum depth explored in all explorations. The gravel is very dense and grey with variable proportions of silt and sand. It is wet and poorly graded. The gravel has low compressibility and high strength. The depth to gravel from the existing ground surface is shown on Figure 2.

### 3.3.3 Groundwater

Groundwater was encountered between 5 and 6.5 feet BGS in the explorations completed at the site. The locations and depths of encountered groundwater are shown on Figure 2. Water well logs included in Appendix C encountered groundwater at similar levels. We anticipate that groundwater could be several feet higher than observed during prolonged rain events.

## *3.4 SEISMIC HAZARDS*

### 3.4.1 Seismic Setting

#### *3.4.1.1 Earthquake Sources*

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

#### *3.4.1.2 Regional Events*

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

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

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

#### *3.4.1.3 Local Events*

A significant earthquake could occur on a local fault near the site within the design life of the facility. Such an event would cause ground shaking at the site that could be more intense than the CSZ events, although the duration would be shorter. Table 1 provides details of the closest mapped faults to the site.

**Table 1. Nearest Mapped Crustal Faults**

| Source | Closest Mapped Distance[1] (km) | Mapped Length[1] (km) |
|---|---|---|
| Turner and Mill Creek Faults | 9.2 | 18 |
| Mountain Angel Fault | 24 | 30 |

1. Based on mapping by USGS

### 3.4.2 Seismic Settlement

Liquefaction is caused by a rapid increase in pore water pressure that reduces the effective stress between soil particles to near zero. Granular soil, which relies on interparticle friction for strength, is susceptible to liquefaction until the excess pore pressures can dissipate. In general, loose, saturated sand soil with low silt and clay content is the most susceptible to liquefaction. Silty soil with low plasticity can be susceptible to strain softening under relatively higher levels of ground shaking. Strain softened soils have volumetric strains much smaller than liquefiable soils due to matrix effects.

Based on the results of explorations, a thin layer of silty sand is present below the groundwater in isolated locations of the site. Based on the silt content within the sand, the soil will be matrix controlled and liquefaction is expected to be negligible at the site.

#### 3.4.3 Lateral Spreading

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

Because liquefiable is expected to negligible and there is no open face at or adjacent to the site, lateral spreading is not a design consideration at the site.

#### 3.4.4 Fault Rupture

Based on USGS mapping, the nearest mapped fault is approximately 9.2 kilometers away from the site. As such, fault rupture is not considered a hazard at the site.

## 4.0 DESIGN

### *4.1 GENERAL*

Based on the results of explorations, laboratory testing, and analysis, development is feasible at the site. A detailed discussion of the geotechnical considerations and recommendations for design and construction are discussed in the following sections.

### *4.2 FOUNDATION SUPPORT*

#### 4.2.1 General

We anticipate structural elements associated with the improvements will be lightly loaded and less than 100 kips and 4 kips per foot. Provided our loading estimates are correct and the subgrades are prepared as discussed in the "Construction" section, it is our opinion that structural elements can be supported on conventional spread footings overlying undisturbed native soil or structural fill on top undisturbed native soil.

Up to 3.5 feet of undocumented fill is present at the site. Where encountered beneath footings, undocumented fill should be completely removed to native soil. Upon verification of native soil by a member of our field staff, the over-excavation should be backfilled with compacted crushed rock to the planned footing base. Over-excavation should extend 6 inches beyond the margins of the footings for every foot excavated below the base grade of the footing. Crushed rock should be compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D1557, or until well keyed, as determined by one of our geotechnical staff.

As an alternative to over-excavating and backfilling with gravel, foundations can be supported on undocumented fill, provided the owner is willing to accept the risk of poor foundation performance (excessive settlement). Based on the thickness and composition of the fill, the likelihood of excessive settlement is low.

#### 4.2.2 Dimensions and Capacities

Footings should be established on undisturbed native soil or on structural fill or granular pads underlain by firm, undisturbed native soil. Footings should be proportioned for an allowable bearing pressure of 2,000 psf. This value is a net bearing pressure; the weight of the footing and overlying backfill can be ignored in calculating footing sizes. The recommended allowable bearing pressure applies to the total of dead plus long-term live loads and can be increased by one-third for short-term loads resulting from wind or seismic forces.

Continuous isolated spread footings or circular footings should be at least 24 inches wide or 24 inches in diameter, respectively. The bottom of exterior footings should be at least 18 inches below the lowest adjacent exterior grade. The bottom of interior footings should be established at least 12 inches below the base of the slab. If footings are excavated in the wet season, we recommend they are covered with a minimum of 6 inches of crushed rock shortly after excavation to prevent softening of the subgrade soil.

Total post-construction consolidation settlement for foundations on native soil is expected to be less than 1 inche with differential settlement less than 0.5 inch over a 50-foot span. Foundations constructed on undocumented fill will likely see total settlements of less than 1 inch, however, there is the possibility of select locations with settlement larger than 1 inch.

#### 4.2.3 Resistance to Sliding

Lateral loads on footings can be resisted by passive earth pressure on the sides of the structure and by friction on the base of the footings. Our analysis indicates that the available passive earth pressure for footings confined by native soil and structural fill is 300 pcf, modeled as an equivalent fluid pressure. Typically, the movement required to develop the available passive resistance may be relatively large. Therefore, we recommend using a reduced passive equivalent fluid pressure of 250 pcf. Adjacent floor slabs, pavement, or the upper 12 inch depth of adjacent, unpaved areas should not be considered when calculating passive resistance. In addition, in order to rely on passive resistance, a minimum of 5 feet of horizontal clearance must exist between the face of the footings and any adjacent down slopes.

For footings in contact with native soil, an ultimate coefficient of friction equal to 0.35 may be used when calculating resistance to sliding. For footings in contact with the granular footing pads, an ultimate coefficient of friction equal to 0.60 may be used when calculating resistance to sliding.

#### 4.2.4 Resistance to Sliding

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

### *4.3 SEISMIC DESIGN CRITERIA*

The building will likely be constructed in accordance with the 2022 Oregon Structural Specialty Code which references ASCE-7-16 for design parameters. Based on explorations and geologic mapping at the site, the appropriate seismic site class for design is D. Seismic design parameters in accordance with ASCE-7-16 are provided in Table 2.

**Table 2. ASCE 7-16 Seismic Design Parameters[1]**

| | Short Period ($T_s$ = 0.2 s) | 1 Second Period ($T_1$ = 1.0 s) |
|---|---|---|
| MCE Spectral Acceleration, S | 0.716 g | 0.366g |
| Site Class | D[2] | |
| Site Coefficient, F | $F_a$ = 1.203 | $F_v$ = 1.934 |
| Adjusted Spectral Acceleration, $S_M$ | $S_{MS}$ = 0.878 g | $S_{M1}$ = 0.708 g |
| Design Spectral Response Acceleration, $S_D$ | $S_{DS}$ = 0.586 g | $S_{D1}$ = 0.472 g |

1. The structural engineer should evaluate ASCE-7-16 code requirements and exceptions to determine if these parameters are valid for design.
2. Seismic site class, site coefficients, and spectral acceleration parameters assume that the fundamental period for proposed structures will be less than 0.5 second.

ASCE 7-16 Section 11.4.8 requires a ground motion hazard study in accordance with Section 21.2 for structures on Site Class D sites with $S_1$ greater than or equal to 0.2 g ($S_1$ at the site is 0.349 g). Exception 2 of ASCE 7-16 Section 11.4.8 indicates a ground motion hazard study is not required for structures on Site Class D sites with S1 greater to or equal 0.2 g, provided the value of the seismic response coefficient $C_S$ is determined by Eq. (12.8-2) for values of T≤1.5Ts and taken as equal to 1.5 times the value computed in accordance with either Eq. (12.8-3) for TL≥T>1.5Ts or Eq. (12.8-4) for T>TL. We anticipate the building will meet these requirements, but if Exception 2 is not applicable, a ground motion hazard analysis will be required. We recommend the structural engineer evaluate these requirements and exceptions to determine if the parameters for Site Class D provided in Table 2 can be used for design or if a site-specific seismic hazard evaluation is required.

### *4.4 FLOOR SLABS*

Replacement of portions of slabs is likely as part of improvements. As previously discussed, undocumented fill is present over portions of the site. Due to the variable composition of the fill and the unknown methods of placement and compaction, reliable strength properties for undocumented fill are extremely difficult to predict and there is a small risk for poor performance of floor slabs established directly over undocumented fill and buried topsoil.

To completely eliminate the risk of poor floor slab performance, undocumented fill should be removed, moisture conditioned, and re-compacted or removed and replaced. Because floor slab work includes isolated areas, we anticipate this is not feasible. As an alternative to the re-compaction or replacement of undocumented fill, floor slabs can be constructed on undocumented fill, provided a small risk of distress is accepted.

Provided subgrades are described as recommended in this report and the owner accepts the small risk of poor floor slab performance if undocumented fill is left in place, satisfactory subgrade support for building floor slabs at existing grades supporting loads up to 200 psf is possible. A modulus of subgrade reaction of 100 pci should be used for design of floor slabs.

A minimum 6-inch-thick layer of imported granular material should be placed and compacted over the prepared subgrade to assist as a capillary break. The floor slab base rock should be crushed rock or crushed gravel and sand meeting the requirements outlined in the "Structural Fill" section. The imported granular material should be placed in one lift and compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D1557. Floor slab base rock contaminated with excessive fines (greater than 5 percent by dry weight passing the U.S. Standard No. 200 sieve) should be replaced.

Flooring manufacturers often require vapor barriers to protect flooring and flooring adhesives. Many flooring manufacturers will warrant their product only if a vapor barrier is installed according to their recommendations. Selection and design of an appropriate vapor barrier, if needed, should be based on discussions among members of the design team. We can provide additional information to assist you with your decision.

## 4.5 RETAINING STRUCTURES

### 4.5.1 Assumptions

Our retaining wall design recommendations are based on the following assumptions: (1) the walls are cantilevered walls, (2) the walls are less than 10 feet in height, (3) drainage is provided behind walls, (4) the retained soil has a slope flatter than 4H:1V, and (5) the ground surface at the toe of the wall has an inclination of flatter than 5H:1V. Re-evaluation of our recommendations will be required if the retaining wall design criteria for the project varies from these assumptions.

### 4.5.2 Wall Design Parameters

Permanent retaining structures free to rotate slightly around the base should be designed for active earth pressures using an equivalent fluid unit pressure of 35 pcf. If retaining walls are restrained against rotation during backfilling, they should be designed for an at-rest earth pressure of 55 pcf.

Seismic lateral forces can be calculated using a dynamic force equal to $7H^2$ pounds per linear foot of wall, where H is the wall height. The seismic force should be applied as a distributed load with the centroid located at 0.6H from the wall base. Footings for retaining walls should be designed as recommended for shallow foundations.

The design equivalent fluid pressure should be increased for walls that retain sloping soil. We recommend the above lateral earth pressures be increased using the factors presented in Table 3 when designing walls that retain sloping soil.

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

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

If other surcharges located within a horizontal distance of twice the height of the wall from the back of the wall, we should be contacted to provide recommendations for increased lateral earth pressures.

Foundations for walls can be designed in accordance with Section 4.2, *Foundation Support*.

#### 4.5.3 Wall Drainage and Backfill

The above design parameters have been provided assuming drains will be installed behind walls to prevent hydrostatic pressures from developing. If a drainage system is not installed, our office should be contacted for revised design forces.

Backfill material placed behind the walls and extending a horizontal distance of ½H, where H is the height of the retaining wall, should consist of retaining wall select backfill placed and compacted in conformance with the "Structural Fill" section.

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

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

### *4.6 DRAINAGE*

#### 4.6.1 Temporary

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

#### 4.6.2 Surface

The ground surface at finished pads should be sloped away from their edges at a minimum 2 percent gradient for a distance of at least 5 feet. Roof drainage from the building should be directed into solid, smooth-walled drainage pipes that carry the collected water to the storm drain system.

