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OREGON
JUNE 2, 2010
BRENT DANIEL NIELSEN
EXPIRES: 12/31/22

PRELIMINARY STORM DRAINAGE REPORT

To
Columbia County

For
NEXT Renewable Fuels
Oregon
Port Westward

Dated
January 8, 2021

Project Number
2200315.01

MACKENZIE
Since 1960

RiverEast Center | 1515 SE Water Ave, Suite 100, Portland, OR 97214
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ATTACHMENTS

I. PROJECT INTRODUCTION

The proposed NEXT Renewables facility includes development of renewable diesel refining, processing, and storage uses at the Port Westward property near Clatskanie, Oregon. The development will include the industrial and processing uses, as well as buildings, parking, utilities, roadways, and rail spurs to support the biofuels production systems.

The project is located at Port Westward along the southern bank of the Columbia River. The vicinity map and site plan below show the project location and overall scope of the project development.

Figure 1: Vicinity Map

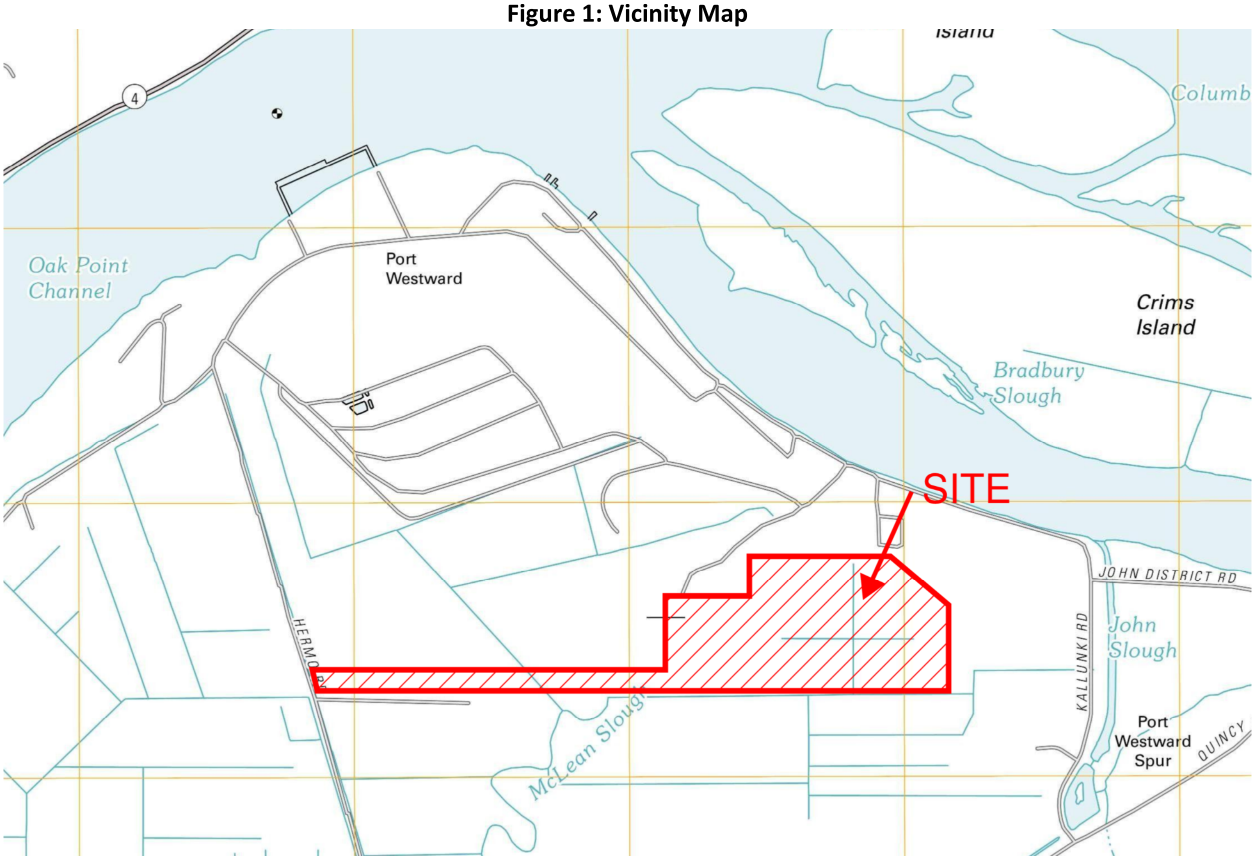
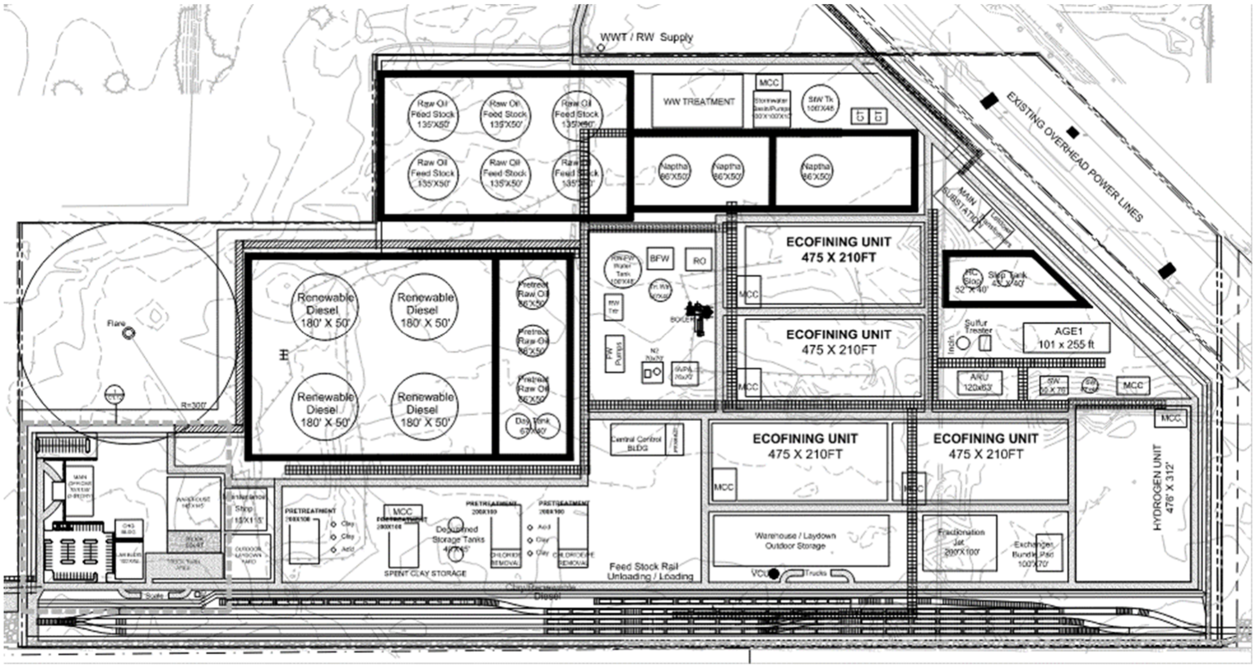


Figure 2: Main Plant Site Plan



I. EXISTING DRAINAGE CONDITIONS

The project site comprises approximately 108.0 acres which is primarily covered with existing agriculture and open land. This area comprises the renewable diesel production facility, access road, rail spur, and pipeline footprints. A network of excavated ditches crosses the site and primarily directs storm water to McLean Slough near the southwest corner of the site.

The following summarizes the existing ground coverage of the project site:

Existing Gravel Roads (Impervious):	0.73 ac
Existing Vegetation (Pervious):	126.70 ac
Total Existing Site Area:	127.43 ac

Existing conditions plans are provided in Appendix A of this report.

The existing site soils primarily consist of the Udipsamments and Silt Loam soils, which generally have the following drainage characteristics:

- Udipsamments: sandy, well-drained soils, hydrologic soil group A
- Wauna-Lacoda Silt Loam: loamy, poorly-drained soils, hydrologic soil group C
- Wauna Silt Loam: loamy, poorly-drained soils, hydrologic soil group C

The soil survey, including the soil map, is provided in Appendix B of this report.

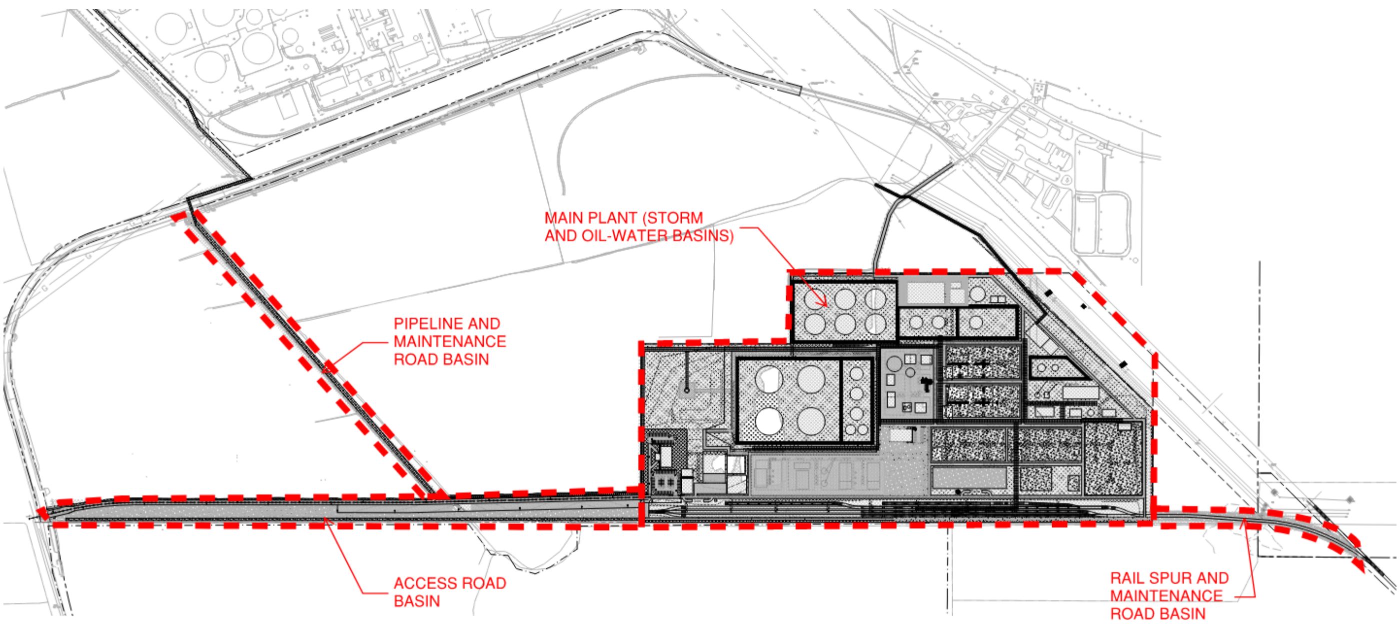
A geotechnical report was prepared in 2001 for a prior development opportunity at the project site. The subsurface investigation located the groundwater between 2 feet to 4 feet below the ground surface. Based on this finding, we do not expect infiltration to be a feasible discharge option for the site runoff. The geotechnical report is provided in Appendix F of this report.

II. PROPOSED DRAINAGE CONDITIONS

The proposed development includes construction of buildings, concrete equipment pads, paved drive aisles, and paved parking areas which comprise new impervious area on the project site. Stormwater runoff from the project area will be routed to separate drainage paths:

- Access road: runoff will be routed to a new drainage swale which discharges to existing channels
- Pipeline maintenance road and rail spur: runoff will be routed to the existing drainage ditch
- Equipment pads within the biorefinery footprint: runoff will be routed to the on-site waste water treatment facility for testing, treatment, and discharge via pump to the Port Westward storm outfall
- Non-equipment impervious surfaces within the plan footprint: runoff will be routed to the on-site storm water basin to be discharged via pumping to the Port Westward storm outfall

Each of the above areas will receive runoff treatment through various BMPs, described later in this report.



III. STORMWATER MANAGEMENT STANDARDS

The project is located within Columbia County, Oregon. Those portions of the site which discharge to non-wetland facilities will be subject to County's "Stormwater and Erosion Control Ordinance" from November 2001.

The project will include wetland fill impacts and mitigation; therefore, the application is subject to SLOPES V regulations under administration by Army Corps of Engineers and National Marine Fisheries Service. The following outlines the applicable standards for the project.

Additionally, this report is prepared to meet the requirements of Oregon DEQ Section 401 for Post-Construction Stormwater Management Plans.