#### 4.6.3 Subsurface

Perimeter footing drains are recommended where the base of new footings are more than 4 feet below existing grades. Perimeter foundation drains should consist of a filter fabric-wrapped, drain rock-filled trench that extends at least 12 inches below the lowest adjacent grade (i.e., slab subgrade elevation). A perforated pipe should be placed at the base to collect water that gathers in the drain rock. The drain rock and filter fabric should meet specifications outlined in the "Structural Fill" section. Discharge for footing drains should not be tied directly into the stormwater drainage system, unless mechanisms are installed to prevent backflow.

#### 4.6.4 Stormwater Infiltration Systems

Due to the shallow depth to groundwater and subsurface soils above groundwater, onsite infiltration systems are not feasible at the site.

### *4.7 PERMANENT SLOPES*

Permanent cut and fill slopes should not exceed 2H:1V. Access roads and pavement should be located at least 5 feet from the top of cut and fill slopes. The setback should be increased to 10 feet for buildings. The slopes should be planted with appropriate vegetation to provide protection against erosion as soon as possible after grading. Surface water runoff should be collected and directed away from slopes to prevent water from running down the face of the slope.

### *4.8 PAVEMENT AND ROADWAYS*

#### 4.8.1 General

New asphalt concrete and portland cement concrete (PCC) pavements will be required in select areas of the site. Pavement should be installed on subgrade prepared in conformance with the "Site Preparation" and "Structural Fill" sections. As previously discussed, undocumented fill is present over portions of the site. Due to the variable composition of the fill and the unknown methods of placement and compaction, reliable strength properties for undocumented fill are difficult to predict and there is a risk for poor performance of pavements established directly over undocumented fill. Our pavement recommendations are based on the following assumptions:

- The top 12 inches of soil subgrade is compacted to at least 92 percent of its maximum dry density, as determined by ASTM D1557, or until proof rolling with heavy equipment indicates that is it firm and unyielding.
- A resilient modulus of 4,000 psi for the subgrade soil and 20,000 psi is assumed for the base rock.
- The design manual provided for the project specifies pavement recommendations based on a design life of 20 years.
- Initial and terminal serviceability indices of 4.2 and 2.5, respectively.
- Reliability of 85 percent and standard deviation of 0.45.

- Fire access will consist of an imposed fire apparatus load of 75,000 pounds on an infrequent basis.
- One-way traffic.
- No growth.

Traffic demand was unknown at the time this report was prepared. Based on our experience with similar developments we have assumed truck traffic will consist of 30 percent FHWA Class 5 vehicles (2-axle, single unit), 20 percent FHWA Class 6 vehicles (3-axle, single unit), 40 percent FHWA Class 8 vehicles (2-axle tractor, 1 axle trailer; 2 axle tractor, 2 axle trailer; 3 axle tractor, 1 axle trailer), and 10 percent FHWA Class 9 vehicles (3 axle tractor, 2 axle trailer). If our assumptions are incorrect, we should be contacted to revise our pavement recommendations.

### 4.8.2 AC Pavement

Our AC pavement design recommendations are presented in Table 4.

**Table 4. 20-Year Standard AC Pavement Sections on Existing Subgrade**

| Pavement Use | Trucks per Day | ESALs | AC Thickness[1] (inches) | Aggregate Base Thickness[1] (inches) |
|---|---|---|---|---|
| Automobile-Only Drive Aisles | 0 | NA | 3.0 | 10.0 |
| Automobile Parking | 0 | NA | 2.5 | 8.0 |
| Truck Areas | 25 | 118,000 | 4.5 | 13.5 |
| | 50 | 236,000 | 4.5 | 15.5 |
| | 75 | 354,000 | 5.0 | 15.5 |
| | 100 | 472,000 | 5.0 | 17.0 |
| | 125 | 590,000 | 5.5 | 16.5 |
| | 150 | 708,000 | 5.5 | 17.5 |
| | 175 | 826,000 | 5.5 | 18.5 |
| | 200 | 944,000 | 5.5 | 19.5 |

1. One-way truck ADT
2. All thicknesses are intended to be the minimum acceptable.

### 4.8.3 PCC Pavement

PCC pavement will be required for new loading docks at the site. Based on correspondence with Mackenzie, the number of trucks at the loading docks will be less than 25 per day. The PCC and aggregate base should meet the requirements outlined in the "Materials" section. We recommend the following PCC section in Table 5 for standard subgrades.

**Table 5. 20-Year PCC Standard Pavement Sections**

| Trucks per Day[1] | PCC (inches) | Base Rock (inches) | Maximum Joint Spacing (feet) |
|---|---|---|---|
| Up to 25 | 6.5 | 6.0 | 12 |

1. One-way truck ADT
2. All thicknesses are intended to be the minimum acceptable.

### 4.8.5 Construction Considerations

All thicknesses are intended to be the minimum acceptable. Design of the recommended pavement section assumes that construction will be completed during an extended period of dry weather. Construction traffic should be limited to non-building, unpaved portions of the site or haul roads. Construction traffic should not be allowed on new pavement. If construction traffic is to be allowed on newly constructed road sections, an allowance for this additional traffic will need to be made in the design pavement section.

The aggregate base does not account for construction traffic, and haul roads and staging areas should be used as described in the "Construction" section.

## 5.0 CONSTRUCTION CONSIDERATIONS

### *5.1 SITE PREPARATION*

### 5.1.1 General

Site grading activities should be performed in accordance with requirements specified in the 2018 International Building Code (IBC), Chapter 18 and Appendix J, with exceptions noted in the text herein. Site preparation and grading activities should be observed and documented by Columbia West.

### 5.1.2 Demolition

Demolition includes removal of structural features in improvement areas. Abandoned foundations and utilities, if present, will need to be removed and the resulting excavations backfilled. Utility lines should be completely removed or, with prior approval, grouted full if left in place. Excavations left from demolition and removal of existing structures should be backfilled with compacted structural fill in accordance with recommendations in Section 5.5, *Materials*.

### 5.1.4 Undocumented Fill Considerations

#### 5.1.4.1 General

Up to 3.5 feet of fill was observed in portions of the site. Refer to Figure 2 for the locations and depths of the fill. Due to the variable composition of the fill and the unknown methods of placement and compaction, reliable strength properties for undocumented fill are extremely difficult to predict.

#### 5.1.4.2 Foundations

Undocumented fill and buried topsoil should be removed from under new foundations and footings should be supported on crushed rock as discussed in the "Shallow Foundations" section.

Foundations for buildings may also be supported on existing fill, provided they are designed and constructed as discussed in the "Shallow Foundations" section and the owner is willing to accept a potential for settlement that exceeds 1 inch of total settlement and 0.5 inch of differential settlement.

#### 5.1.4.3 Floor Slabs and Pavement Areas

There is a small risk for poor performance of floor slabs and pavement established directly over undocumented fill. To eliminate the risk of poor performance, undocumented fill should be removed, moisture conditioned, and re-compacted or removed and replaced after site stripping and cuts.

Floor slabs and pavements can be constructed on undocumented fill, provided a small risk of distress is accepted (minor floor slab cracking, localized "bird baths" in pavement areas, and irregular settlement in gravel areas) and they are evaluated as described in the "Subgrade Evaluation" section. To reduce the potential for cracking, floor slabs can incorporate additional reinforcement to span areas where differential settlement occurs. This does not completely eliminate the risk for settlement; however, in our experience, it is more cost effective than removal and re-compaction (or replacement with imported structural fill).

### 5.1.5 Subgrade Evaluation

Upon completion of stripping and prior to the placement of structural fill or pavement improvements, exposed subgrade soil should be evaluated by proof rolling with a fully loaded dump truck or similar heavy, rubber tire construction equipment. When the subgrade is too wet for proof rolling or space does not allow, a foundation probe may be used to identify areas of soft, loose, or unsuitable soil. Subgrade evaluation should be performed by Columbia West. If soft or yielding subgrade areas are identified during evaluation, we recommend the subgrade be over-excavated and backfilled with compacted imported granular fill.

## *5.2 CONSTRUCTION TRAFFIC AND STAGING*

The fine-grained soil below the asphalt concrete, portland cement, and gravel at the site is easily disturbed. We recommend existing asphalt concrete, Portland cement, and gravel stay in place during construction. If fine-grained soil is exposed to equipment traffic, site preparation, utility trench work, and roadway excavation can create extensive soft areas and significant repair costs can result. Earthwork planning, regardless of the time of year, should include considerations for minimizing subgrade disturbance.

If construction occurs during or extends into the wet season, or if the moisture content of the surficial soil is more than a couple percentage points above optimum, and fine-grained soil is exposed, granular haul roads and staging areas will be necessary for support of construction traffic. The amount of staging and haul road areas, as well as the required thickness of granular material, will vary with the contractor's sequencing of a project and type/frequency of construction equipment and should, therefore, be the responsibility of the contractor. Based on our experience, between 12 and 18 inches of imported granular material is generally required in staging areas and between 18 and 24 inches in haul roads areas. The contractor should also be responsible for selecting the type of material or construction of haul roads and staging areas. A geotextile fabric can be placed as a barrier between the subgrade and imported granular material in areas of repeated construction traffic to help prevent silt migration into the base rock. The imported granular material, stabilization material, and geotextile fabric should meet the specifications in the "Materials" section.

As an alternative to thickened crushed rock sections, haul roads and utility work zones may be constructed using cement-amended subgrades overlain by a crushed rock wearing surface. Based on the scope of the project we do not anticipate cement-amending will be cost effective. If cement-amending is considered, Columbia West can provide recommendations at a later date.

## *5.3 EXCAVATION AND TEMPORARY SLOPES*

### 5.3.1 General

Based on observations during explorations and review of water well logs, groundwater could be within several feet of the ground surface during the wet season. The near surface soil at the site consists of a variable mixture of fill, silt, sand, and gravel.

Temporary excavation sidewalls in silt should stand vertical to a depth of approximately 4 feet, provided groundwater seepage does not occur in the sidewalls. Caving should be expected at all depths where fill, sand, and gravel are present. Below 4 feet where silt is present or at all depths in fill, sand, and gravel, open excavation techniques may be used to excavate trenches with depths between 4 and 8 feet, provided the walls of the excavation are cut at a slope of 1H:1V or flatter and groundwater seepage does not occur. Excavations should be flattened to 1.5H:1V or flatter if excessive sloughing occurs.

Given the relatively shallow groundwater, temporary shoring and dewatering will likely be required. Use of approved temporary shoring is recommended where the slopes cannot be cut back, within the influence area of structural elements, and for cuts below the water table. The influence area can be defined as a 1H:1V slope extending down from a 5-foot setback from the edge of a foundation element. A wide variety of shoring and dewatering systems are available. Consequently, we recommend the contractor be responsible for selecting the appropriate shoring and dewatering systems.

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

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

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

### 5.3.2 Cobbles, Boulders, and Construction Debris

While not directly observed in explorations, we anticipate that cobbles and small boulders could also be in the native gravel at the site. In addition, construction debris, including large concrete chunks, could also be present within the undocumented fill at the site. Construction considerations associated with cobbles, boulders, and construction debris include the following:

- Excavations can become difficult, if not impossible, with conventional equipment.
- Excavation volumes for utility trenches may be greater than anticipated due to sloughing and the need to remove oversized material.
- We recommend that project bid documents include a contingency for boulder removal, as well as the associated increased trench volumes for backfilling.

## *5.4 CONSTRUCTION DEWATERING*

### 5.4.1 General

Groundwater was observed between 5 and 6.5 feet during explorations. During extended periods of rain, groundwater levels could even be closer to the ground surface. We expect groundwater will be encountered in trench excavations and possibly in foundation excavations. Temporary and permanent dewatering systems may be required.

### 5.4.2 Construction Dewatering

The contractor should be responsible for temporary drainage of surface water, perched water, and groundwater as necessary to prevent standing water and/or erosion at the working surface. Because of the instability of saturated, low plasticity silt, sloughing and “running” conditions can occur if the excavation extends below groundwater seepage levels. Positive control of groundwater will be required to maintain stable trench sides and base. The proposed dewatering plan should be capable of maintaining groundwater levels at least 2 feet below the base of the trench excavation (including the depth required for trench bedding and stabilization material). In addition to safety considerations, running soil, caving, or other loss of ground will increase backfill volumes and can result in damage to adjacent structures or utilities.

Flow rates for dewatering are likely to vary depending on location, soil type, and the season in which the excavation occurs. Dewatering systems should be capable of adapting to variable flows. Because of the tendency of saturated, low plasticity silt with sand to “run,” we recommend dewatering wells or well points be considered if trench excavations extend below groundwater levels. Tight-joint driven sheets in conjunction with a scaled-down dewatering program can also be an effective way to control groundwater seepage, provided the sheets are driven deep enough to control heaving conditions at the base of the excavation.

Trench dewatering will be required to maintain dry working conditions if the invert elevations of the proposed utilities encounter groundwater. Pumping from sumps located within the trench may result in excessive sloughing, caving, or running conditions, and dewatering by well points may be required. If groundwater is present at the base of utility excavations, we recommend placing 1.5 to 2 feet of stabilization material at the base of the excavation. The use of a subgrade geotextile fabric may reduce the amount of stabilization material required. The actual thickness should be based on field observations during construction. Trench stabilization material and the subgrade geotextile fabric should meet the requirements described in the “Materials” section. Trench stabilization material should be placed in one lift and compacted until well keyed.

While we have described certain approaches to the excavation dewatering, it is the contractor's responsibility to select the dewatering methods.

#### 5.4.3 Permanent Dewatering

Permanent dewatering systems are currently not anticipated; however, we request the opportunity to review the final grading plan.

### *5.5 MATERIALS*

#### 5.5.1 Structural Fill

##### 5.5.1.1 General

Areas proposed for fill placement should be appropriately prepared as described in Section 5.1, Site Preparation and Grading. Engineered fill placement should be observed by Columbia West. Compaction of engineered structural fill should be verified proof rolling or nuclear gauge field compaction testing performed in accordance with ASTM D1557. Field compaction testing should be performed for each vertical foot of engineered fill placed.

Various materials may be acceptable for use as structural fill. Structural fill should be free of organic material or other unsuitable material and meet specifications provided in the following sections. Representative samples of proposed engineered structural fill should be submitted for laboratory analysis and approval by Columbia West prior to placement. All structural fill should be free of organics and have a particle size less than 6 inches.

##### 5.5.1.2 Onsite Soil

The near surface soil at the site consists of variable mixture of fill, silt, sand, and gravel. The soils are the site are suitable for use as structural fill if adequately dried or moisture conditioned to achieve recommended compaction specifications. Based on laboratory testing and shallow groundwater, we anticipate that the moisture content of the soil will generally be above the optimum moisture content required to meet compaction requirements and drying of the soil will be necessary. Accordingly, extended dry weather will be required to adequately condition and place the soil as structural fill. It will be difficult, if not impossible, to adequately compact on-site soil during the rainy season or during prolonged periods of rainfall.

Onsite soil used as structural fill should be placed in loose lifts not exceeding 8 inches in depth and compacted using standard conventional compaction equipment. The soil moisture content should be within a few percentage points of optimum conditions. The soil should be compacted to at least 95 percent of maximum dry density as determined by the modified Proctor moisture-density relationship test (ASTM D1557).

Onsite soil will likely expand during excavation and transport and consolidate during compaction. Development of site-specific expansion and consolidation factors is beyond the scope of this investigation. We can provide site-specific factors upon request.

#### 5.5.1.3 Imported Granular Material

Imported granular material should consist of pit- or quarry-run rock, crushed rock, or crushed gravel and sand. Imported granular material should be placed in loose lifts not exceeding 12 inches in depth and compacted to at least 95 percent of maximum dry density as determined by the modified Proctor moisture-density relationship test (ASTM D1557). During wet-weather conditions or where wet subgrade conditions are present, the initial loose lift of granular fill should be approximately 18 inches thick and should be compacted with a smooth-drum roller operating in static mode.

#### 5.5.1.4 Stabilization Material

Stabilization material should consist of durable, 4- or 6-inch-minus pit- or quarry-run rock, crushed rock, or crushed gravel and sand that is free of organics and other deleterious material. The material should have a maximum particle size of 6 inches with less than 5 percent by dry weight passing the U.S. Standard No. 4 sieve. The material should have at least two mechanically-fractured faces.

Stabilization material should be placed in loose lifts between 12 and 24 inches thick and be compacted to a firm, unyielding condition. Equipment with vibratory action should not be used when compacting stabilization material over wet, fine-textured soils. If stabilization material is used to stabilize soft subgrade below pavement or construction haul roads, a subgrade geotextile should be placed as a separation barrier between the soil subgrade and the stabilization material.

#### 5.5.1.5 Trench Backfill

Trench backfill placed beneath, adjacent to, and for at least 12 inches above utility lines (i.e., the pipe zone) should consist of durable, well-graded granular material with a maximum particle size of 1½ inches, should have less than 7 percent fines by dry weight, and should have at least two mechanically fractured faces. The pipe zone backfill should be compacted to at least 90 percent of the maximum dry density, as determined by ASTM D1557, or as required by the pipe manufacturer or local building department.

Within roadway alignments, the remainder of the trench backfill up to the subgrade elevation should consist of durable, well-graded granular material with a maximum particle size of 2½ inches, should have less than 7 percent fines by dry weight, and should have at least two mechanically fractured faces. This material should be compacted to at least 92 percent of the maximum dry density, as determined by ASTM D1557, or as required by the pipe manufacturer or local building department. The upper 3 feet of the trench backfill should be compacted to at least 95 percent of the maximum dry density, as determined by ASTM D1557.

Outside of structural areas, trench backfill placed above the pipe zone should be compacted to at least 90 percent of the maximum dry density as determined by the modified Proctor moisture-density relationship test (ASTM D1557), or as required by the local jurisdictional agency or pipe manufacturer.

#### 5.5.1.6 Retaining Wall Backfill

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

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

#### 5.5.1.7 Retaining Wall Leveling Pad

Crushed aggregate used as a leveling pad for retaining wall footings should consist of ¾ - 1 ¼ inch minus crushed rock and have less than 7 percent fines by dry weight. The leveling pad material should be compacted to at least 95 percent of the maximum dry density as determined by ASTM D1557.

#### 5.5.1.8 Floor Slab and Pavement Base Aggregate

Imported granular material used as base rock for building floor slabs should consist of ¾- or 1½-inch-minus material (depending on the application). In addition, the aggregate should have less than 5 percent fines by dry weight and have at least two mechanically fractured faces. The aggregate base should be compacted to not less than 95 percent of the maximum dry density, as determined by ASTM D1557.

#### 5.5.1.9 Drain Rock

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

### 5.5.2 Geotextile Fabric

#### 5.5.2.1 Subgrade Geotextile

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

#### 5.5.2.2 Drainage Geotextile

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

#### 5.5.3 Pavement

##### 5.5.3.1 Asphalt Concrete Paving

The AC should be Level 2, ½-inch, dense ACP in the parking areas and Level 3, ½-inch, dense ACP in the truck areas according to OSSC 00744 (Asphalt Concrete Pavement). The AC should be compacted to 91 percent of the theoretical maximum density of the mix, as determined by AASHTO T 209. The minimum and maximum lift thicknesses are 2 and 3 inches, respectively, for ½-inch ACP. Asphalt binder should be performance graded and conform to PG 64-22. The binder grade should be adjusted depending on the aggregate gradation and amount of recycled asphalt pavement and/or recycled asphalt shingles in the contractor's mix design submittal.

##### 5.5.3.2 Cold Weather Paving Considerations

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

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

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

##### 5.5.3.3 PCC

PCC should be Class 4000, 1½-inch paving concrete according to OSSC 02001 (Concrete) with a minimum 28-day flexural strength of 600 psi. The length-to-width ratio for any panel should be at least 0.80 and should not exceed 1.25. Joints in truck bays should have a maximum 14-foot joint spacing. Reinforcing and specifications should be provided by the site civil and structural engineering team.

### *5.6 EROSION CONTROL*

The site soil is susceptible to erosion; therefore, erosion control measures should be carefully planned and in place before construction begins. Surface water runoff should be collected and directed away from slopes to prevent water from running down the slope face. Erosion control measures (such as straw bales, sediment fences, and temporary detention and settling basins) should be used in accordance with local and state ordinances and the project 1200C permit.

♦ ♦ ♦

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

Sincerely,
Columbia West

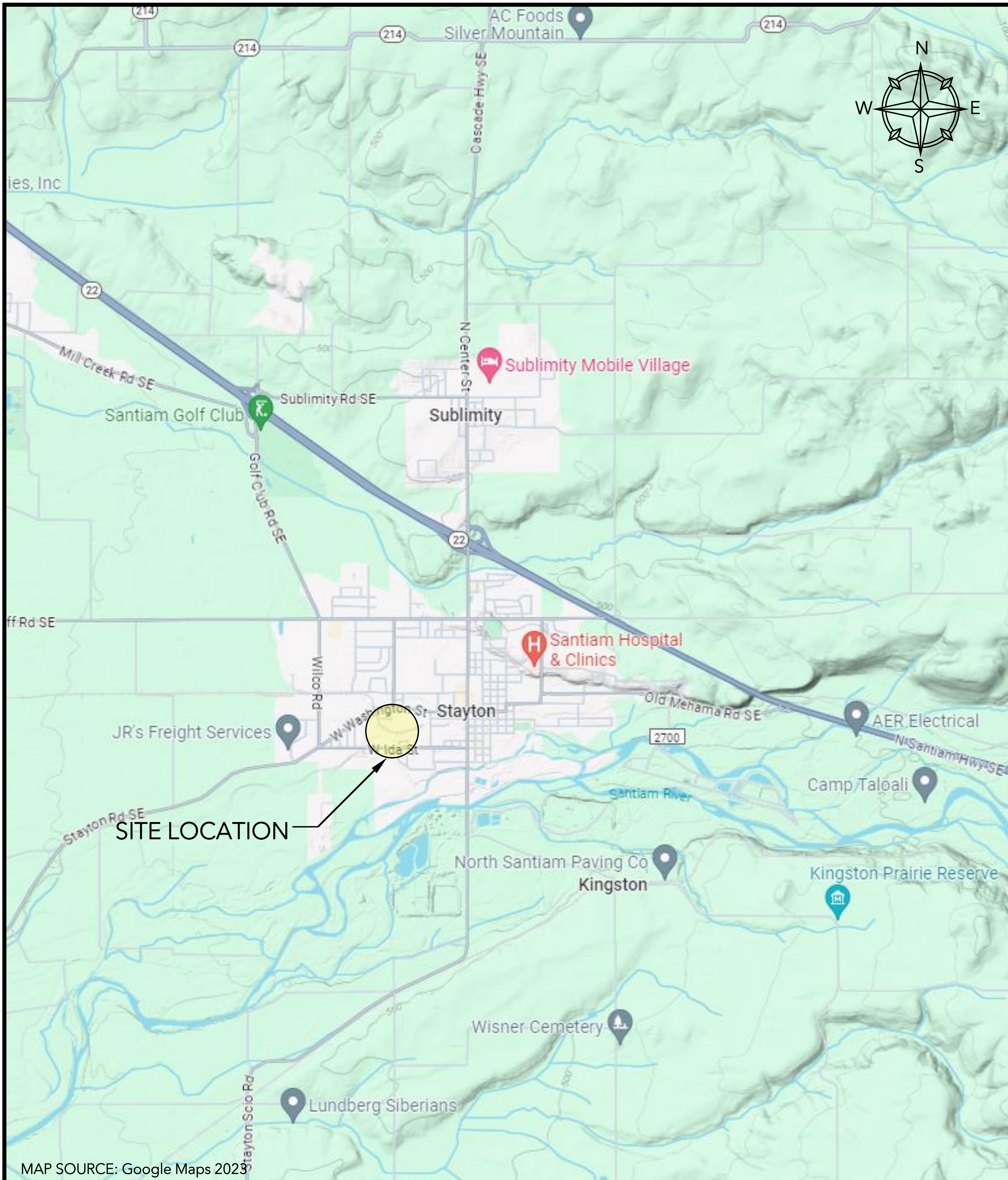
Nick Paveglio, P.E.
Principal Engineer

## REFERENCES

ASCE, 2016. Minimum Design Loads for Buildings and Other Structures. ASCE Standard ASCE/SEI 7-16. American Society of Civil Engineers.