Water Quality Treatment

From Columbia County *Stormwater and Erosion Control Ordinance, November 21, 2001*:

> **Section 1.C.18**: "Water quality storm" means the rainfall from a six-month, 24-hour storm. This rainfall equals approximately 64% of rainfall from the 2-year, 24-hour storm or 0.83 inches.
>
> **Appendix E**: "The Water Quality Storm equals one-third of the 2-year storm." (For Clatskanie, the water quality storm depth equates to 0.93".)
>
> **Section III.B.2.a.i**: Stormwater and Runoff from parking lots, driveways, and other exposed traffic areas shall be treated using one of the following treatment methods: biofiltration swales, vegetative filter strips, or *alternative treatment methods*.

From National Marine Fisheries Services *SLOPES for Stormwater, Transportation, or Utilities NWR-2013-10411*:

> **Section 36.e**: All stormwater quality treatment practices and facilities will be designed to accept and fully treat the volume of water equal to 50% of the cumulative rainfall from the 2-year 24-hour storm for that site. (For Clatskanie, the SLOPES V water quality storm depth equates to 1.40".)
>
> **Section 36.f**: Use low impact development practices to infiltrate or evaporate runoff to the maximum extent feasible. For runoff that cannot be infiltrated or evaporated and therefore will discharge into surface or subsurface waters, apply one or more of the following specific primary treatment practices, supplemented with appropriate soil amendments:
>
> i. Bioretention cell
> ii. Bioslope, also known as an "ecology embankment"
> iii. Bioswale
> iv. Constructed wetlands
> v. Infiltration pond
> vi. Media filter devices with demonstrated effectiveness.
> vii. Porous pavement, with no soil amendments and appropriate maintenance

From the DEQ Section 401 *Post-Construction Stormwater Management Plan Submission Guidelines:*

Section E.1.1: Multiply the 2-year 24-hour precipitation by the appropriate water quality design storm factor: ... 0.5 for the rest of the state…. If the results are less than 0.7 inch, use 0.7 inch.

For water quality treatment, the SLOPES V standards exceed the Columbia County and DEQ standards; therefore, we have used the SLOPES V standard for water quality design.

Runoff Control and Water Quantity

From Columbia County *Stormwater and Erosion Control Ordinance, November 21, 2001*:

Section III.B.2.b.i: Runoff from the development site shall be controlled such that the following criteria are met:

A) The peak flows for the 10 and 100-year design storms after development does not exceed the respective predevelopment peak flows.
B) The peak flow for the 2-year design storm after development does not exceed one-half the predevelopment peak flow for the 2-year storm.

From National Marine Fisheries Services *SLOPES for Stormwater, Transportation, or Utilities NWR-2013-10411*:

Section 36.c.iii: Water quantity treatment (retention or detention facilities), unless the outfall discharges directly into a major water body (e.g., mainstem Columbia River, Willamette River (downstream of Eugene), large lakes, reservoir, ocean, or estuary). Retention or detention facilities must limit discharge to match pre-developed discharge rates (i.e., the discharge rate of the site based on its natural groundcover and grade before any development occurred) using a continuous simulation for flows between 50% of the 2-year event and the 10-year flow event (annual series).

For runoff control, the Columbia County and SLOPES V standards are equivalent.

Storm Conveyance Design

From Columbia County *Stormwater and Erosion Control Ordinance, November 21, 2001*:

Section II.E.1: Conveyance systems shall be designed to carry runoff from the 25-year storm where the contributing drainage area is less than 40 acres and the 100-year storm where the contributing drainage area exceeds 40 acres.

From National Marine Fisheries Services *SLOPES for Stormwater, Transportation, or Utilities NWR-2013-10411*:

Section 36.g: When conveyance is necessary to discharge treated stormwater directly into surface water or a wetland, the following requirements apply:

i. Maintain natural drainage patterns.
ii. To the maximum extent feasible, ensure that water quality treatment for contributing impervious area runoff is completed before commingling with offsite runoff for conveyance.

iii. Prevent erosion of the flow path from the project to the receiving water and, if necessary, provide a discharge facility made entirely of manufactured elements (e.g., pipes, ditches, discharge facility protection) that extends at least to OHW.

For conveyance design, the Columbia County standards apply for piped and channelized flow paths.

Design Storms

The design storms used for the project are based on the Columbia County Stormwater Ordinance, Appendix E, using the rainfall depth for Clatskanie.

Table 1: Columbia County Design Storm Rainfall Depths

Storm Event	Water Quality (SLOPES V)	2-yr 24-hr	5-yr 24-hr	10-yr 24-hr	25-yr 24-hr	100-yr 24-hr
Rainfall Depth	1.40”	2.8”	3.4”	3.9”	4.5”	5.4”

Groundwater at the site is estimated to be within 5 feet of the ground surface and seasonally reaching up to the ground surface during, which limits the infiltration opportunity on the site. Storm facilities for this project are designed with the assumption that infiltration is negligible. The runoff curve numbers for the site soils are selected for hydrologic soil group C and D to reflect the low-infiltration conditions, as follows.

Table 2: Runoff Curve Numbers

Surface Coverage	Runoff Curve Number
Paved Roadway, Building Roof, and Sidewalks	98
Gravel Surfacing and Roadways	92
Proposed Landscaping	78
Existing Grass or Vegetated Field	80

We used the software Hydraflow to calculate hydrograph volumes and peak runoff rates based on a Type 1A storm and the Santa Barbara Urban Hydrograph (SBUH) calculation method.

Conveyance calculations are performed using the Rational Method. Per the Columbia County 2001 Stormwater Ordinance, the conveyance design storm is the 10-year event for basins up to 40 acres, and the 100-year event for larger basins. The rainfall intensity for conveyance flow rate determination is based on the ODOT Zone 5 IDF Curves published in the 2014 ODOT Hydraulics Manual.

IV. RUNOFF WATER QUALITY TREATMENT

Runoff water quality treatment will be provided through a variety of facilities within the four generalized drainage basins across the site:

- **Access road**: treatment swale along the south edge of the access road
- **Pipeline maintenance road and rail spur**: filter strip
- **Equipment pads within the biorefinery footprint**: oil and sediment treatment provided through on-site wastewater treatment facility
- **Non-equipment impervious surfaces within the plant footprint**: filtration treatment through final stage of on-site wastewater treatment facility

Access Road Swale Design

The proposed paved access road runs from Hermo Road at the West to the proposed main plant at the east. The road is approximately 3,800 lf and comprises a 30-ft wide paved road along the north edge and an approximately 85-ft wide gravel laydown yard to the south. The laydown area will be used for equipment staging during construction of the facility and during ongoing operation.

The access road is designed to drain to a swale along the south side of the laydown yard, which will provide water quality treatment before discharging to the adjacent existing drainage ditch. The ditch eventually connects to McLean Slough south of the project boundary.

Access road is designed to drain runoff to three discharge points along the southern edge of the laydown yard, with connecting channels to the existing drainage ditch to the south. This divides the roadway into 6 sub-basins, each of which utilizes a swale sized to handle the resulting outflow. The swales will be constructed with growing medium suitable for filtration and planted with vegetation or grass per Columbia County standards. Planting details will be developed for final design of the project. The following summarizes the preliminary swale basin sizing along the road:

Table 3: Access Road Basin Swale Sizing Summary

Swale ID	Road Stationing	Sub-Basin Area	WQ Flow	Swale Dimensions
A	STA 1+50 to 4+50	Paving: 0.29 ac Gravel: 0.29 ac Total: 0.58 ac	0.13 cfs	2’ bottom 115’ long 1.2% slope
B	STA 4+50 to 16+00	Paving: 0.79 ac Gravel: 2.20 ac Total: 2.99 ac	0.60 cfs	8’ bottom 115’ long 0.6% slope
C	STA 16+00 to 27+50	Paving: 1.72 ac Gravel: 1.39 ac Total: 3.12 ac	0.69 cfs	9.5’ bottom 115’ long 0.6% slope
D	STA 27+50 to 31+50	Paving: 0.74 ac Gravel: 0.42 ac Total: 1.16 ac	0.28 cfs	4’ bottom 105’ long 0.6% slope
E	STA 31+50 to 35+00	Paving: 0.63 ac Gravel: 0.43 ac Total: 1.05 ac	0.26 cfs	4’ bottom 100’ long 0.6% slope
F	STA 35+00 to 39+50	Paving: 0.88 ac Gravel: 0.65 ac Total: 1.53 ac	0.34 cfs	4’ bottom 100’ long 0.5% slope

Maintenance Road and Rail Spur Basin Treatment

The proposed maintenance road and rail spur basins include development of gravel-surfaced roadways and rail subgrade located northwest and southeast of the main plant area. Each of these gravel areas will be infrequently traveled by vehicles and will be surfaced with primarily open-graded aggregate base materials. Runoff from these surfaces will be treated with filter strips adjacent to the roadway, then continue to sheet flow to adjacent existing drainage basins.

The proposed filter strips will be sized to meet the required 9-minute residence time per the Columbia County 2001 Stormwater Ordinance. The filter strips are expected to extend the length of the gravel roadway with a minimum width of 5 feet as recommended in the Clean Water Services LIDA Handbook, since the Columbia County ordinance does not specify dimensional guidelines.

Oily Water Sewer Basin Treatment

The proposed NEXT Renewables facility includes equipment, piping, and structures which handle oil-based products. As is standard for similar facilities, the project proposes to provide wastewater treatment process facilities on site to monitor and treat stormwater runoff from the plant areas which may accumulate oil in the runoff due to contact with the oil-handling equipment. The proposed NEXT Renewables wastewater treatment plant will be located near the north side of the property.

The proposed wastewater treatment plant comprises a proprietary system designed to treat effluent and oily water sewer runoff from the renewable diesel facility. The system will treat effluent for oil, suspended solids, and temperature variations to meet applicable regulatory requirements. The treated discharge from the wastewater plant will flow to the existing Port Westward stormwater outfall to the Columbia River. Since the oily water sewer drainage will be mixed with other effluents from the buildings and plant facilities, the treatment and discharge standards will meet regulatory standards for those effluent streams instead of the Columbia County Stormwater Ordinance requirements.

Additional detail for the proposed wastewater treatment system is provided in Appendix E of this report.

Plant Stormwater Basin Treatment

Stormwater runoff from non-oily areas within the proposed renewable diesel plant will be collected and routed to a separate stormwater treatment facility located near the wastewaster treatment plant near the north edge of the project. The site is graded to drain internal roadways to a system of gutters and catch basins which capture runoff and isolate the non-oily portions of the plant from the equipment areas described above. Additionally, runoff from building roofs, laydown yards, parking areas, and other non-process areas is conveyed via pipe to the stormwater treatment facility.

The stormwater treatment facility consists of a surge storage tank, filtration system, and pump station. The surge storage tank will be used to moderate peak runoff flows before the runoff is routed to the tertiary filtration system located within the wastewater treatment plant. The filtration system will be designed to accommodate flows from both the wastewater treatment process and the on-site stormwater drainage system. This filtration will provide stormwater treatment to meet discharge requirements per the Columbia County 2001 Stormwater Ordinance.

The stormwater ordinance requires that "runoff from parking lots, driveways, and other exposed traffic areas shall be treated" with treatment methods sized to handle the water quality storm. The following summarizes the plant stormwater basin coverage and water quality storm:

- Paved roadways: 8.82 ac
- Gravel roadways: 19.40 ac
- Total water quality treatment area: 37.37 ac
- Water quality design flow: 5.75 cfs

Additional details of the filtration system are provided in Appendix E of this report.

V. RUNOFF WATER QUANTITY TREATMENT

Runoff water quantity control will be provided through a variety of facilities within the four generalized drainage basins across the site:

- **Access road**: check dams and wiers within the drainage swales
- **Pipeline maintenance road and rail spur**: filter strips
- **Equipment pads within the biorefinery footprint**: pumped discharge metered to regulatory limits
- **Non-equipment impervious surfaces within the plant footprint**: pumped discharge metered to regulatory limits

Access Road Runoff Flow Control

As described above, the access road basin comprises approximately 10.37 ac of paved roadway and gravel laydown yard located west of the main plant area. The drainage basin is graded to drain to the south edge of the laydown yard to a series of swales which connect to the adjacent drainage ditch at three locations.

The swale is sized to provide water quality treatment for the design storm, and periodic wier structures along the ditch will provide detention storage to reduce the discharge rate from the impervious surfaces. The following summarizes the preliminary Access Road basin detention flows.

Table 4: Access Road Basin Runoff Flow Control Summary

Road Sub-Basin ID	Sub-Basin Area and CN	Pre-Development Peak Flow Rate (cfs)	Post-Development Peak Flow Rate (cfs)	Maximum Peak Flow Rate Discharge to Ditch (cfs)
A-B STA 1+50 to 16+00	3.57 ac CN: 94	2-yr: 0.79 10-yr: 1.57 100-yr: 2.78	2-yr: 2.07 10-yr: 3.09 100-yr: 4.47	2-yr: 0.39 10-yr: 1.57 100-yr: 2.78
C-D STA 16+00 to 31+50	4.28 ac CN: 94	2-yr: 0.95 10-yr: 1.89 100-yr: 3.33	2-yr: 2.48 10-yr: 3.70 100-yr: 5.36	2-yr: 0.47 10-yr: 1.89 100-yr: 3.33
E-F STA 31+50 to 39+50	2.59 ac CN: 96	2-yr: 0.57 10-yr: 1.14 100-yr: 2.02	2-yr: 1.63 10-yr: 2.37 100-yr: 3.36	2-yr: 0.28 10-yr: 1.14 100-yr: 2.02

Maintenance Road and Rail Spur Basin Runoff Flow Control

As described above, the proposed pipeline maintenance road and rail spur basins include gravel surfaces which will be open graded aggregate base. Therefore, the runoff from these areas is expected to mimic drainage patterns from pervious ground surfacing. No flow control is required since this runoff will be similar intensity to pre-development drainage patterns.

Erosion protection will be provided to prevent sediment transport and flow channelizing off the gravel areas.

Main Plant Stormwater and Oily Water Sewer Basins Flow Control

Runoff from the proposed main plant drainage basins will be piped to the proposed treatment facilities located at the north side of the site. The treatment systems will be sized to handle incoming flows from the drainage basins, provide treatment, and discharge to the existing Port Westward stormwater outfall in accordance with the Port's regulatory requirements. Since the main plant basins will not discharge directly to the wetland or surface drainage, runoff flow control limits will be based on the Port's outfall capacity, not the Columbia County Stormwater Ordinance standards. Details of the main plant basin treatment and pumping systems will be provided with final design. The following summarizes the expected peak runoff flows from the basins. Runoff calculations are presented in Appendix D of this report.

Table 5: Main Plant Basins Runoff Flow Summary

Basin	Drainage Discharge Point	Peak Developed Runoff Flow Rate (cfs)
Oily Water Sewer	On-Site Wastewater Treatment Plant	2-yr: 18.76 10-yr: 30.71 100-yr: 47.32
Plant Stormwater	On-Site Stormwater Treatment System	2-yr: 21.03 10-yr: 35.80 100-yr: 56.71
Total Plant Drainage Basin	Treated Runoff Discharged to Port Westward Storm[1]	2-yr: 40.05 10-yr: 66.85 100-yr: 104.42

[1] Peak runoff flow reported for the total site is the drainage runoff. Discharge to the Port outfall system will be determined based on final design of the treatment plant and pumped discharge facilities.

VI. CONVEYANCE SIZING

The proposed development will include stormwater conveyance to separate discharge points for each of the generalized drainage basins across the site. The following summarizes the expected conveyance system, design storm parameters, and preliminary sizing.

- **Access Road Basin**: stormwater runoff will be conveyed through the swale along the south side of the laydown yard. Preliminary swale sizing is provided in Appendix C of this report. The swale will discharge to the adjacent drainage ditch at three locations along the length of the access road. The Access Road basin comprises approximately 10.44 acres, so the 10-year design storm is used for conveyance sizing.

- **Pipeline Maintenance and Rail Spur Basins**: stormwater runoff from the pervious gravel surfaces will be non-concentrated sheet flow which is expected to follow existing drainage paths to the nearby ditches. No specific conveyance system sizing is required for these drainage areas.

- **Oily Water Sewer Basin:** the oily water sewer drainage basin runoff will be conveyed via pipe to the on-site wastewater treatment plant near the north side of the site. The conveyance system is expected to include at least one lift station in the southeast portion of the site to pump drainage from the outlying Hydrogen Plant and Ecofining Units and reduce the overall pipe depth approaching the wastewater treatment plant. The oily water sewer basin comprises approximately 45.16 acres, so the conveyance design storm is the 100-year event. Gravity storm pipes in this basin are expected to range from 18” to 36” diameter.

- **Main Plant Stormwater Basin:** the plant stormwater drainage basin runoff will be conveyed via pipe to the on-site stormwater treatment plant near the north side of the site. The conveyance system is expected to include at least one lift station near the control building to pump drainage from the outlying Office, Warehouse, and Parking areas. The plant stormwater basin comprises approximately 57.30 acres, so the conveyance design storm is the 100-year event. Gravity storm pipes in this basin are expected to range from 18” to 36” diameter.

VII. OPERATIONS AND MAINTENANCE GUIDELINES

The proposed stormwater treatment systems for the NEXT Renewable Fuels Oregon project include a variety of facilities located across the site. Maintenance of the facilities will be the responsibility of the plant owner and operator, NEXT Renewable Fuels Oregon. The following summarizes typical maintenance requirements for the types of facilities expected to be utilized on site:

- Vegetated Swales: periodic inspection, pruning, debris removal, sediment removal, replanting dead vegetation, irrigation during establishment period

- Filter Strips: periodic inspection, debris removal, sediment removal, replanting dead vegetation, irrigation during establishment period, re-grading channelized areas

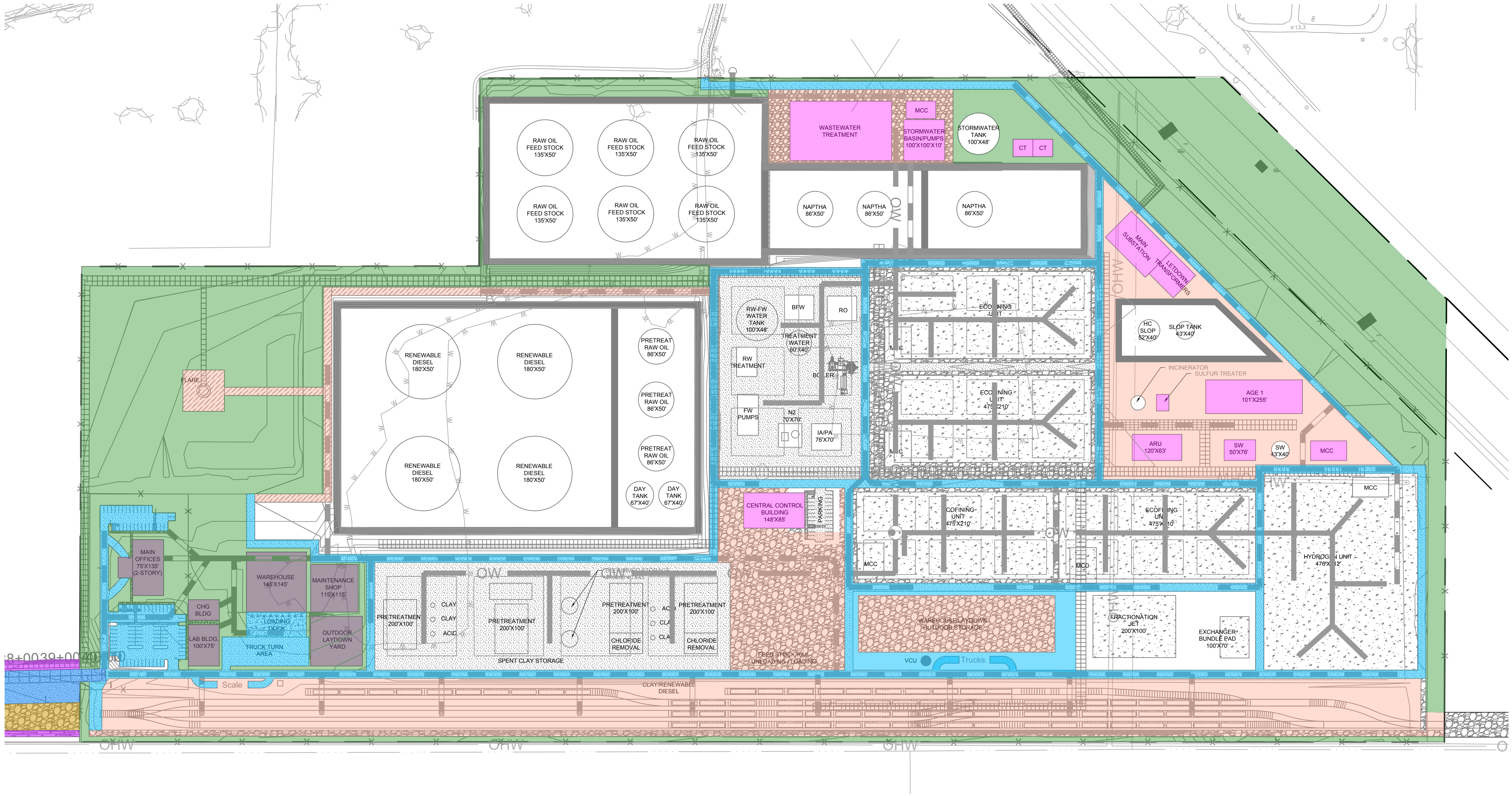
- Catch basins: periodic inspection, sediment removal

- Lift stations: periodic inspection, sediment removal, pump maintenance

Specific operation and maintenance details will be provided with final design of the project.

APPENDIX A

BASIN MAPS AND SITE PLANS

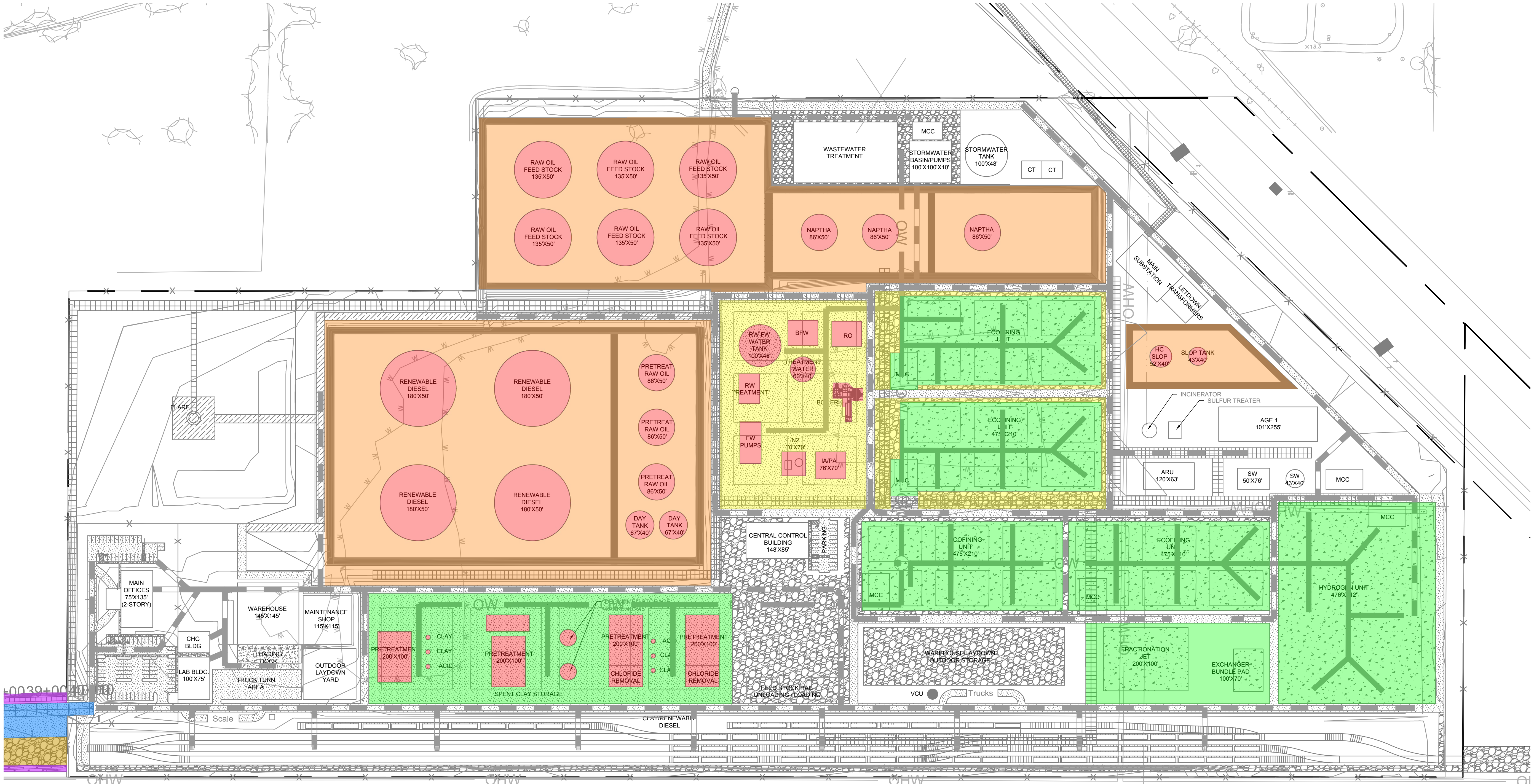


PLANT AREA - STORMWATER DRAINAGE BASIN

1 EX1 1"=250'

	PAVEMENT AREA
	BUILDING AREA
	GRAVEL AREA
	LANDSCAPE AND PERVIOUS SURFACING

PLANT FACILITY STORMWATER BASIN		
SURFACE COVERAGE	TOTAL AREA	CURVE NUMBER
ASPHALT AND CONCRETE PAVING	8.82 AC	98
BUILDINGS AND TANKS	4.39 AC	98
GRAVEL ROADS AND LAYDOWN YARDS	19.40 AC	92
PERVIOUS GRAVEL AND LANDSCAPING	24.69 AC	78
TOTAL STORMWATER BASIN AREA	57.30 AC	87.4 COMPOSITE