O'Connor, Jim E., Andrei Sarna-Wojcicki, Karl C. Wozniak, Danial J. Polette, and Robert J. Fick. Geologic Map of Quaternary Units in the Willamette Valley, Oregon, 2001.

# FIGURES

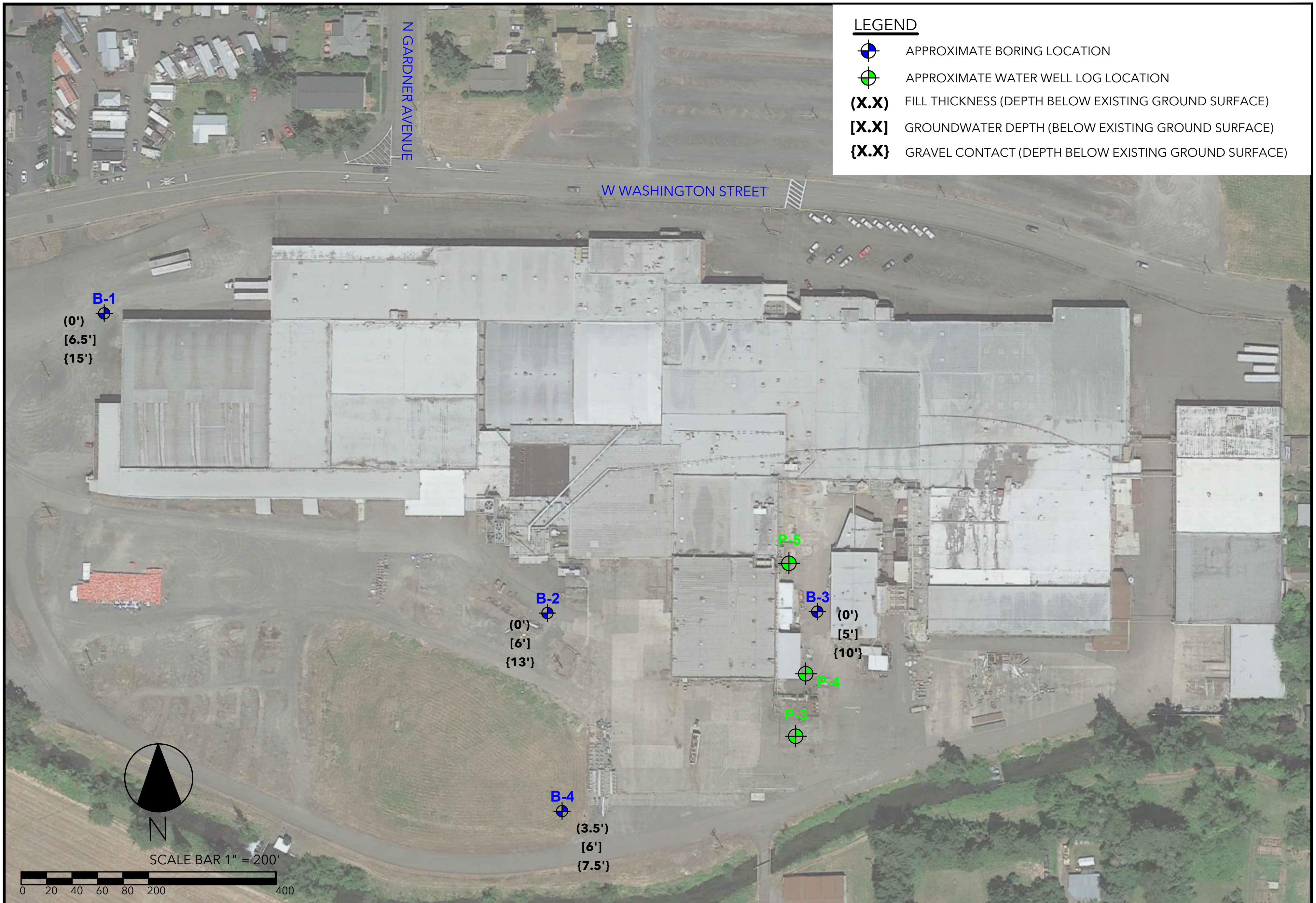



Geotechnical ▪ Environmental ▪ Special Inspections
Columbia West Engineering, Inc

Job No: IRG-1-01-1
Date: 11/10/2023
Drawn: SSC
Checked: NNP

SITE LOCATION MAP
SANTIAM INDUSTRIAL CENTER
STAYTON, OREGON

FIGURE
1




Geotechnical ▪ Environmental ▪ Special Inspections

**Columbia West** Engineering, Inc

Job No: IRG-1-01-1
Date: 11/10/23
Drawn: SSC
Checked: NNP

EXPLORATION LOCATION MAP

SANTIAM INDUSTRIAL CENTER

NOTES:
1. APPROXIMATE SITE LOCATION: 930 W WASHINGTON STREET, STAYTON, OREGON 97383.
2. SITE CONSISTS OF EASTERN PORTION OF MAP TAX LOT NOS. 091W10CB02400A7, 091W10CB02400A8, AND 091W10CB02400 TOTALING APPROXIMATELY 33 ACRES.
3. AERIAL PHOTO SOURCED FROM GOOGLE EARTH.
4. EXPLORATION LOCATIONS ARE APPROXIMATE AND NOT SURVEYED.
5. BORINGS BACKFILLED WITH BENTONITE ON NOVEMBER 9, 2023. (B-2 AND B-3 CAPPED WITH ASPHALT)

FIGURE
2

# APPENDIX A
# SUBSURFACE EXPLORATIONS

## *GENERAL*

We explored subsurface conditions at the site by drilling four borings (B-1 through B-4) to depths between approximately 14 and 17 feet BGS. The boring locations are shown on Figure 2. The logs are presented in this appendix.

Drilling services were provided by Western States Soil Conservation, Inc. of Hubbard, Oregon. The explorations were conducted with a drill rig using mud-rotary drilling techniques on November 9, 2023. The borings were logged on a full time basis by Columbia West personnel. The locations were determined in the field by pacing or measuring from existing site features. This information should be considered accurate only to the degree implied by the methods used.

## *SOIL SAMPLING*

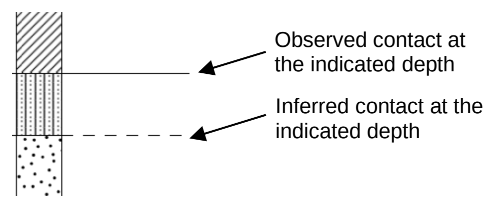
Soil samples were collected from the borings by conducting SPTs in general conformance with ASTM D1586. The sampler was driven with a 140-pound, automatic-trip hammer free falling 30 inches. The number of blows required to drive the sampler 1 foot, or as otherwise indicated, into the soil is shown adjacent to the sample symbols on the exploration log. Disturbed samples were collected from the split barrel for subsequent classification and index testing. Sampling methods and intervals are shown on the exploration log.

The average efficiency of the automatic SPT hammer used by Western States Soil Conservation, Inc. was 85.6 percent. The calibration testing results are presented at the end of this appendix.

## *SOIL CLASSIFICATION*

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

# TABLE A-1: EXPLORATION LEGEND

<table>
<tr><th>Symbol</th><th colspan="2">Description</th></tr>
<tr><td>SPT</td><td colspan="2">Sample obtained from the indicated depth in general accordance with ASTM D1586, Standard Penetration Test and Split-Barrel Sampling of Soils</td></tr>
<tr><td>SHELBY</td><td colspan="2">Sample obtained from the indicated depth using thin-wall Shelby tube in general accordance with ASTM D1587, Thin-Walled Tube Sampling of Fine-Grained Soils</td></tr>
<tr><td>D&M 300</td><td colspan="2">Sample obtained from the indicated depth using Dames & Moore sampler and 300-pound hammer or pushed</td></tr>
<tr><td>D&M 140</td><td colspan="2">Sample obtained from the indicated depth using Dames & Moore sampler and 140-pound hammer or pushed</td></tr>
<tr><td>CSS</td><td colspan="2">Sample obtained from the indicated depth using 3-inch-outer-diameter California split-spoon sampler and 140-pound hammer</td></tr>
<tr><td>GRAB</td><td>Grab sample obtained from the indicated depth</td><td rowspan="3"><u>Graphical Log of Subsurface Lithology</u><br><br></td></tr>
<tr><td>CORE</td><td>Rock core interval at the indicated depth</td></tr>
<tr><td></td><td>Water level observed during exploration</td></tr>
</table>

| Geotechnical Acronyms | | | |
|---|---|---|---|
| AASHTO | American Association of State Highway and Transportation Officials | P | Push Sample |
| ASTM | American Society for Testing and Materials | PP | Pocket Penetrometer |
| ATT | Atterberg Limits | PSF | Pounds Per Square Foot |
| BGS | Below Ground Surface | P200 | Percent Passing No. 200 Sieve |
| CBR | California Bearing Ratio | RES | Resilient Modulus |
| CON | Consolidation Test | SIEV | Sieve Analysis |
| DCPT | Dynamic Cone Penetration Test | SPT | Standard Penetration Test |
| DD | Dry Density | TS | Torvane Shear |
| DS | Direct Shear | UC | Unconfined Compressive Strength |
| HYD | Hydrometer | UU | Unconsolidated Undrained Triaxial Test |
| IR | Infiltration Rate | USCS | United Soil Classification System |
| MC | Moisture Content | VS | Vane Shear |
| MD | Moisture-Density Relationship | WD | Wet Density |
| OC | Organic Content | | |

# TABLE A-2: SOIL CLASSIFICATION SYSTEM

## Particle-Size Classification

| COMPONENT | ASTM / USCS | | AASHTO | |
|---|---|---|---|---|
| | size range | sieve size range | size range | sieve size range |
| Boulders | Greater than 300 mm | Greater than 12 inches | - | - |
| Cobbles | 75 mm to 300 mm | 3 inches to 12 inches | Greater than 75 mm | Greater than 3 inches |
| Gravel | 75 mm to 4.75 mm | 3 inches to No. 4 sieve | 75 mm to 2.00 mm | 3 inches to No. 10 sieve |
| Coarse | 75 mm to 19.0 mm | 3 inches to 3/4-inch sieve | - | - |
| Fine | 19.0 mm to 4.75 mm | 3/4-inch to No. 4 sieve | - | - |
| Sand | 4.75 mm to 0.075 mm | No. 4 to No. 200 sieve | 2.00 mm to 0.075 mm | No. 10 to No. 200 sieve |
| Coarse | 4.75 mm to 2.00 mm | No. 4 to No. 10 sieve | 2.00 mm to 0.425 mm | No. 10 to No. 40 sieve |
| Medium | 2.00 mm to 0.425 mm | No. 10 to No. 40 sieve | - | - |
| Fine | 0.425 mm to 0.075 mm | No. 40 to No. 200 sieve | 0.425 mm to 0.075 mm | No. 40 to No. 200 sieve |
| Fines (Silt and Clay) | Less than 0.075 mm | Passing No. 200 sieve | Less than 0.075 mm | Passing No. 200 sieve |

## Consistency for Cohesive Soil

| CONSISTENCY | SPT N-VALUE (BLOWS PER FOOT) | D&M N-VALUE (BLOWS PER FOOT) | POCKET PENETROMETER (UNCONFINED COMPRESSIVE STRENGTH, tsf) |
|---|---|---|---|
| Very Soft | Less than 2 | Less than 3 | Less than 0.25 |
| Soft | 2 to 4 | 3 to 6 | 0.25 to 0.50 |
| Medium Stiff | 4 to 8 | 6 to 12 | 0.50 to 1.0 |
| Stiff | 8 to 15 | 12 to 25 | 1.0 to 2.0 |
| Very Stiff | 15 to 30 | 25 to 65 | 2.0 to 4.0 |
| Hard | 30 to 60 | 65 to 145 | Greater than 4.0 |
| Very Hard | Greater than 60 | Greater than 145 | - |