1 EX2 PLANT AREA - OIL-WATER SEWER DRAINAGE BASIN

1"=250'

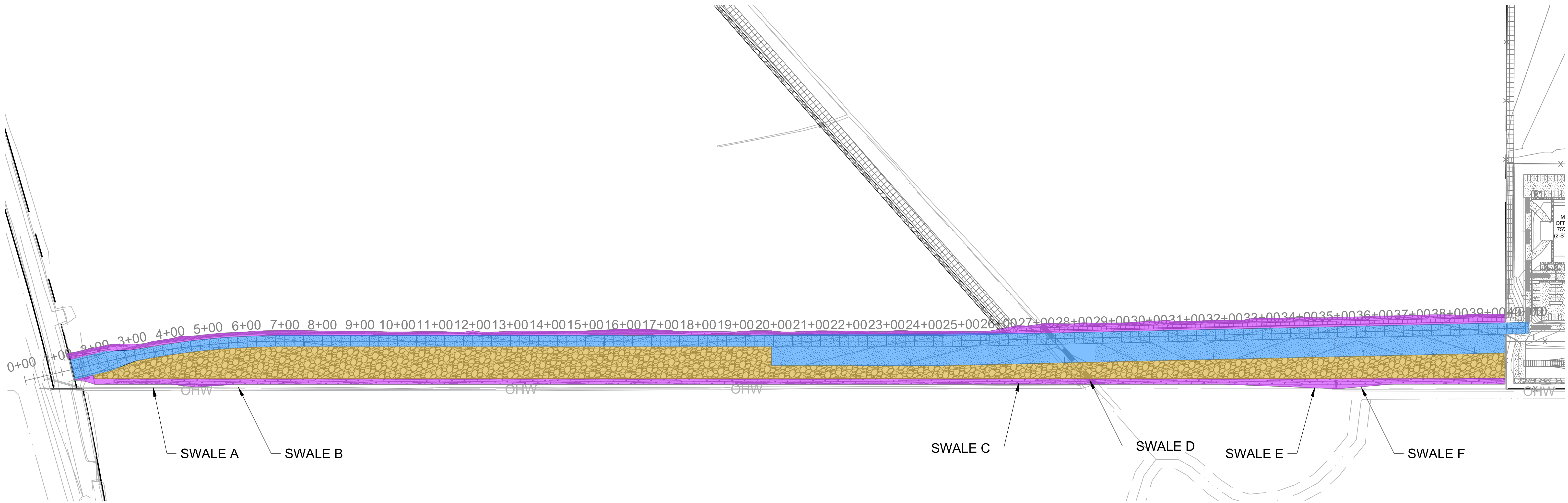
PAVEMENT AREA

BUILDING AREA

GRAVEL AREA

LANDSCAPE AND PERVIOUS SURFACING

PLANT FACILITY OILY WATER SEWER BASIN		
SURFACE COVERAGE	TOTAL AREA	CURVE NUMBER
ASPHALT AND CONCRETE PAVING	19.11 AC	98
BUILDINGS AND TANKS	2.02 AC	98
GRAVEL ROADS AND LAYDOWN YARDS	4.46 AC	92
PERVIOUS GRAVEL AND LANDSCAPING	19.57 AC	78
TOTAL STORMWATER BASIN AREA	45.16 AC	88.7 COMPOSITE



1 EX3 ACCESS ROAD DRAINAGE BASIN
1"=250'

PAVEMENT AREA

GRAVEL AREA

LANDSCAPE AND PERVIOUS SURFACING

ACCESS ROAD BASIN		
SURFACE COVERAGE	TOTAL AREA	CURVE NUMBER
ASPHALT AND CONCRETE PAVING	4.00 AC	98
GRAVEL ROADS AND LAYDOWN YARDS	6.37 AC	92
PERVIOUS GRAVEL AND LANDSCAPING	2.83 AC	78
TOTAL ACCESS ROAD BASIN AREA	13.20 AC	90.8 COMPOSITE

LEGEND:

SEE LEGEND ON SHEET C0.01

1 V1.10 EXISTING SITE PLAN - WEST

100 0 50 100 200 400

(IN FEET)
1 inch = 100 ft.

M.

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Portland, OR
503.224.9560
Vancouver, WA
360.695.7879
Seattle, WA
206.749.9993
www.mcknze.com

MACKENZIE.
DESIGN DRIVEN | CLIENT FOCUSED

Client

NEXT RENEWABLE FUELS OREGON

11767 KATY FREEWAY
SUITE 705
HOUSTON, TX 77079

Project

NEXT RENEWABLE FUELS, INC.
PORT WESTWARD
COLUMBIA COUNTY, OR

REGISTERED PROFESSIONAL ENGINEER
82697PE
OREGON
JUNE 2, 2010
BRENT DANIEL NIELSEN
EXPIRES: 12/31/22

REVISION SCHEDULE		
Delta	Issued As	Issue Date

SHEET TITLE:

EXISTING SITE PLAN WEST

DRAWN BY: CAM

CHECKED BY: BDN

SHEET

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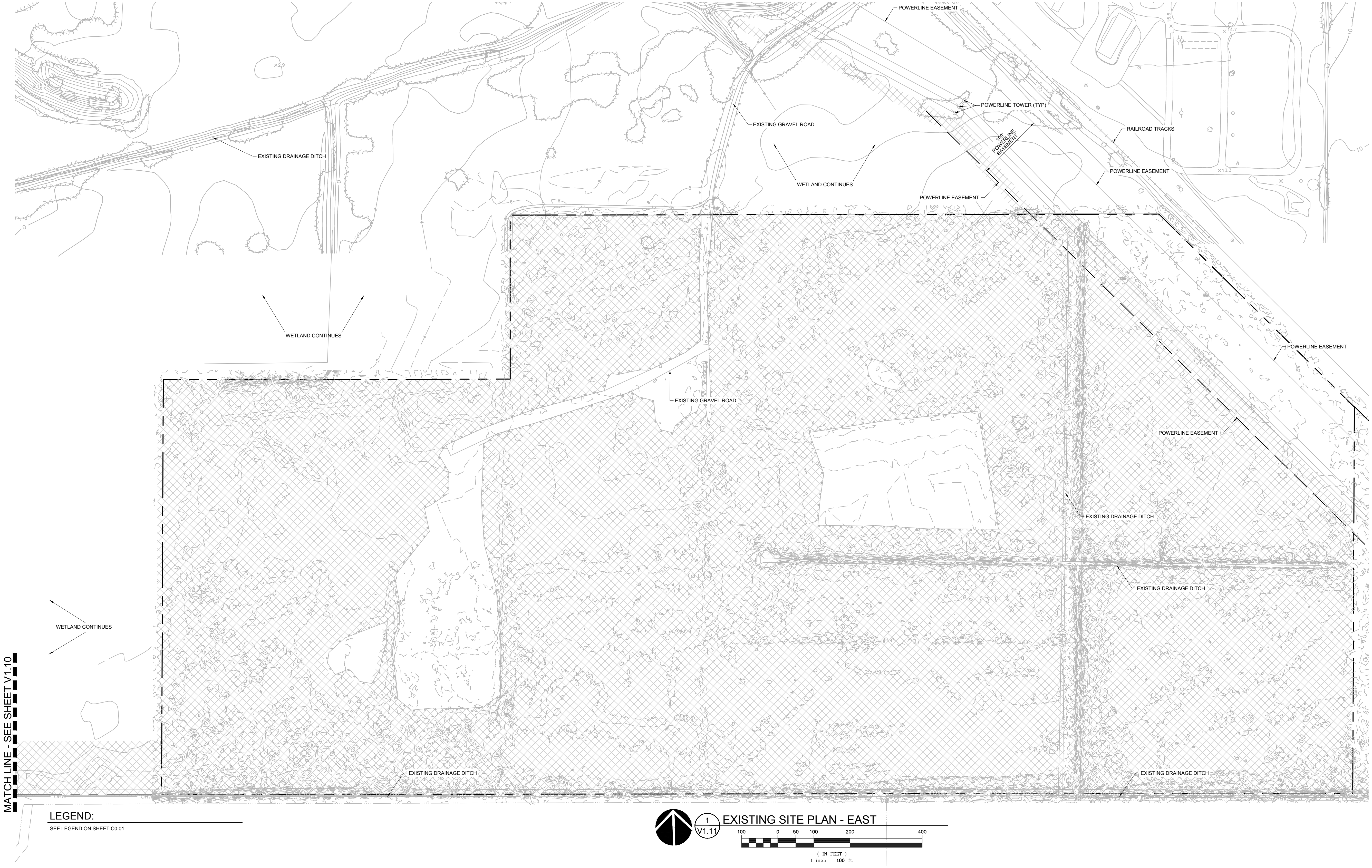

LEGEND:

SEE LEGEND ON SHEET C0.01

1
V1.11
EXISTING SITE PLAN - EAST

100 0 50 100 200 400

(IN FEET)
1 inch = 100 ft.

M.

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Portland, OR
503.224.9560
Vancouver, WA
360.695.7879
Seattle, WA
206.749.9993
www.mcknze.com

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DESIGN DRIVEN I CLIENT FOCUSED

Client

NEXT RENEWABLE FUELS OREGON

11767 KATY FREEWAY
SUITE 705
HOUSTON, TX 77079

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REVISION SCHEDULE

Delta	Issued As	Issue Date

SHEET TITLE:

EXISTING SITE PLAN EAST

DRAWN BY: CAM

CHECKED BY: BDN

SHEET

V1.11

JOB NO. **2200315.01**

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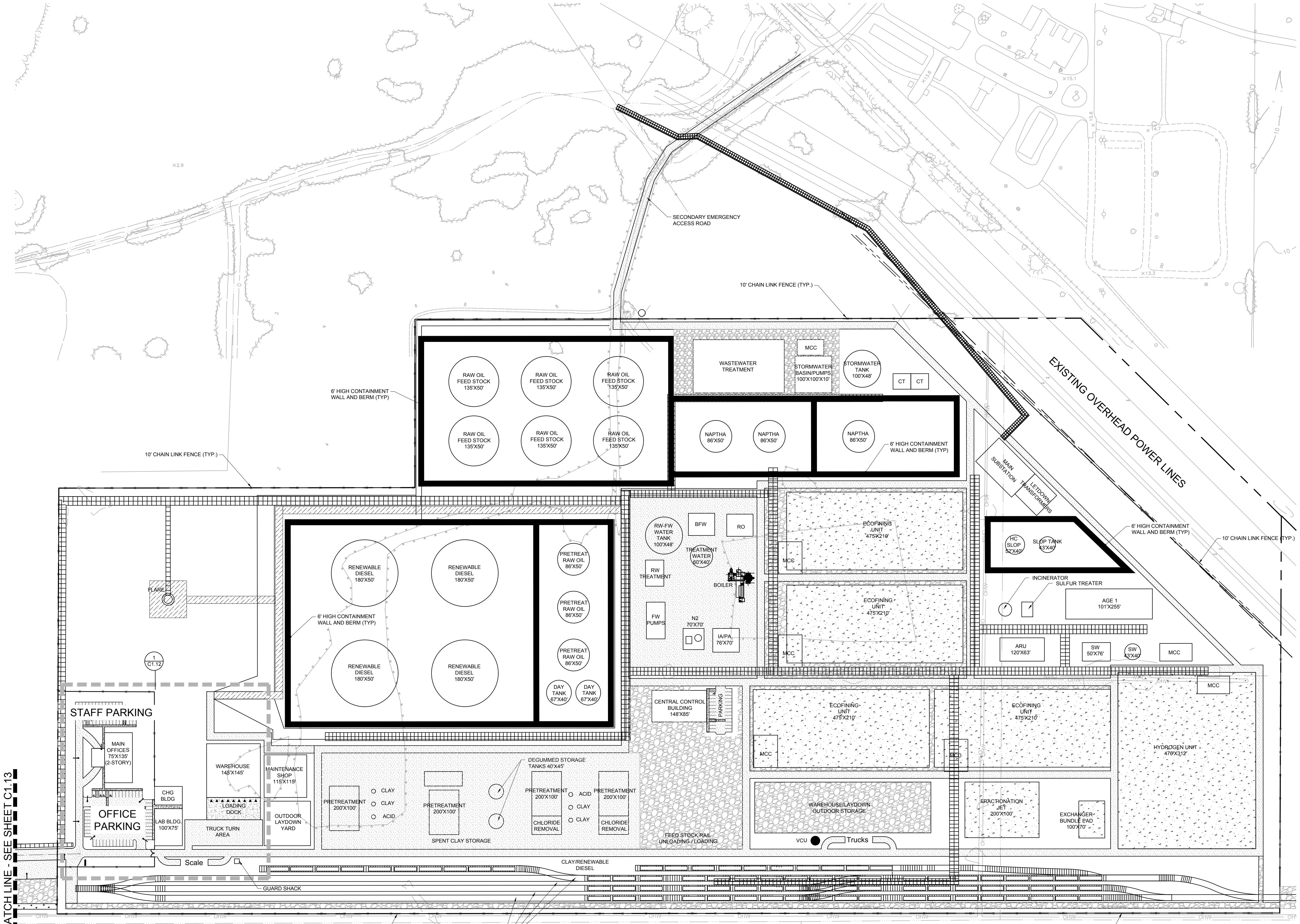

LEGEND:

SEE LEGEND ON SHEET C0.01

10' HIGH CHAIN LINK FENCE LOCATED OUTSIDE OF DITCH SLOPE(TYP.)

RAIL SPUR

1 C1.11 PROPOSED SITE PLAN

100 0 50 100 200 400

(IN FEET)
1 inch = 100 ft.

10' HIGH CHAIN LINK FENCE LOCATED OUTSIDE OF DITCH SLOPE(TYP.)

PARKING SUMMARY:

	STANDARD SPACES	ACCESSIBLE SPACES	TOTAL SPACES
OFFICE PARKING	68	4	65
STAFF PARKING	28	1	29
CONTROL BLDG PARKING	23	1	24
TOTAL PARKING	119	6	118

BUILDING AREA SUMMARY:

MAIN OFFICE:	20,250 SF
LAB:	7,500 SF
CHANGE BUILDING:	3,750 SF
WAREHOUSE:	21,025 SF
MAINTENANCE:	13,335 SF
CENTRAL CONTROL :	12,580 SF
TOTAL BUILDING FLOOR AREA:	78,330 SF

SITE COVERAGE SUMMARY:

TOTAL SITE AREA:	5,550,980 SF	(127.43 AC)
PROPOSED ASPHALT AREA:	831,245 SF	(19.08 AC)
PROPOSED CONCRETE AREA:	626,056 SF	(14.37 AC)
PROPOSED GRAVEL ROAD AREA:	223,768 SF	(5.13 AC)
PROPOSED BUILDING AREA:	276,163 SF	(6.34 AC)
PROPOSED TANK AREA:	257,214 SF	(5.90 AC)
PROPOSED IMPERVIOUS AREA:	2,214,446 SF	(50.83 AC)
PROPOSED GRAVEL AREA:	661,387 SF	(15.18 AC)
PROPOSED LANDSCAPE AREA:	1,561,433 SF	(35.84 AC)
PROPOSED PERVIOUS AREA:	2,222,820 SF	(51.02 AC)
SITE AREA OUTSIDE DEVELOPMENT:	1,113,714	(25.58 AC)

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OREGON
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Delta	Issued As	Issue Date

SHEET TITLE:

PROPOSED SITE PLAN

DRAWN BY: CAM

CHECKED BY: BDN

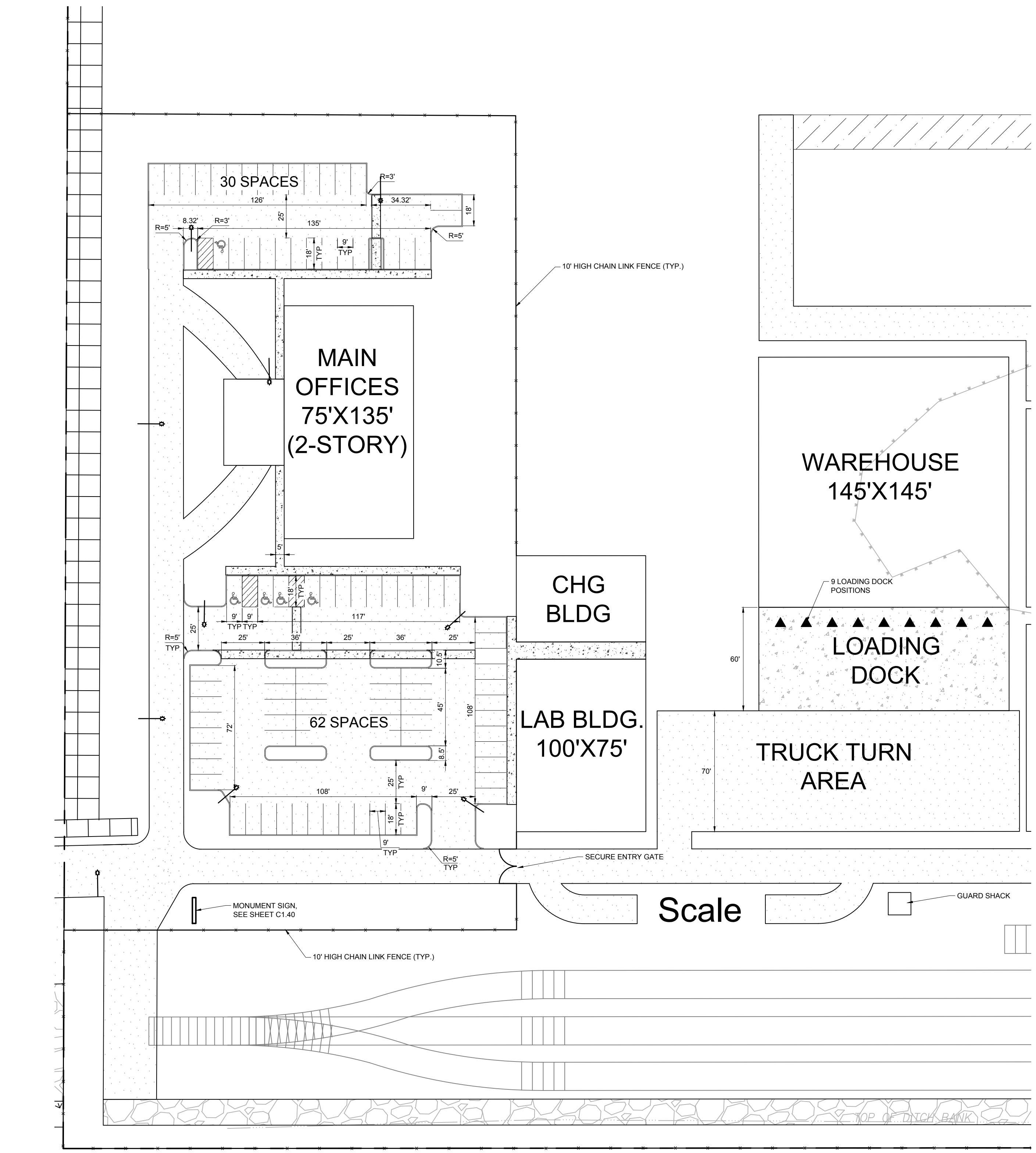
SHEET

C1.11

JOB NO. 2200315.01

NEXT PERMIT DRAWING SET - 1/8/2021

220031500\DRAWINGS\CIVIL\315-C1.11.DWG BDN 01/08/21 11:07 1:100



LEGEND:
SEE LEGEND ON SHEET C0.01

1 C1.12 MAIN PARKING AND OFFICE AREA

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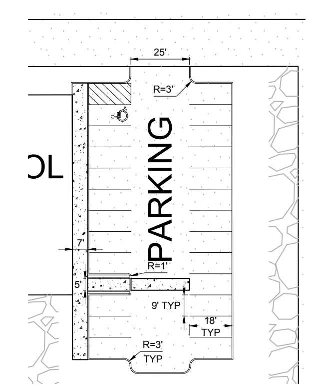
(IN FEET)
1 inch = 30 ft.



2 C1.12 CENTRAL PARKING AREA

30 0 15 30 60 120
(IN FEET)
1 inch = 30 ft.

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Delta	Issued As	Issue Date

SHEET TITLE:
ACCESS, PARKING, AND CIRCULATION PLAN

DRAWN BY: CAM
CHECKED BY: BDN
SHEET
C1.12

JOB NO. 2200315.01

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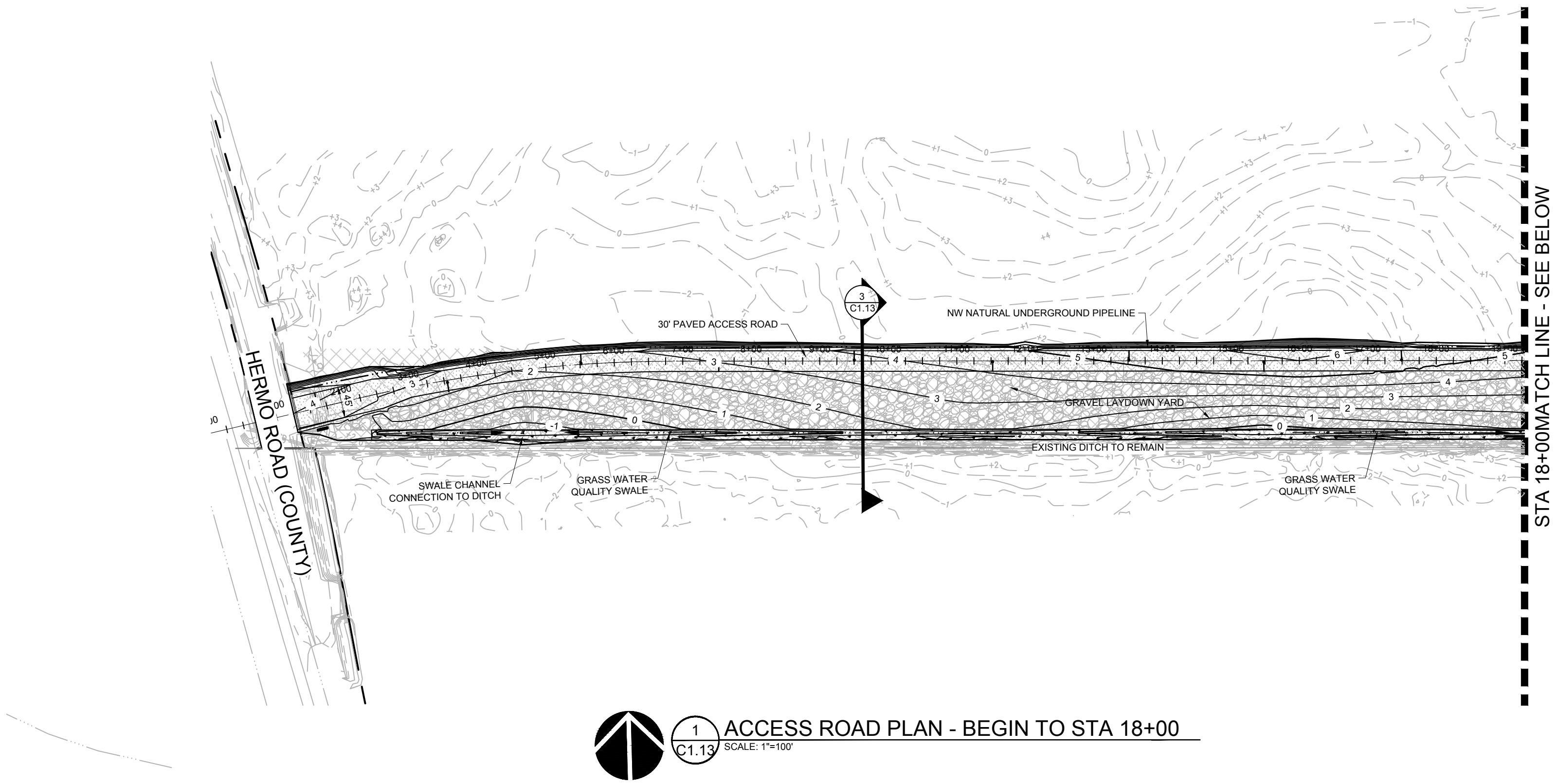
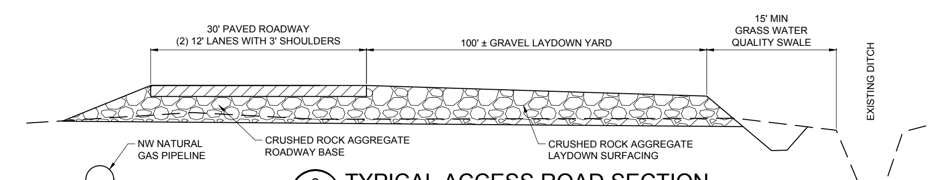