## Relative Density for Granular Soil

| RELATIVE DENSITY | SPT N-VALUE (BLOWS PER FOOT) | D&M N-VALUE (BLOWS PER FOOT) |
|---|---|---|
| Very Loose | 0 to 4 | 0 to 11 |
| Loose | 4 to 10 | 11 to 26 |
| Medium Dense | 10 to 30 | 26 to 74 |
| Dense | 30 to 50 | 74 to 120 |
| Very Dense | Greater than 50 | Greater than 120 |

## Moisture Designations

| TERM | FIELD IDENTIFICATION |
|---|---|
| Dry | No moisture. Dusty or dry. |
| Damp | Some moisture. Cohesive soils are usually below plastic limit and are moldable. |
| Moist | Grains appear darkened, but no visible water is present. Cohesive soils will clump. Sand will bulk. Soils are often at or near plastic limit. |
| Wet | Visible water on larger grains. Sand and silt exhibit dilatancy. Cohesive soil can be readily remolded. Soil leaves wetness on the hand when squeezed. Soil is much wetter than optimum moisture content and is above plastic limit. |

## Additional Constituents

| Percent | Silt and Clay In: | | Percent | Sand and Gravel In: | |
|---|---|---|---|---|---|
| | Fine-Grained Soil | Coarse-Grained Soil | | Fine-Grained Soil | Coarse-Grained Soil |
| < 5 | trace | trace | < 5 | trace | trace |
| 5 – 12 | minor | with | 5 – 15 | minor | minor |
| > 12 | some | silty/clayey | 15 – 30 | with | with |
| | | | > 30 | sandy/gravelly | with (approx. percentage) |

# AASHTO SOIL CLASSIFICATION SYSTEM

**TABLE 1. Classification of Soils and Soil-Aggregate Mixtures**

| General Classification | Granular Materials (35 Percent or Less Passing .075 mm) | | | Silt-Clay Materials (More than 35 Percent Passing 0.075) | | | |
|---|---|---|---|---|---|---|---|
| Group Classification | A-1 | A-3 | A-2 | A-4 | A-5 | A-6 | A-7 |
| Sieve analysis, percent passing: | | | | | | | |
| 2.00 mm (No. 10) | - | - | - | | | | |
| 0.425 mm (No. 40) | 50 max | 51 min | - | - | - | - | - |
| 0.075 mm (No. 200) | 25 max | 10 max | 35 max | 36 min | 36 min | 36 min | 36 min |
| Characteristics of fraction passing 0.425 mm (No. 40) | | | | | | | |
| Liquid limit | | | | 40 max | 41 min | 40 max | 41 min |
| Plasticity index | 6 max | N.P. | | 10 max | 10 max | 11 min | 11 min |
| General rating as subgrade | Excellent to good | | | Fair to poor | | | |

Note: The placing of A-3 before A-2 is necessary in the "left to right elimination process" and does not indicate superiority of A-3 over A-2.

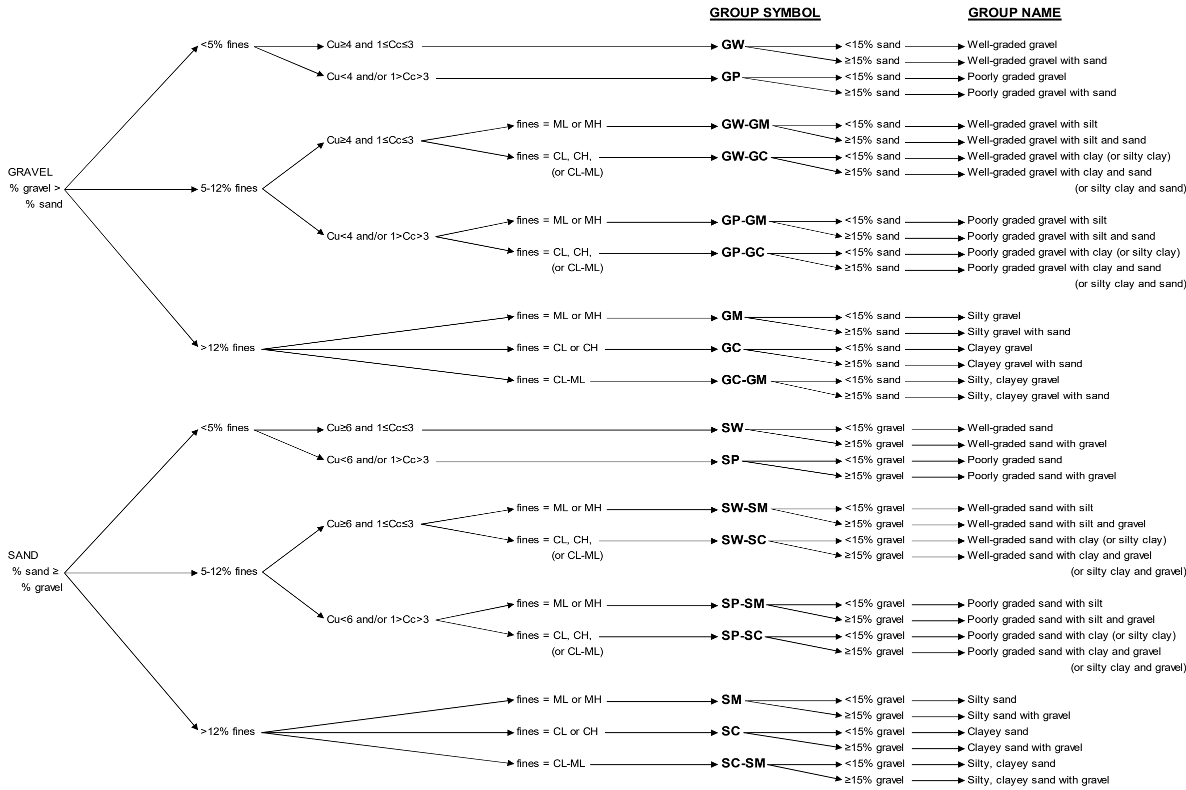
**TABLE 2. Classification of Soils and Soil-Aggregate Mixtures**

| General Classification | Granular Materials (35 Percent or Less Passing 0.075 mm) | | | | | | | Silt-Clay Materials (More than 35 Percent Passing 0.075 mm) | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | A-1 | | | A-2 | | | | | | | A-7 |
| Group Classification | A-1-a | A-1-b | A-3 | A-2-4 | A-2-5 | A-2-6 | A-2-7 | A-4 | A-5 | A-6 | A-7-5, A-7-6 |
| Sieve analysis, percent passing: | | | | | | | | | | | |
| 2.00 mm (No. 10) | 50 max | - | - | - | - | - | - | - | - | - | - |
| 0.425 mm (No. 40) | 30 max | 50 max | 51 min | - | - | - | - | - | - | - | - |
| 0.075 mm (No. 200) | 15 max | 25 max | 10 max | 35 max | 35 max | 35 max | 35 max | 36 min | 36 min | 36 min | 36 min |
| Characteristics of fraction passing 0.425 mm (No. 40) | | | | | | | | | | | |
| Liquid limit | | | | 40 max | 41 min | 40 max | 41 min | 40 max | 41 min | 40 max | 41 min |
| Plasticity index | 6 max | | N.P. | 10 max | 10 max | 11 min | 11 min | 10 max | 10 max | 11 min | 11min |
| Usual types of significant constituent materials | Stone fragments, gravel and sand | | Fine sand | Silty or clayey gravel and sand | | | | Silty soils | | Clayey soils | |
| General ratings as subgrade | Excellent to Good | | | | | | | Fair to poor | | | |

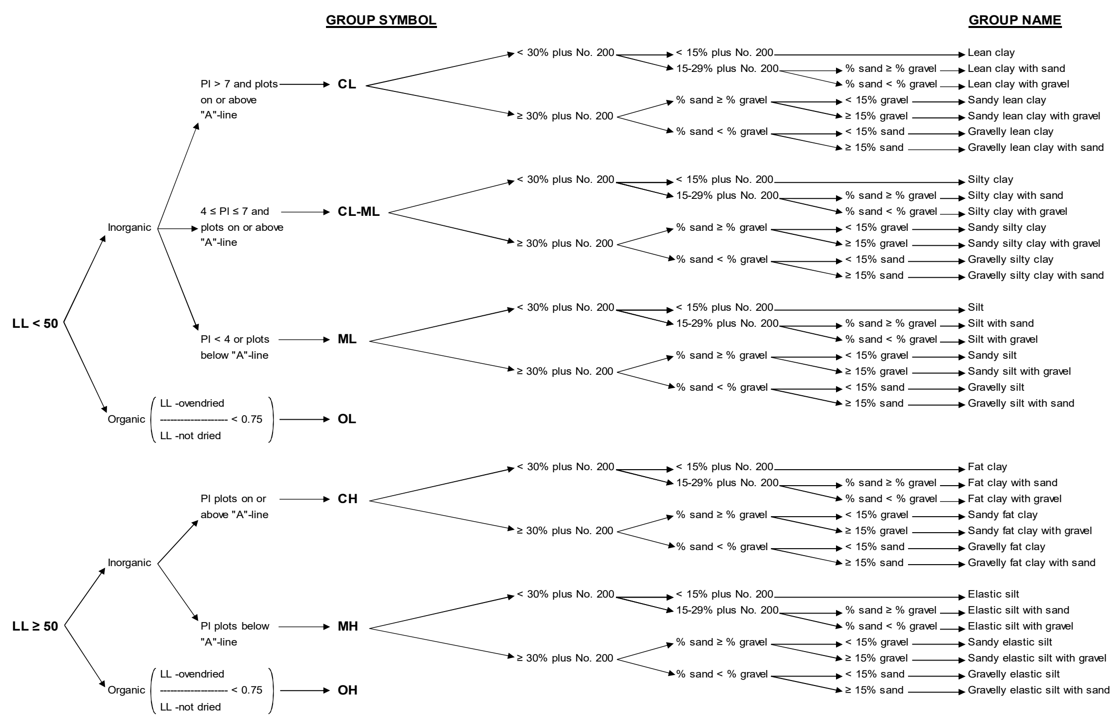
Note: Plasticity index of A-7-5 subgroup is equal to or less than LL minus 30. Plasticity index of A-7-6 subgroup is greater than LL minus 30 (see Figure 2).