1 C1.13 ACCESS ROAD PLAN - BEGIN TO STA 18+00
SCALE: 1"=100'



3 C1.13 TYPICAL ACCESS ROAD SECTION
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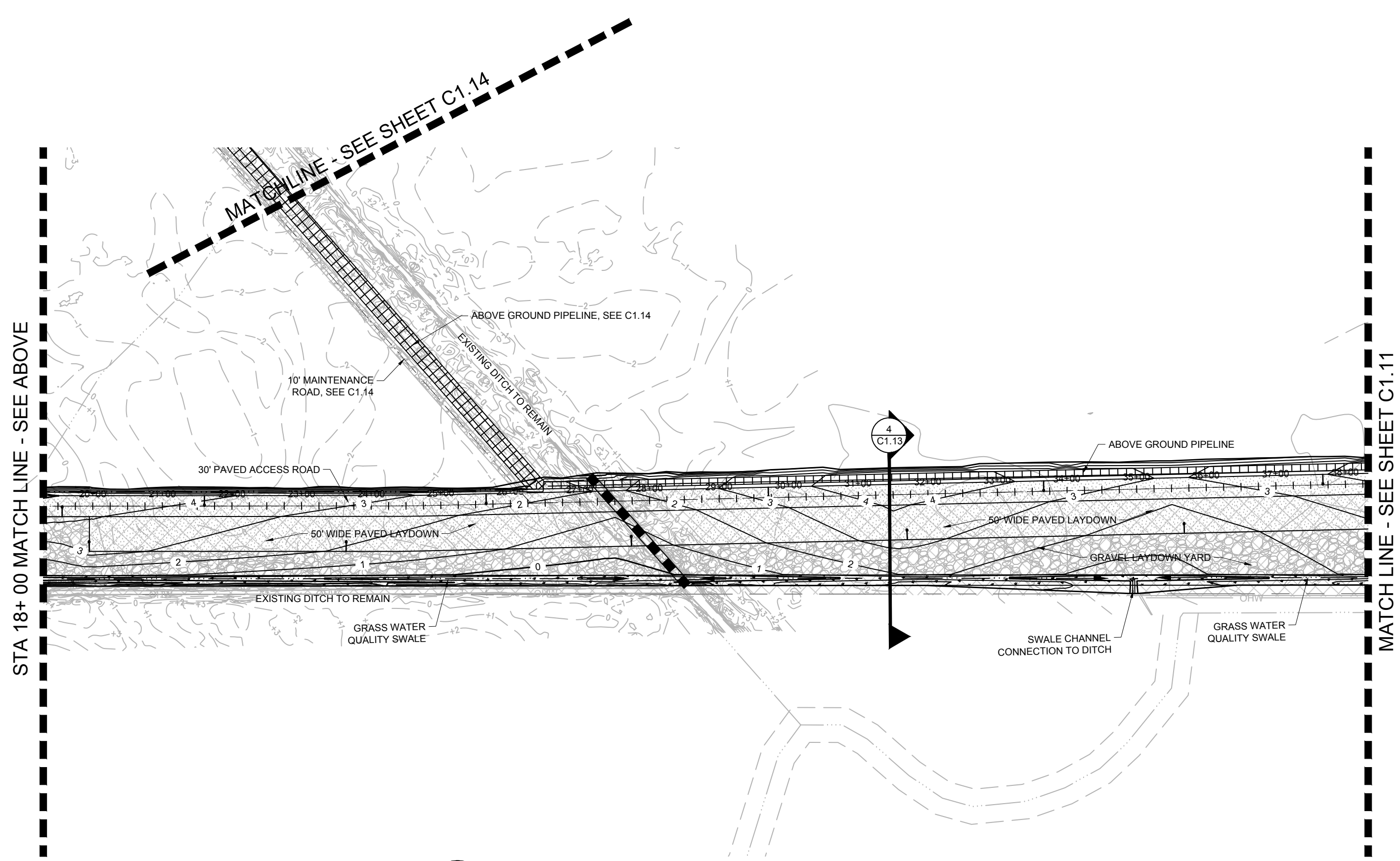
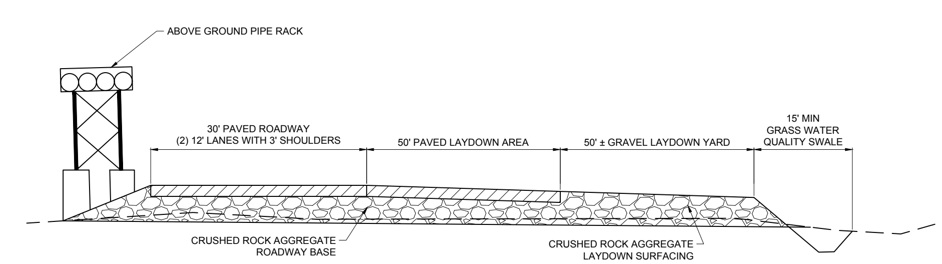


2 C1.13 ACCESS ROAD PLAN - STA 18+00 TO STA 37+00
SCALE: 1"=100'



4 C1.13 TYPICAL ACCESS ROAD SECTION
NOT TO SCALE

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SHEET TITLE:
ACCESS ROAD PLAN

DRAWN BY: BMR
CHECKED BY: BDN
SHEET

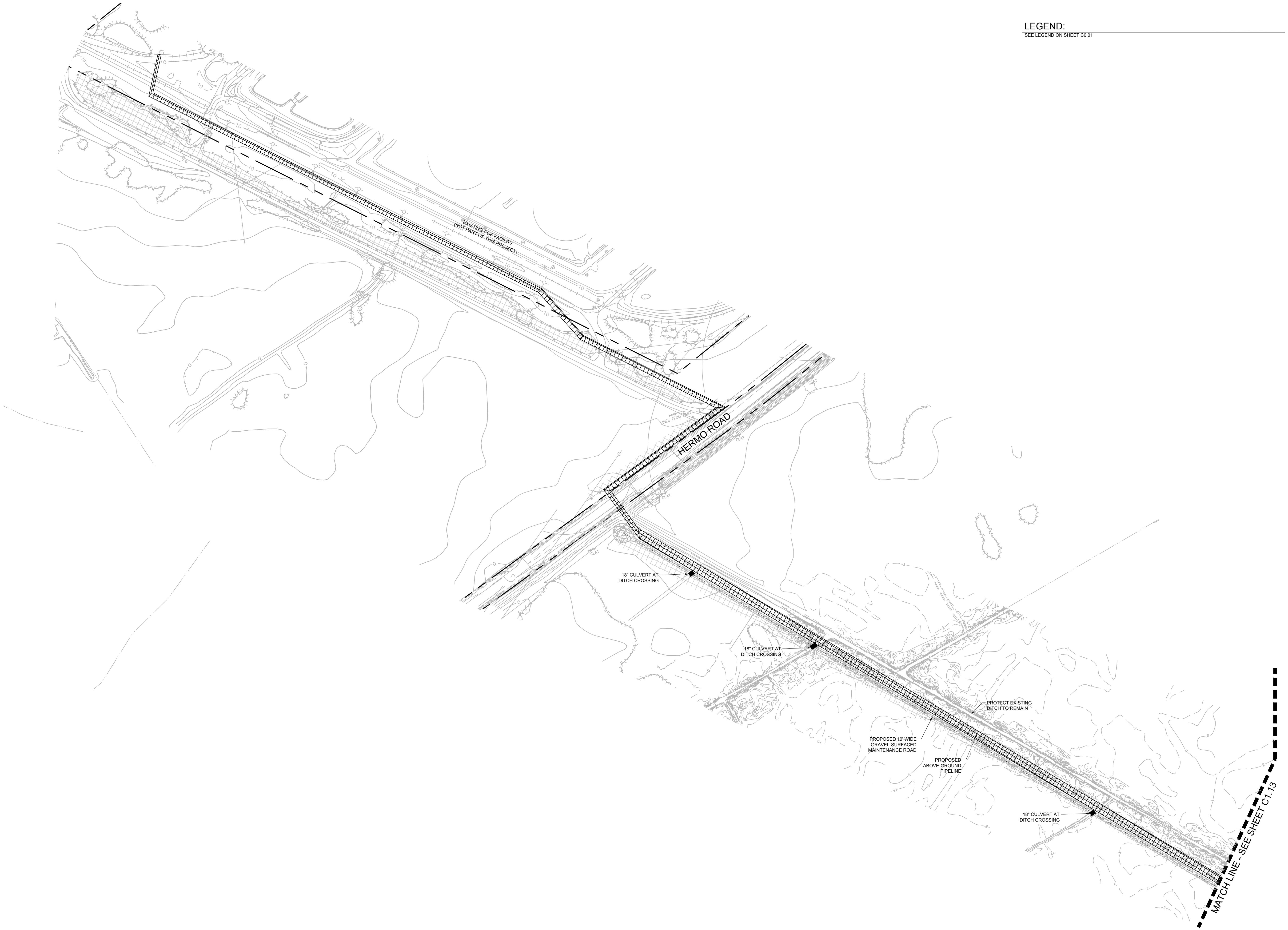
C1.13

JOB NO. 2200315.01

NEXT PERMIT DRAWING SET - 1/8/2021
220031500\DRAWINGS\CIVIL\315-C1.13.DWG BDN 01/08/21 11:09 1:100.00

LEGEND:

SEE LEGEND ON SHEET C0.01



1
C1.14
PIPELINE ALIGNMENT PLAN
SCALE: 1"=100'

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Delta	Issued As	Issue Date

SHEET TITLE:

PIPELINE
ALIGNMENT
PLAN

DRAWN BY: BMR

CHECKED BY: BDN

SHEET

C1.14

JOB NO. 2200315.01

NEXT PERMIT DRAWING SET - 1/8/2021

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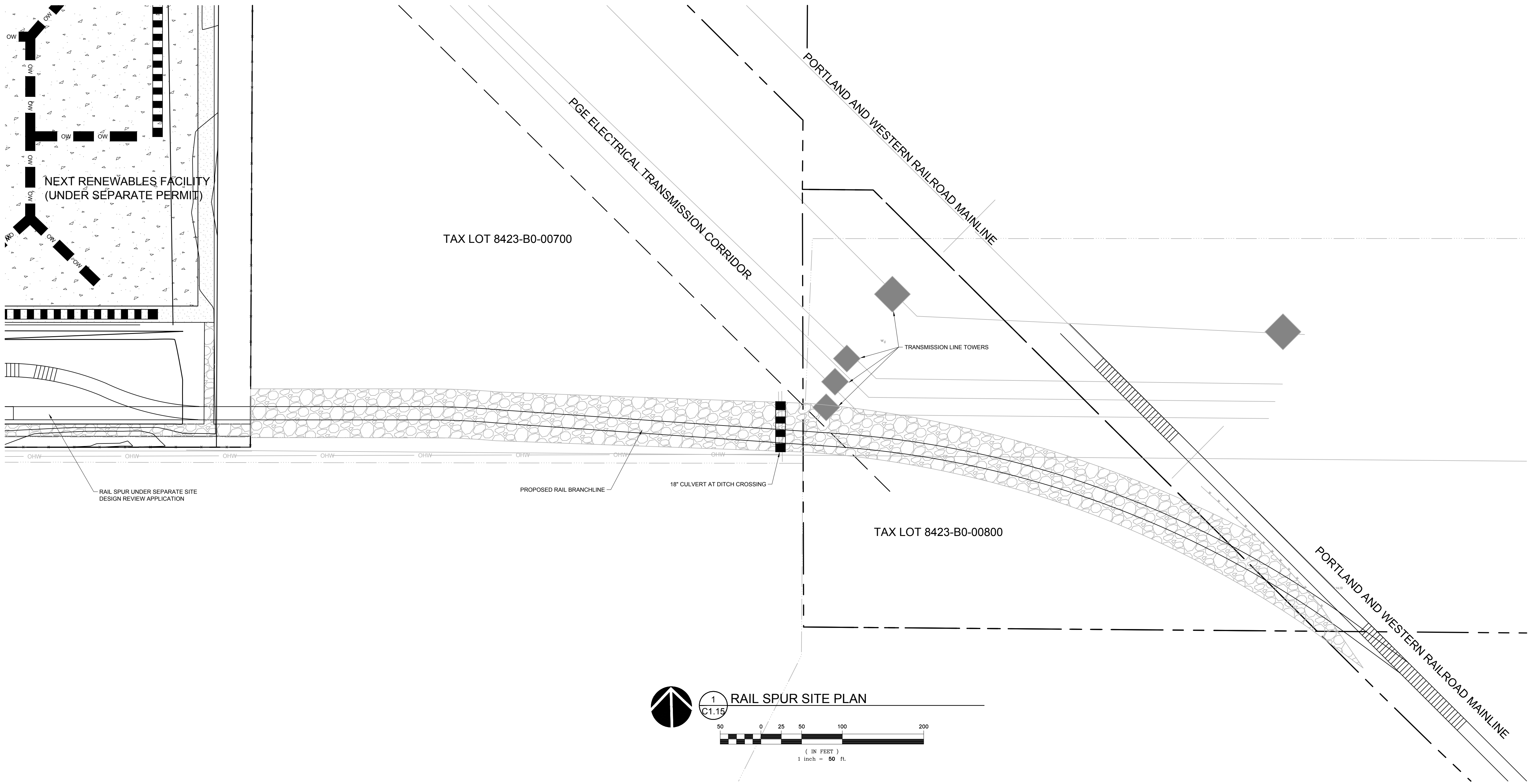

1
C1.15
RAIL SPUR SITE PLAN

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SHEET TITLE:
RAIL SPUR
SITE PLAN

DRAWN BY: CAM
CHECKED BY: BDN
SHEET

C1.15

JOB NO. 2200315.01

NEXT PERMIT DRAWING SET - 1/8/2021

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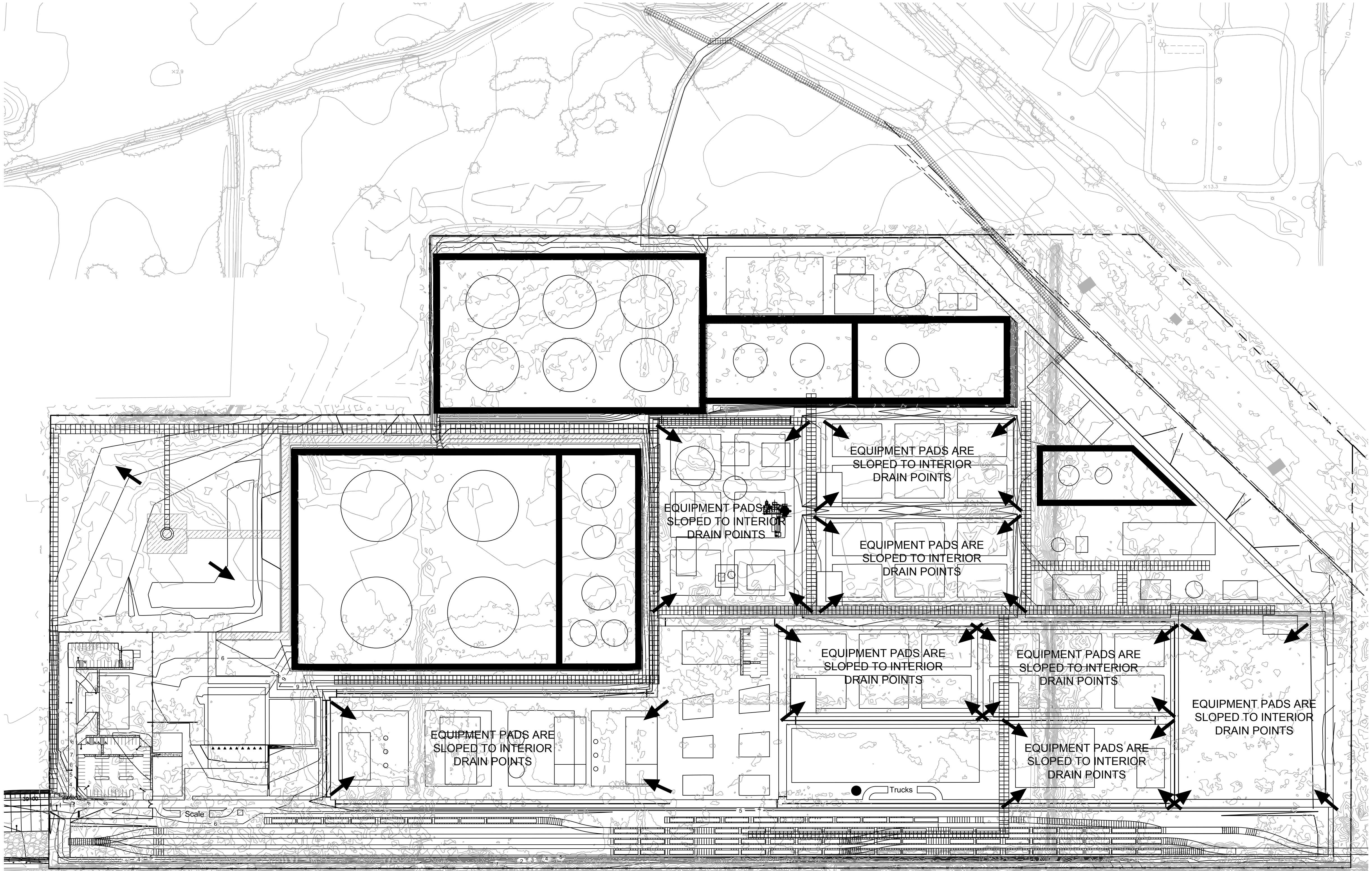

LEGEND:

SEE LEGEND ON SHEET C0.01

1 C1.20 GRADING PLAN

100 0 50 100 200 400

(IN FEET)
1 inch = 100 ft.

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SHEET TITLE:
GRADING PLAN

DRAWN BY: CAM

CHECKED BY: BDN

SHEET

C1.20

JOB NO. 2200315.01

NEXT PERMIT DRAWING SET - 1/8/2021

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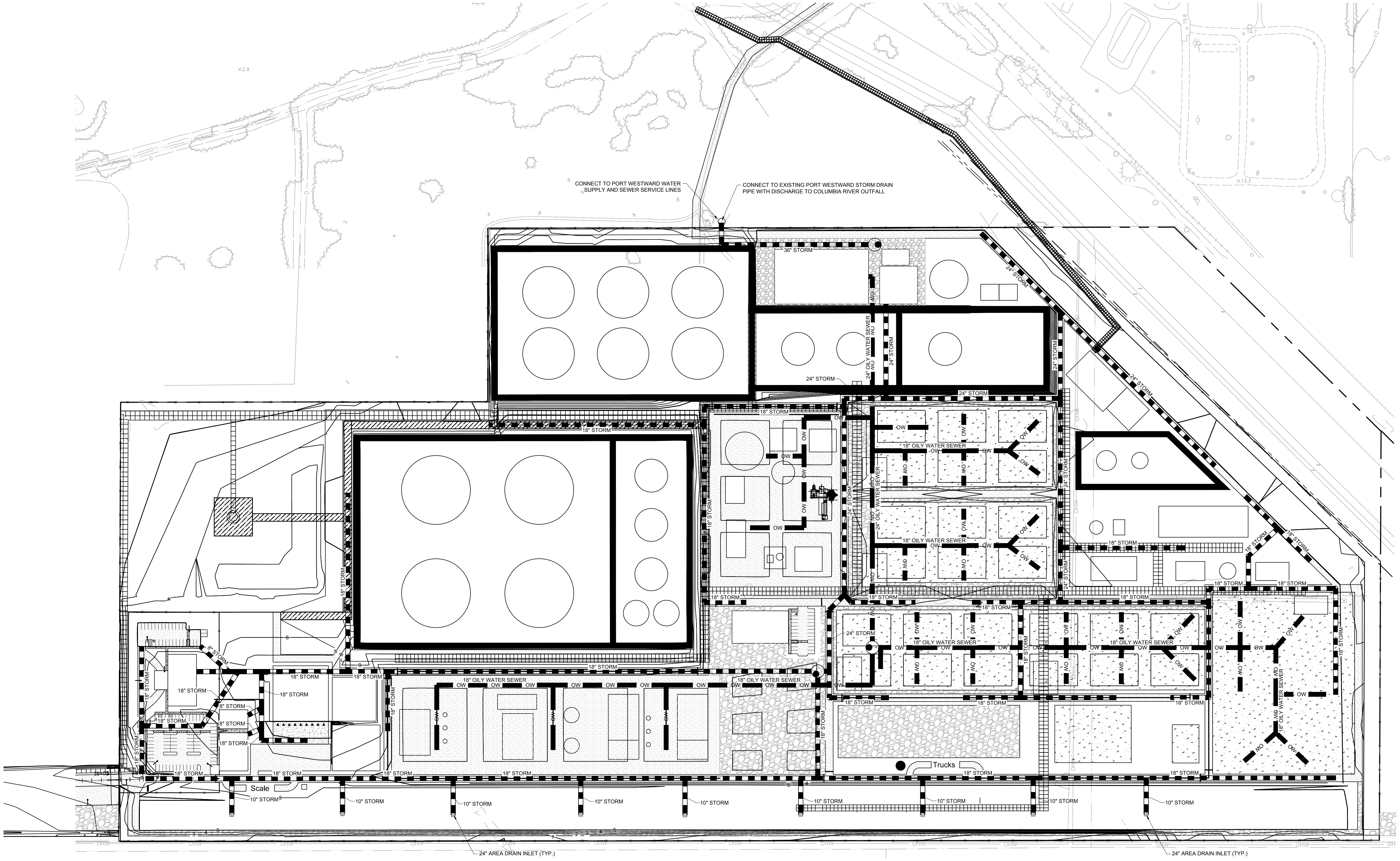

LEGEND:

REFER TO SHEET C0.01 FOR GENERAL LEGEND

1 C1.30 DRAINAGE PLAN

100 0 50 100 200 400

(IN FEET)
1 inch = 100 ft.

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SHEET TITLE:
DRAINAGE PLAN

DRAWN BY: CAM
CHECKED BY: BDN
SHEET

C1.30

JOB NO. 2200315.01

NEXT PERMIT DRAWING SET - 1/8/2021
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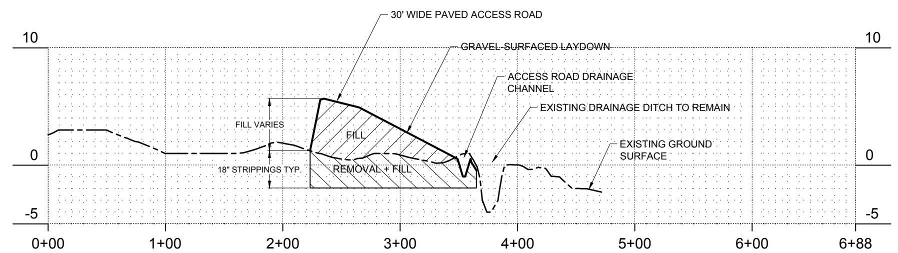
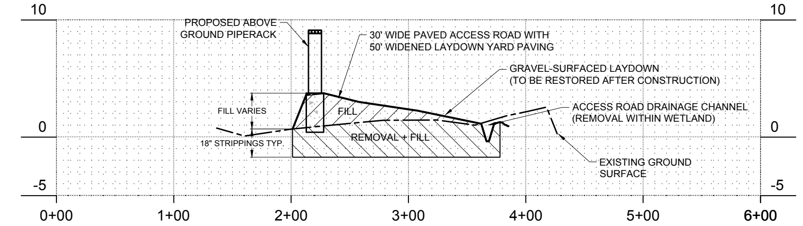
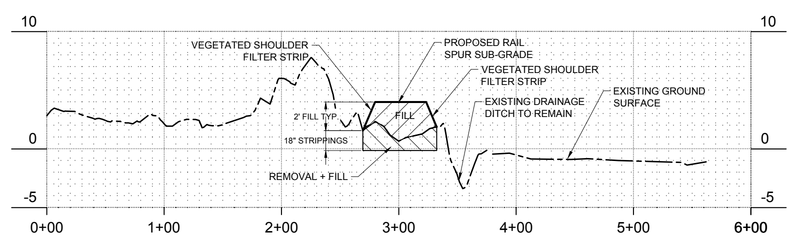