AASHTO = American Association of State Highway and Transportation Officials

# UNIFIED SOIL CLASSIFICATION SYSTEM




Flow Chart for Classifying Coarse-Grained Soils (More Than 50% Retained on No. 200 Sieve)




Flow Chart for Classifying Fine-Grained Soil (50% or More Passes No. 200 Sieve)

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# BORING LOG

| PROJECT NAME | CLIENT | | PROJECT NO. | BORING NO. |
|---|---|---|---|---|
| Santiam Industrial Center | Santiam Industrial Center | | IRG-1-01-1 | B-1 |
| **PROJECT LOCATION** | **DRILLING CONTRACTOR** | **DRILL RIG** | **ENGINEER** | **PAGE NO.** |
| Stayton, Oregon | Western States | Tracked Rig #2 | SSC | 1 of 1 |
| **BORING LOCATION** | **DRILLING METHOD** | **SAMPLING METHOD** | **START DATE** | **START TIME** |
| See Figure 2 | Hollow Stem Auger | SPT (automatic) | 11/9/23 | 0905 |
| **REMARKS** | | **GROUNDWATER DEPTH** | **FINISH DATE** | **FINISH TIME** |
| None | | 6.5 feet bgs on 11-9-23 | 11/9/23 | 1005 |

| Depth (ft) | Field ID + Sample Type | SPT N-value (uncorrected) | DRIVE (in) | RECOVERY (in) | USCS Soil Type | LITHOLOGIC DESCRIPTION AND REMARKS | Organic Matter (%) | Moisture Content (%) | Passing No. 200 Sieve (%) | Liquid Limit | Plasticity Index |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | | | | | 30 inches of crushed aggregate. FILL. | | | | | |
| 2.5–4 | SPT B1.1 | 2 | 18 | 18 | | Very soft to soft, brown and black SILT, moist, low to medium plasticity, trace organics. | 6.0 | 38 | 61 | | |
| 4.5–6.5 | SHELBY B1.1A | | 24 | 20 | | | | 38 | | | |
| 6.5–8 | SPT B1.2 | 4 | 18 | 14 | ML | Soft to medium stiff, brown SILT with sand, wet. | | | | | |
| 10–11.5 | SPT B1.3 | 1 | 18 | 12 | | Very soft at 10 feet. | | 45 | 52 | | |
| 13 | | | | | | Becomes with gravel at 13 feet. | | | | | |
| 15–15.8 | SPT B1.4 | 50/4" | 10 | 6 | GP-GM | Very dense, grey poorly graded GRAVEL with silt and sand, wet. | | | | | |
| 15.8 | | | | | | Bottom of boring at 15.8 feet. | | | | | |

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# BORING LOG

| PROJECT NAME | CLIENT | | PROJECT NO. | BORING NO. |
|---|---|---|---|---|
| Santiam Industrial Center | Santiam Industrial Center | | IRG-1-01-1 | B-2 |
| **PROJECT LOCATION** | **DRILLING CONTRACTOR** | **DRILL RIG** | **ENGINEER** | **PAGE NO.** |
| Stayton, Oregon | Western States | Tracked Rig #2 | SSC | 1 of 1 |
| **BORING LOCATION** | **DRILLING METHOD** | **SAMPLING METHOD** | **START DATE** | **START TIME** |
| See Figure 2 | Hollow Stem Auger | SPT (automatic) | 11/9/23 | 1015 |
| **REMARKS** | | **GROUNDWATER DEPTH** | **FINISH DATE** | **FINISH TIME** |
| None | | 6 feet bgs on 11-9-23 | 11/9/23 | 1140 |

| Depth (ft) | Field ID + Sample Type | SPT N-value (uncorrected) | DRIVE (in) | RECOVERY (in) | USCS Soil Type | LITHOLOGIC DESCRIPTION AND REMARKS | Organic Matter (%) | Moisture Content (%) | Passing No. 200 Sieve (%) | Liquid Limit | Plasticity Index |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0–2.5 | | | | | | 6 inches of asphalt concrete underlain by 24 inches of crushed aggregate. FILL. | | | | | |
| 2.5–4 | SPT B2.1 | 4 | 18 | 14 | | Soft to medium stiff, brown and black SILT, moist, low to medium plasticity, trace organics. | 4.7 | | | | |
| 5–6.5 | SPT B2.2 | 14 | 18 | 3 | SP-SM | Very loose to loose, brown and grey SAND with silt, wet, fine to medium. | | 20 | 10 | | |
| 7.5–9 | SPT B2.3 | 6 | 18 | 2 | | | | | | | |
| 10–11.5 | SPT B2.4 | 80 | 18 | 12 | GP-GM | Very dense, grey poorly graded GRAVEL with silt and sand, wet. | | | | | |
| 13–14.3 | SPT B2.5 | 48-50/3" | 15 | 10 | | | | | | | |
| 14.3 | | | | | | Bottom of boring at 14.3 feet. | | | | | |

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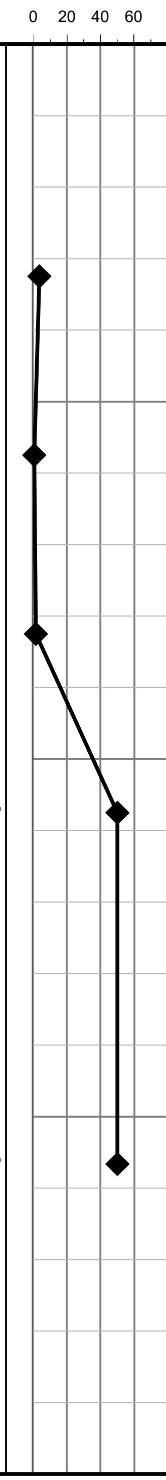
# BORING LOG

| PROJECT NAME | CLIENT | | PROJECT NO. | BORING NO. |
|---|---|---|---|---|
| Santiam Industrial Center | Santiam Industrial Center | | IRG-1-01-1 | B-3 |
| PROJECT LOCATION | DRILLING CONTRACTOR | DRILL RIG | ENGINEER | PAGE NO. |
| Stayton, Oregon | Western States | Tracked Rig #2 | SSC | 1 of 1 |
| BORING LOCATION | DRILLING METHOD | SAMPLING METHOD | START DATE | START TIME |
| See Figure 2 | Hollow Stem Auger | SPT (automatic) | 11/9/23 | 1154 |
| REMARKS | | GROUNDWATER DEPTH | FINISH DATE | FINISH TIME |
| None | | 5 feet bgs on 11-9-23 | 11/9/23 | 1305 |

| Depth (ft) | Field ID + Sample Type | SPT N-value (uncorrected) | DRIVE (in) | RECOVERY (in) | USCS Soil Type | LITHOLOGIC DESCRIPTION AND REMARKS | Organic Matter (%) | Moisture Content (%) | Passing No. 200 Sieve (%) | Liquid Limit | Plasticity Index |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0–1.5 | | | | | | 6 inches of asphalt concrete underlain by 12 inches of crushed aggregate. FILL. | | | | | |
| 1.5–2.5 | | | | | | Soft to medium stiff, brown and black SILT, moist, low to medium plasticity. | | | | | |
| 2.5–4 | SPT B3.1 | 4 | 18 | 8 | | | | 28 | 63 | | |
| 5–6.5 | SPT B3.2 | 1 | 18 | 8 | ▼ (5 ft) | | | | | | |
| 7.5–9 | SPT B3.3 | 2 | 18 | 12 | SM | Very loose, brown and grey silty SAND, wet, fine to medium. | | 40 | 34 | | |
| 10–11.5 | SPT B3.4 | 42-50/2" | 14 | 8 | GP-GM | Very dense, grey poorly graded GRAVEL with silt and sand, wet. | | | | | |
| 15–16.3 | SPT B3.5 | 19-50/4" | 16 | 8 | | | | | | | |
| 16.3 | | | | | | Bottom of boring at 16.3 feet. | | | | | |



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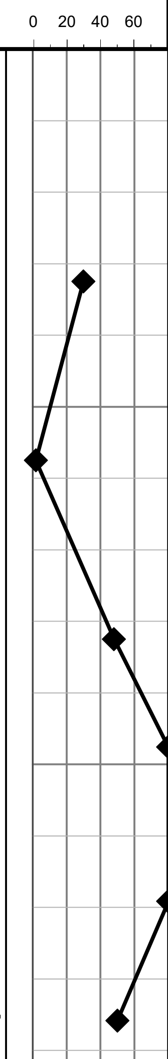
# BORING LOG

| PROJECT NAME | CLIENT | | PROJECT NO. | BORING NO. |
|---|---|---|---|---|
| Santiam Industrial Center | Santiam Industrial Center | | IRG-1-01-1 | B-4 |
| **PROJECT LOCATION** | **DRILLING CONTRACTOR** | **DRILL RIG** | **ENGINEER** | **PAGE NO.** |
| Stayton, Oregon | Western States | Tracked Rig #2 | SSC | 1 of 1 |
| **BORING LOCATION** | **DRILLING METHOD** | **SAMPLING METHOD** | **START DATE** | **START TIME** |
| See Figure 2 | Hollow Stem Auger | SPT (automatic) | 11/9/23 | 1318 |
| **REMARKS** | | **GROUNDWATER DEPTH** | **FINISH DATE** | **FINISH TIME** |
| None | | 6 feet bgs on 11-9-23 | 11/9/23 | 1445 |




| Depth (ft) | Field ID + Sample Type | SPT N-value (uncorrected) | DRIVE (in) | RECOVERY (in) | USCS Soil Type | Graphic Log | LITHOLOGIC DESCRIPTION AND REMARKS | Organic Matter (%) | Moisture Content (%) | Passing No. 200 Sieve (%) | Liquid Limit | Plasticity Index |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0–0.3 | | | | | | | 1-inch root zone underlain by 3 inches of topsoil. | | | | | |
| 0.3–3.5 | | | | | | | Medium dense to dense, brown and grey silty GRAVEL, moist, (rounded cobbles up to 6"). FILL. | | | | | |
| 2.5–4 | SPT B4.1 | 30 | 18 | 0 | | | | | | | | |
| 3.5–7.5 | | | | | | | Soft to medium stiff, brown and black SILT, moist, low to medium plasticity, trace organics. | | | | | |
| 5–6.5 | SPT B4.2 | 2 | 18 | 12 | ▼ (6) | | | 6.0 | | | | |
| 7.5–14.2 | | | | | GP-GM | | Very dense, grey poorly graded GRAVEL with silt and sand, wet, fine gravel. | | | | | |
| 7.5–9 | SPT B4.3 | 48 | 18 | 8 | | | | | | | | |
| 10–11.5 | SPT B4.4 | 101 | 18 | 8 | | | | | 17 | 6 | | |
| 13–14.2 | SPT B4.5 | 50-50/2" | 14 | 12 | | | | | | | | |
| 14.2 | | | | | | | Bottom of boring at 14.2 feet. | | | | | |

# APPENDIX B
# LABORATORY TESTING

## CLASSIFICATION

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

## ORGANIC CONTENT TESTING

Organic content testing was completed on soil samples collected in the observed fill in general accordance with ASTM D2974. The test results are presented in this appendix.

## PARTICLE-SIZE ANALYSIS

We completed particle-size analyses on select soil samples in order to determine the distribution of soil particle sizes. The testing consisted of percent fines determination (percent passing the U.S. Standard No. 200 sieve) analyses completed in general accordance with ASTM D1140 (P200). The test results are presented in this appendix.

## MOISTURE CONTENT

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

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# MOISTURE CONTENT, PERCENT PASSING NO. 200 SIEVE BY WASHING

| PROJECT | CLIENT | PROJECT NO. | REPORT DATE |
|---|---|---|---|
| Santiam Industrial Center<br>Stayton, Oregon | Santiam Industrial Center<br>c/o IRG Realty Advisors, LLC<br>4020 Kinross Lakes Parkway, Ste 200<br>Richfield, Ohio 44286 | IRG-1-01-1 | 11/27/23 |
| | | SAMPLED BY: SSC | PAGE: 1 of 1 |
| | | DATE SAMPLED: 11/09/23 | |

## LABORATORY TEST DATA

TEST PROCEDURE
ASTM D2216 - Method A, ASTM D1140

| LAB ID | CONTAINER MASS (g) | MOIST MASS + CONTAINER (g) | DRY MASS + CONTAINER (g) | AFTER WASH DRY MASS + CONTAINER (g) | FIELD ID | SAMPLE DEPTH (ft) | PERCENT MOISTURE CONTENT | PERCENT PASSING NO. 200 SIEVE |
|---|---|---|---|---|---|---|---|---|
| S23-1495 | 541.67 | 781.37 | 714.85 | 609.59 | B1.1 | 2.5 | 38% | 61% |
| S23-1496 | 547.84 | 811.78 | 739.17 | - | B1.1A | 4.5 | 38% | - |
| S23-1497 | 555.83 | 788.46 | 716.49 | 633.40 | B1.3 | 10 | 45% | 52% |
| S23-1499 | 556.20 | 842.73 | 794.52 | 769.69 | B2.2 | 5 | 20% | 10% |
| S23-1500 | 540.69 | 794.53 | 739.08 | 614.82 | B3.1 | 2.5 | 28% | 63% |
| S23-1501 | 542.34 | 789.16 | 718.28 | 658.80 | B3.3 | 7.5 | 40% | 34% |
| S23-1503 | 548.29 | 1,264.44 | 1,159.96 | 1,124.61 | B4.4 | 10 | 17% | 6% |
| | | | | | | | | |
| | | | | | | | | |
| | | | | | | | | |
| | | | | | | | | |
| | | | | | | | | |
| | | | | | | | | |
| | | | | | | | | |
| | | | | | | | | |
| | | | | | | | | |

NOTES:
Sample weights received for Lab ID: S23-1499 and 1503 did not meet the minimum size requirements; entire sample used for analysis.

DATE TESTED: 11/21/23
TESTED BY: KMS

COLUMBIA WEST ENGINEERING, INC. authorized signature

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November 27, 2023

Santiam Industrial Center
c/o IRG Realty Advisors, LLC
Attn: Coby Holley
4020 Kinross Lakes Parkway, Suite 200
Richfield, Ohio 44286

**Re: Percent Organics**
**Santiam Industrial Center**
**Stayton, Oregon**
**CWE Project: IRG-1-01-1**

Columbia West Engineering, Inc. is pleased to submit test results for material sampled during the November 9, 2023 subsurface investigation at the above-referenced site. Final weights were recorded November 17, 2023. Results for moisture and organic content are presented in Table 1.

*Table 1. Determination of Moisture and Organic Content*

| ASTM D2974 *Moisture, Ash, and Organic Matter of Peat and Other Organic Soils – method A (440 °C)* | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| LAB ID | FIELD ID | SAMPLE DEPTH | SPECIMEN WET MASS (g) | SPECIMEN DRY MASS (g) | MASS ASHED SPECIMEN (g) | TIME IN FURNACE (hrs) | PERCENT MOISTURE (oven-dried) | PERCENT ASH | ORGANIC MATTER |
| S23-1495 | B1.1 | 2.5 ft | 54.44 | 38.77 | 36.44 | 15 | 40% | 94.0% | 6.0% |
| S23-1498 | B2.1 | 2.5 ft | 54.23 | 40.99 | 39.06 | 15 | 32% | 95.3% | 4.7% |
| S23-1502 | B4.2 | 5 ft | 52.19 | 37.77 | 35.49 | 15 | 38% | 94.0% | 6.0% |

Results apply only to the samples analyzed. Columbia West appreciates the opportunity to provide materials testing services. Please call me at 360-823-2900 if you have any questions or need additional information.

♦♦♦

Sincerely,
Columbia West Engineering, Inc.

Jared J. Comastro, CET
Laboratory Manager

## APPENDIX C
## WATER WELL LOGS

This appendix contains water well logs from explorations completed at the site. The logs were obtained from the Oregon Water Resources Department. The locations of the logs are shown on Figure 2.

# MARI 70315

**STATE OF OREGON**
**GEOTECHNICAL HOLE REPORT**
**(as required by OAR 690-240-0035)**

**3/14/2022**

## (1) OWNER/PROJECT

Hole Number P3

**PROJECT NAME/NBR:** NDHC01PH1.21E

First Name ______ Last Name ______
Company NORPAC STAYTON FACILITY
Address 930 W WASHINGTON STREET
City STAYTON State OR Zip 97383

## (2) TYPE OF WORK

[X] New [ ] Deepening [X] Abandonment
[ ] Alteration (repair/recondition)

## (3) CONSTRUCTION

[ ] Rotary Air [ ] Hand Auger [ ] Hollow stem auger
[ ] Rotary Mud [ ] Cable [X] Push Probe
[ ] Other ______

## (4) TYPE OF HOLE:

(●) Uncased Temporary ( ) Cased Permanent
( ) Uncased Permanent ( ) Slope Stablity
( ) Other
Other: ______

## (5) USE OF HOLE

SOIL AND GW SAMPLING

## (6) BORE HOLE CONSTRUCTION

Special Standard [ ] (Attach copy)
Depth of Completed Hole 15.00 ft.

| BORE HOLE Dia | From | To | SEAL Material | From | To | Amt | sacks/ lbs |
|---|---|---|---|---|---|---|---|
| 2.5 | 0 | 15 | Bentonite Chips | 0 | 15 | 15 | P |
| | | | | | | | |
| | | | | | | | |
| | | | | | | | |

Backfill placed from ______ ft. to ______ ft. Material ______
Filter pack from ______ ft. to ______ ft. Material ______ Size ______

## (7) CASING/SCREEN

| Casing | Screen | Dia | + | From | To | Gauge | Stl | Plstc | Wld | Thrd |
|---|---|---|---|---|---|---|---|---|---|---|
| (●) | ( ) | 0.75 | [ ] | 0 | 5 | Sch.40 | ( ) | (●) | [ ] | [X] |
| ( ) | (●) | 0.75 | [ ] | 5 | 15 | Sch.40 | ( ) | (●) | [ ] | [X] |
| ( ) | ( ) | | [ ] | | | | ( ) | ( ) | [ ] | [ ] |
| ( ) | ( ) | | [ ] | | | | ( ) | ( ) | [ ] | [ ] |
| ( ) | ( ) | | [ ] | | | | ( ) | ( ) | [ ] | [ ] |

## (8) WELL TESTS

( ) Pump ( ) Bailer ( ) Air ( ) Flowing Artesian

| Yield gal/min | Drawdown | Drill stem/Pump depth | Duration(hr) |
|---|---|---|---|
| | | | |
| | | | |
| | | | |

Temperature 55 °F Lab analysis [X] Yes By APEX
Supervising Geologist/Engineer Steve Omo, BB&A Environmental, Inc.
Water quality concerns? [ ] Yes (describe below) TDS amount 0 ug/L

| From | To | Description | Amount | Units |
|---|---|---|---|---|
| | | | | |
| | | | | |
| | | | | |

## (9) LOCATION OF HOLE (legal description)

County MARION Twp 9.00 S N/S Range 1.00 W E/W WM
Sec 10 NW 1/4 of the SW 1/4 Tax Lot 2400
Tax Map Number ______ Lot ______
Lat ___° ___' ___" or ______ DMS or DD
Long ___° ___' ___" or ______ DMS or DD
(●) Street address of hole ( ) Nearest address

930 W WASHINGTON STREET
STAYTON, OR 97383

## (10) STATIC WATER LEVEL

| | Date | SWL(psi) | + | SWL(ft) |
|---|---|---|---|---|
| Existing Well / Predeepening | | | [ ] | |
| Completed Well | 2/9/2022 | | [ ] | 5.5 |

Flowing Artesian? [ ]
WATER BEARING ZONES
Depth water was first found 5.50

| SWL Date | From | To | Est Flow | SWL(psi) | + | SWL(ft) |
|---|---|---|---|---|---|---|
| 2/9/2022 | 5.5 | 15 | 1 | | [ ] | 5.5 |
| | | | | | [ ] | |
| | | | | | [ ] | |
| | | | | | [ ] | |
| | | | | | [ ] | |

## (11) SUBSURFACE LOG

Ground Elevation ______

| Material | From | To |
|---|---|---|
| Asphalt/Gravel Fill | 0 | 1.5 |
| Silt, brown Clayey, firm | 1.5 | 5.5 |
| Gravel, Sandy, wet, alluvium | 5.5 | 15 |
| | | |
| | | |
| | | |
| | | |
| | | |
| | | |
| | | |

Date Started 2/9/2022 Completed 2/9/2022

## (12) ABANDONMENT LOG:

| Material | From | To | Amt | sacks/ lbs |
|---|---|---|---|---|
| Bentonite Chips | 0 | 15 | 15 | P |
| | | | | |
| | | | | |
| | | | | |
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| | | | | |
| | | | | |
| | | | | |

Date Started 2/9/2022 Completed 2/9/2022

**Professional Certification** (to be signed by an Oregon licensed water or monitoring well constructor, Oregon registered geologist or professional engineer).

I accept responsibility for the construction, deepening, alteration, or abandonment work performed during the construction dates reported above. All work performed during this time is in compliance with Oregon geotechnical hole construction standards. This report is true to the best of my knowledge and belief.

License/Registration Number 10288 Date 3/14/2022
First Name ROBERT Last Name BOESE
Affiliation BB&A ENVIRONMENTAL, INC.

GEOTECHNICAL HOLE REPORT - Map with location identified must be attached and shall include an approximate scale and north arrow

**MARI 70315**

**3/14/2022**

Map of Hole

# MARI 70316

**STATE OF OREGON**
**GEOTECHNICAL HOLE REPORT**
**(as required by OAR 690-240-0035)**

**3/14/2022**

## (1) OWNER/PROJECT

Hole Number P4

**PROJECT NAME/NBR:** NDHC01PH1.21E

First Name
Last Name
Company NORPAC STAYTON FACILITY
Address 930 W WASHINGTON STREET
City STAYTON State OR Zip 97383

## (2) TYPE OF WORK

☒ New ☐ Deepening ☒ Abandonment ☐ Alteration (repair/recondition)

## (3) CONSTRUCTION

☐ Rotary Air ☐ Hand Auger ☐ Hollow stem auger
☐ Rotary Mud ☐ Cable ☒ Push Probe
☐ Other

## (4) TYPE OF HOLE:

◉ Uncased Temporary ○ Cased Permanent
○ Uncased Permanent ○ Slope Stablity
○ Other
Other:

## (5) USE OF HOLE

SOIL AND GW SAMPLING

## (6) BORE HOLE CONSTRUCTION

Special Standard ☐ (Attach copy)

Depth of Completed Hole 15.00 ft.

| BORE HOLE Dia | From | To | SEAL Material | From | To | Amt | sacks/ lbs |
|---|---|---|---|---|---|---|---|
| 2.5 | 0 | 15 | Bentonite Chips | 0 | 15 | 15 | P |
| | | | | | | | |
| | | | | | | | |
| | | | | | | | |

Backfill placed from ft. to ft. Material
Filter pack from ft. to ft. Material Size

## (7) CASING/SCREEN

| Casing | Screen | Dia | + | From | To | Gauge | Stl | Plstc | Wld | Thrd |
|---|---|---|---|---|---|---|---|---|---|---|
| ◉ | ○ | 0.75 | ☐ | 0 | 5 | Sch.40 | ○ | ◉ | ☐ | ☒ |
| ○ | ◉ | 0.75 | ☐ | 5 | 15 | Sch.40 | ○ | ◉ | ☐ | ☒ |
| ○ | ○ | | ☐ | | | | ○ | ○ | ☐ | ☐ |
| ○ | ○ | | ☐ | | | | ○ | ○ | ☐ | ☐ |
| ○ | ○ | | ☐ | | | | ○ | ○ | ☐ | ☐ |

## (8) WELL TESTS

○ Pump ○ Bailer ○ Air ○ Flowing Artesian

| Yield gal/min | Drawdown | Drill stem/Pump depth | Duration(hr) |
|---|---|---|---|
| | | | |
| | | | |
| | | | |

Temperature 55 °F Lab analysis ☒ Yes By APEX

Supervising Geologist/Engineer Steve Omo, BB&A Environmental, Inc.

Water quality concerns? ☐ Yes (describe below) TDS amount 0 ug/L

| From | To | Description | Amount | Units |
|---|---|---|---|---|
| | | | | |
| | | | | |
| | | | | |

## (9) LOCATION OF HOLE (legal description)

County MARION Twp 9.00 S N/S Range 1.00 W E/W WM
Sec 10 NW 1/4 of the SW 1/4 Tax Lot 2400
Tax Map Number Lot
Lat ° ' " or DMS or DD
Long ° ' " or DMS or DD

◉ Street address of hole ○ Nearest address

930 W WASHINGTON STREET
STAYTON, OR 97383

## (10) STATIC WATER LEVEL

| | Date | SWL(psi) | + | SWL(ft) |
|---|---|---|---|---|
| Existing Well / Predeepening | | | ☐ | |
| Completed Well | 2/9/2022 | | ☐ | 6.5 |

Flowing Artesian? ☐

WATER BEARING ZONES

Depth water was first found 6.50

| SWL Date | From | To | Est Flow | SWL(psi) | + | SWL(ft) |
|---|---|---|---|---|---|---|
| 2/9/2022 | 6.5 | 15 | 1 | | ☐ | 6.5 |
| | | | | | ☐ | |
| | | | | | ☐ | |
| | | | | | ☐ | |
| | | | | | ☐ | |

## (11) SUBSURFACE LOG

Ground Elevation

| Material | From | To |
|---|---|---|
| Asphalt/Concrete | 0 | 1 |
| Silt, brown Clayey, firm | 1 | 9 |
| Sand, brown-grey, silty, wet | 9 | 12 |
| Gravel, brown-grey, silty-sandy/alluvium | 12 | 15 |
| | | |
| | | |
| | | |
| | | |
| | | |
| | | |

Date Started 2/9/2022 Completed 2/9/2022

## (12) ABANDONMENT LOG:

| Material | From | To | Amt | sacks/ lbs |
|---|---|---|---|---|
| Bentonite Chips | 0 | 15 | 15 | P |
| | | | | |
| | | | | |
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Date Started 2/9/2022 Completed 2/9/2022

**Professional Certification** (to be signed by an Oregon licensed water or monitoring well constructor, Oregon registered geologist or professional engineer).

I accept responsibility for the construction, deepening, alteration, or abandonment work performed during the construction dates reported above. All work performed during this time is in compliance with Oregon geotechnical hole construction standards. This report is true to the best of my knowledge and belief.

License/Registration Number 10288 Date 3/14/2022

First Name ROBERT Last Name BOESE
Affiliation BB&A ENVIRONMENTAL, INC.

GEOTECHNICAL HOLE REPORT - Map with location identified must be attached and shall include an approximate scale and north arrow

**MARI 70316**

**3/14/2022**

# Map of Hole

# MARI 70317

**STATE OF OREGON**
**GEOTECHNICAL HOLE REPORT**
**(as required by OAR 690-240-0035)**

**3/14/2022**

## (1) OWNER/PROJECT

Hole Number: P5

**PROJECT NAME/NBR:** NDHC01PH1.21E

First Name: Last Name:
Company: NORPAC STAYTON FACILITY
Address: 930 W WASHINGTON STREET
City: STAYTON State: OR Zip: 97383

## (2) TYPE OF WORK

☒ New ☐ Deepening ☒ Abandonment
☐ Alteration (repair/recondition)

## (3) CONSTRUCTION

☐ Rotary Air ☐ Hand Auger ☐ Hollow stem auger
☐ Rotary Mud ☐ Cable ☒ Push Probe
☐ Other

## (4) TYPE OF HOLE:

◉ Uncased Temporary ○ Cased Permanent
○ Uncased Permanent ○ Slope Stablity
○ Other
Other:

## (5) USE OF HOLE

SOIL AND GW SAMPLING

## (6) BORE HOLE CONSTRUCTION

Special Standard ☐ (Attach copy)

Depth of Completed Hole 13.50 ft.

| BORE HOLE Dia | From | To | SEAL Material | From | To | Amt | sacks/lbs |
|---|---|---|---|---|---|---|---|
| 2.5 | 0 | 13.5 | Bentonite Chips | 0 | 13.5 | 15 | P |
| | | | | | | | |
| | | | | | | | |
| | | | | | | | |

Backfill placed from ft. to ft. Material
Filter pack from ft. to ft. Material Size

## (7) CASING/SCREEN

| Casing | Screen | Dia | + | From | To | Gauge | Stl | Plstc | Wld | Thrd |
|---|---|---|---|---|---|---|---|---|---|---|
| ◉ | ○ | 0.75 | ☐ | 0 | 3.5 | Sch.40 | ○ | ◉ | ☐ | ☒ |
| ○ | ◉ | 0.75 | ☐ | 3.5 | 13.5 | Sch.40 | ○ | ◉ | ☐ | ☒ |
| ○ | ○ | | ☐ | | | | ○ | ○ | ☐ | ☐ |
| ○ | ○ | | ☐ | | | | ○ | ○ | ☐ | ☐ |
| ○ | ○ | | ☐ | | | | ○ | ○ | ☐ | ☐ |

## (8) WELL TESTS

○ Pump ○ Bailer ○ Air ○ Flowing Artesian

| Yield gal/min | Drawdown | Drill stem/Pump depth | Duration(hr) |
|---|---|---|---|
| | | | |
| | | | |
| | | | |

Temperature 55 °F Lab analysis ☒ Yes By APEX

Supervising Geologist/Engineer: Steve Omo, BB&A Environmental, Inc.

Water quality concerns? ☐ Yes (describe below) TDS amount 0 ug/L

| From | To | Description | Amount | Units |
|---|---|---|---|---|
| | | | | |
| | | | | |
| | | | | |

## (9) LOCATION OF HOLE (legal description)

County: MARION Twp: 9.00 S N/S Range: 1.00 W E/W WM
Sec: 10 NW 1/4 of the SW 1/4 Tax Lot: 2400
Tax Map Number: Lot:
Lat: ° ' " or DMS or DD
Long: ° ' " or DMS or DD

◉ Street address of hole ○ Nearest address

930 W WASHINGTON STREET
STAYTON, OR 97383

## (10) STATIC WATER LEVEL

| | Date | SWL(psi) | + | SWL(ft) |
|---|---|---|---|---|
| Existing Well / Predeepening | | | ☐ | |
| Completed Well | 2/9/2022 | | ☐ | 6 |

Flowing Artesian? ☐

WATER BEARING ZONES

Depth water was first found 6.00

| SWL Date | From | To | Est Flow | SWL(psi) | + | SWL(ft) |
|---|---|---|---|---|---|---|
| 2/9/2022 | 6 | 13.5 | 1 | | ☐ | 5.5 |
| | | | | | ☐ | |
| | | | | | ☐ | |
| | | | | | ☐ | |
| | | | | | ☐ | |

## (11) SUBSURFACE LOG

Ground Elevation

| Material | From | To |
|---|---|---|
| Asphalt/Gravel Fill | 0 | 1 |
| Silt, brown Clayey, firm | 1 | 9 |
| Gravel, grey, Silty-Sandy, wet, alluvium | 9 | 13.5 |
| | | |
| | | |
| | | |
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| | | |
| | | |

Date Started 2/9/2022 Completed 2/9/2022

## (12) ABANDONMENT LOG:

| Material | From | To | Amt | sacks/lbs |
|---|---|---|---|---|
| Bentonite Chips | 0 | 13.5 | 15 | P |
| | | | | |
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Date Started 2/9/2022 Completed 2/9/2022

**Professional Certification** (to be signed by an Oregon licensed water or monitoring well constructor, Oregon registered geologist or professional engineer).

I accept responsibility for the construction, deepening, alteration, or abandonment work performed during the construction dates reported above. All work performed during this time is in compliance with Oregon geotechnical hole construction standards. This report is true to the best of my knowledge and belief.

License/Registration Number 10288 Date 3/14/2022

First Name ROBERT Last Name BOESE
Affiliation BB&A ENVIRONMENTAL, INC.

GEOTECHNICAL HOLE REPORT - Map with location identified must be attached and shall include an approximate scale and north arrow

**MARI 70317**

**3/14/2022**

# Map of Hole

# APPENDIX D
# REPORT LIMITATATIONS AND IMPORTANT INFORMATION

## Report Purpose, Use, and Standard of Care

This report has been prepared in accordance with standard fundamental principles and practices of geotechnical engineering and/or environmental consulting, and in a manner consistent with the level of care and skill typical of currently practicing local engineers and consultants. This report has been prepared to meet the specific needs of specific individuals for the indicated site. It may not be adequate for use by other consultants, contractors, or engineers, or if change in project ownership has occurred. It should not be used for any other reason than its stated purpose without prior consultation with Columbia West Engineering, Inc. (Columbia West). It is a unique report and not applicable for any other site or project. If site conditions are altered, or if modifications to the project description or proposed plans are made after the date of this report, it may not be valid. Columbia West cannot accept responsibility for use of this report by other individuals for unauthorized purposes, or if problems occur resulting from changes in site conditions for which Columbia West was not aware or informed.

## Report Conclusions and Preliminary Nature

This geotechnical or environmental report should be considered preliminary and summary in nature. The recommendations contained herein have been established by engineering interpretations of subsurface soils based upon conditions observed during site exploration. The exploration and associated laboratory analysis of collected representative samples identifies soil conditions at specific discreet locations. It is assumed that these conditions are indicative of actual conditions throughout the subject property. However, soil conditions may differ between tested locations at different seasonal times of the year, either by natural causes or human activity. Distinction between soil types may be more abrupt or gradual than indicated on the soil logs. This report is not intended to stand alone without understanding of concomitant instructions, correspondence, communication, or potential supplemental reports that may have been provided to the client.

Because this report is based upon observations obtained at the time of exploration, its adequacy may be compromised with time. This is particularly relevant in the case of natural disasters, earthquakes, floods, or other significant events. Report conclusions or interpretations may also be subject to revision if significant development or other manmade impacts occur within or in proximity to the subject property. Groundwater conditions, if presented in this report, reflect observed conditions at the time of investigation. These conditions may change annually, seasonally or as a result of adjacent development.

## Additional Investigation and Construction QA/QC

Columbia West should be consulted prior to construction to assess whether additional investigation above and beyond that presented in this report is necessary. Even slight variations in soil or site conditions may produce impacts to the performance of structural facilities if not adequately addressed. This underscores the importance of diligent QA/QC construction observation and testing to verify soil conditions do not differ materially or significantly from the interpreted conditions utilized for preparation of this report.

Therefore, this report contains several recommendations for field observation and testing by Columbia West personnel during construction activities. Actual subsurface conditions are more readily observed and discerned during the earthwork phase of construction when soils are exposed. Columbia West cannot accept responsibility for deviations from recommendations described in this report or future performance of structural facilities if another consultant is retained during the construction phase or Columbia West is not engaged to provide construction observation to the full extent recommended.

Collected Samples

Uncontaminated samples of soil or rock collected in connection with this report will be retained for thirty days. Retention of such samples beyond thirty days will occur only at client's request and in return for payment of storage charges incurred. All contaminated or environmentally impacted materials or samples are the sole property of the client. Client maintains responsibility for proper disposal.

## Report Contents

This geotechnical or environmental report should not be copied or duplicated unless in full, and even then, only under prior written consent by Columbia West, as indicated in further detail in the following text section entitled Report Ownership. The recommendations, interpretations, and suggestions presented in this report are only understandable in context of reference to the whole report. Under no circumstances should the soil boring or test pit excavation logs, monitor well logs, or laboratory analytical reports be separated from the remainder of the report. The logs or reports should not be redrawn or summarized by other entities for inclusion in architectural or civil drawings, or other relevant applications.

## Report Limitations for Contractors

Geotechnical or environmental reports, unless otherwise specifically noted, are not prepared for the purpose of developing cost estimates or bids by contractors. The extent of exploration or investigation conducted as part of this report is usually less than that necessary for contractor's needs. Contractors should be advised of these report limitations, particularly as they relate to development of cost estimates. Contractors may gain valuable information from this report, but should rely upon their own interpretations as to how subsurface conditions may affect cost, feasibility, accessibility and other components of the project work. If believed necessary or relevant, contractors should conduct additional exploratory investigation to obtain satisfactory data for the purposes of developing adequate cost estimates. Clients or developers cannot insulate themselves from attendant liability by disclaiming accuracy for subsurface ground conditions without advising contractors appropriately and providing the best information possible to limit potential for cost overruns, construction problems, or misunderstandings.

## Report Ownership

Columbia West retains the ownership and copyright property rights to this entire report and its contents, which may include, but may not be limited to, figures, text, logs, electronic media, drawings, laboratory reports, and appendices. This report was prepared solely for the client, and other relevant approved users or parties, and its distribution must be contingent upon prior express written consent by Columbia West. Furthermore, client or approved users may not use, lend, sell, copy, or distribute this document without express written consent by Columbia West. Client does not own nor have rights to electronic media files that constitute this report, and under no circumstances should said electronic files be distributed or copied. Electronic media is susceptible to unauthorized manipulation or modification, and may not be reliable.

## Consultant Responsibility

Geotechnical and environmental engineering and consulting is much less exact than other scientific or engineering disciplines, and relies heavily upon experience, judgment, interpretation, and opinion often based upon media (soils) that are variable, anisotropic, and non-homogenous. This often results in unrealistic expectations, unwarranted claims, and uninformed disputes against a geotechnical or environmental consultant. To reduce potential for these problems and assist relevant parties in better understanding of risk, liability, and responsibility, geotechnical and environmental reports often provide definitive statements or clauses defining and outlining consultant responsibility. The client is encouraged to read these statements carefully and request additional information from Columbia West if necessary.