1 C2.23 ACCESS ROAD CROSS-SECTION
HORIZONTAL SCALE = 1"=60'
VERTICAL SCALE = 1"=6'



2 C2.23 ACCESS ROAD CROSS SECTION WITH WIDENED PAVING
HORIZONTAL SCALE = 1"=60'
VERTICAL SCALE = 1"=6'



3 C2.23 RAIL SPUR BRANCH LINE TYPICAL CROSS-SECTION
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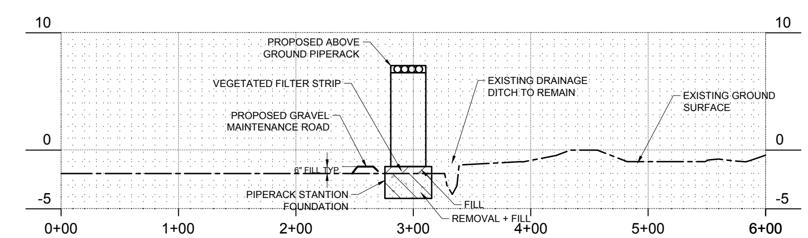
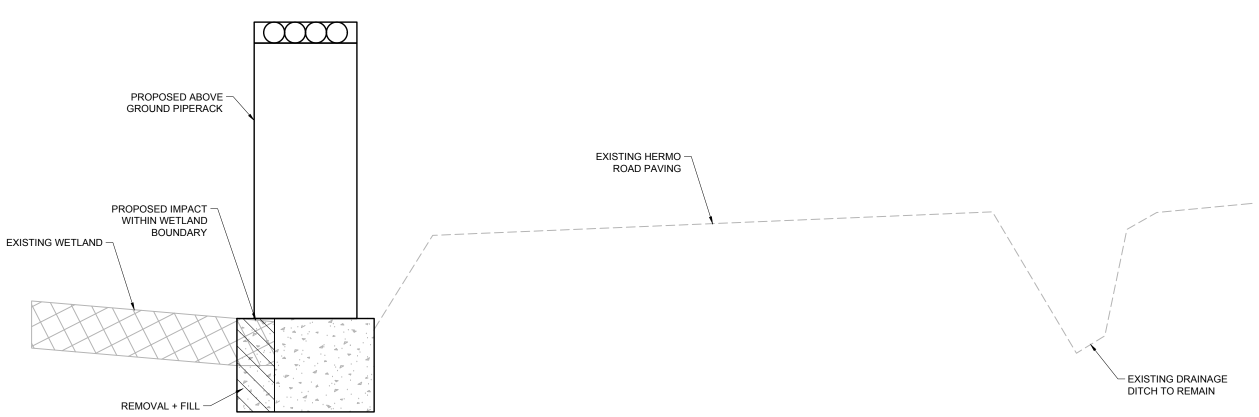
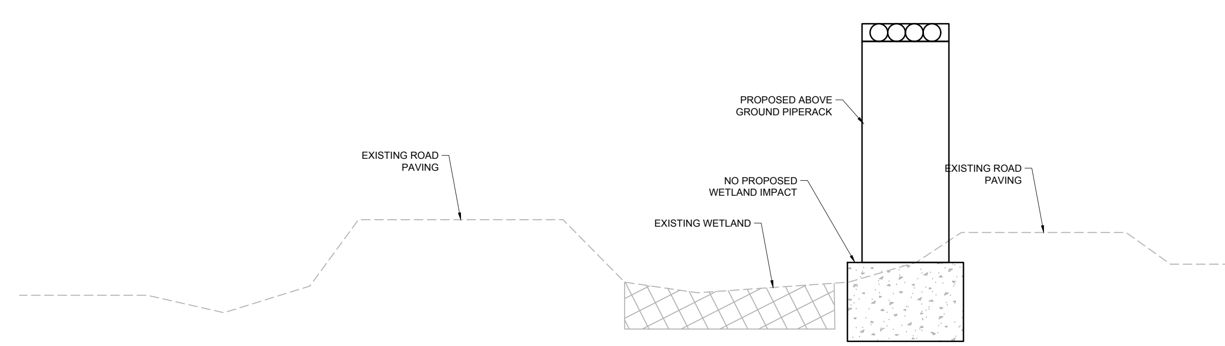
VERTICAL SCALE = 1"=6'



4 C2.23 PIPELINE MAINTENANCE ROAD TYPICAL CROSS-SECTION
HORIZONTAL SCALE = 1"=60'
VERTICAL SCALE = 1"=6'



5 C2.23 TYPICAL CROSS-SECTION OF PIPERACK ALONG HERMO ROAD
NOT TO SCALE



6 C2.23 TYPICAL CROSS-SECTION OF PIPERACK ALONG SERVICE ROAD
NOT TO SCALE

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SHEET TITLE:
WETLAND IMPACTS - TYPICAL SECTIONS

DRAWN BY: CAM
CHECKED BY: BDN
SHEET
C2.23

JOB NO. 2200315.01

APPENDIX B

SOIL SURVEY REPORT

United States Department of Agriculture

NRCS

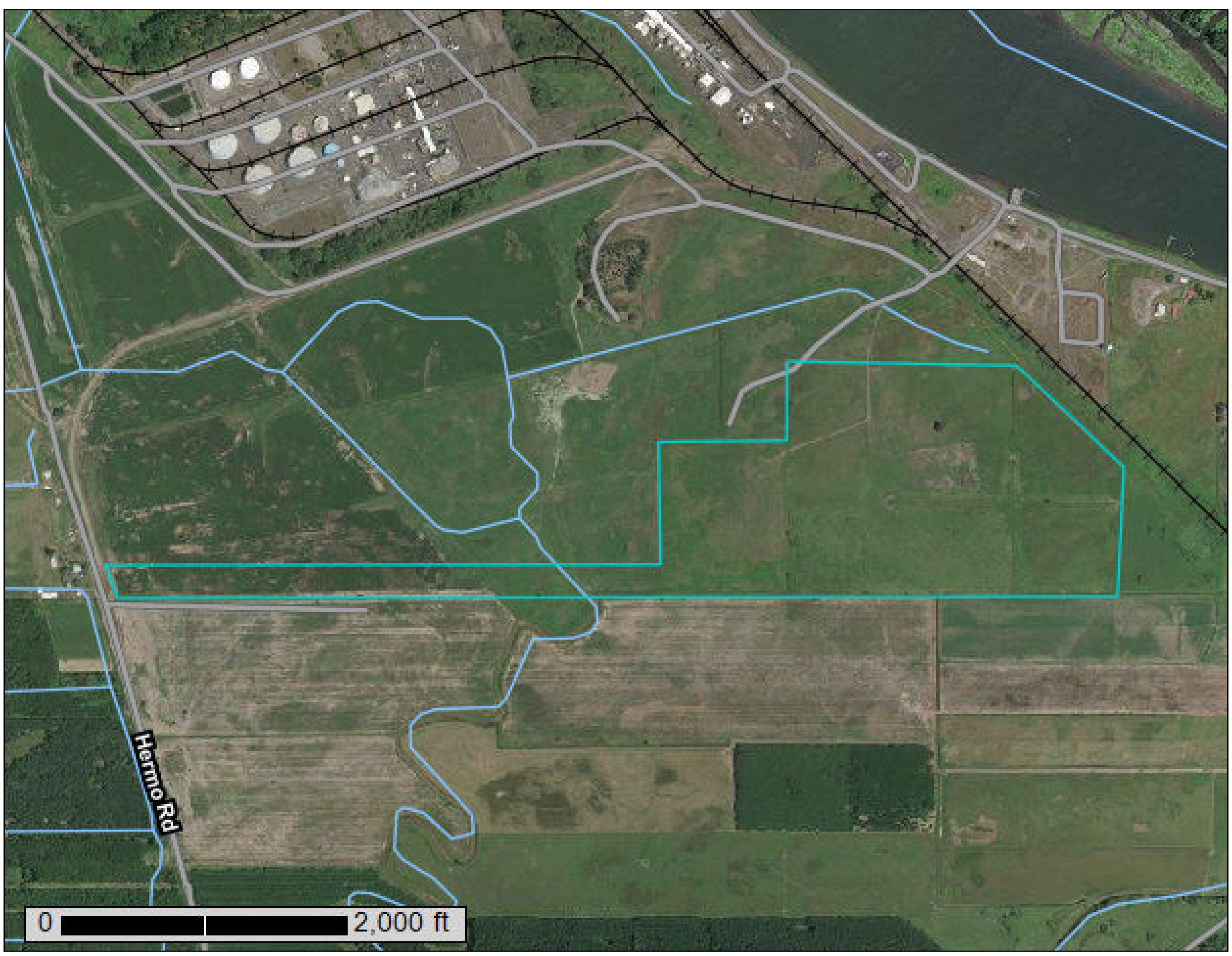
Natural Resources Conservation Service

A product of the National Cooperative Soil Survey, a joint effort of the United States Department of Agriculture and other Federal agencies, State agencies including the Agricultural Experiment Stations, and local participants

Custom Soil Resource Report for

Columbia County, Oregon

NEXT Renewables



December 8, 2020

Preface

Soil surveys contain information that affects land use planning in survey areas. They highlight soil limitations that affect various land uses and provide information about the properties of the soils in the survey areas. Soil surveys are designed for many different users, including farmers, ranchers, foresters, agronomists, urban planners, community officials, engineers, developers, builders, and home buyers. Also, conservationists, teachers, students, and specialists in recreation, waste disposal, and pollution control can use the surveys to help them understand, protect, or enhance the environment.

Various land use regulations of Federal, State, and local governments may impose special restrictions on land use or land treatment. Soil surveys identify soil properties that are used in making various land use or land treatment decisions. The information is intended to help the land users identify and reduce the effects of soil limitations on various land uses. The landowner or user is responsible for identifying and complying with existing laws and regulations.

Although soil survey information can be used for general farm, local, and wider area planning, onsite investigation is needed to supplement this information in some cases. Examples include soil quality assessments (http://www.nrcs.usda.gov/wps/portal/nrcs/main/soils/health/) and certain conservation and engineering applications. For more detailed information, contact your local USDA Service Center (https://offices.sc.egov.usda.gov/locator/app?agency=nrcs) or your NRCS State Soil Scientist (http://www.nrcs.usda.gov/wps/portal/nrcs/detail/soils/contactus/?cid=nrcs142p2_053951).

Great differences in soil properties can occur within short distances. Some soils are seasonally wet or subject to flooding. Some are too unstable to be used as a foundation for buildings or roads. Clayey or wet soils are poorly suited to use as septic tank absorption fields. A high water table makes a soil poorly suited to basements or underground installations.

The National Cooperative Soil Survey is a joint effort of the United States Department of Agriculture and other Federal agencies, State agencies including the Agricultural Experiment Stations, and local agencies. The Natural Resources Conservation Service (NRCS) has leadership for the Federal part of the National Cooperative Soil Survey.

Information about soils is updated periodically. Updated information is available through the NRCS Web Soil Survey, the site for official soil survey information.

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Contents

How Soil Surveys Are Made

Soil surveys are made to provide information about the soils and miscellaneous areas in a specific area. They include a description of the soils and miscellaneous areas and their location on the landscape and tables that show soil properties and limitations affecting various uses. Soil scientists observed the steepness, length, and shape of the slopes; the general pattern of drainage; the kinds of crops and native plants; and the kinds of bedrock. They observed and described many soil profiles. A soil profile is the sequence of natural layers, or horizons, in a soil. The profile extends from the surface down into the unconsolidated material in which the soil formed or from the surface down to bedrock. The unconsolidated material is devoid of roots and other living organisms and has not been changed by other biological activity.

Currently, soils are mapped according to the boundaries of major land resource areas (MLRAs). MLRAs are geographically associated land resource units that share common characteristics related to physiography, geology, climate, water resources, soils, biological resources, and land uses (USDA, 2006). Soil survey areas typically consist of parts of one or more MLRA.

The soils and miscellaneous areas in a survey area occur in an orderly pattern that is related to the geology, landforms, relief, climate, and natural vegetation of the area. Each kind of soil and miscellaneous area is associated with a particular kind of landform or with a segment of the landform. By observing the soils and miscellaneous areas in the survey area and relating their position to specific segments of the landform, a soil scientist develops a concept, or model, of how they were formed. Thus, during mapping, this model enables the soil scientist to predict with a considerable degree of accuracy the kind of soil or miscellaneous area at a specific location on the landscape.

Commonly, individual soils on the landscape merge into one another as their characteristics gradually change. To construct an accurate soil map, however, soil scientists must determine the boundaries between the soils. They can observe only a limited number of soil profiles. Nevertheless, these observations, supplemented by an understanding of the soil-vegetation-landscape relationship, are sufficient to verify predictions of the kinds of soil in an area and to determine the boundaries.

Soil scientists recorded the characteristics of the soil profiles that they studied. They noted soil color, texture, size and shape of soil aggregates, kind and amount of rock fragments, distribution of plant roots, reaction, and other features that enable them to identify soils. After describing the soils in the survey area and determining their properties, the soil scientists assigned the soils to taxonomic classes (units). Taxonomic classes are concepts. Each taxonomic class has a set of soil characteristics with precisely defined limits. The classes are used as a basis for comparison to classify soils systematically. Soil taxonomy, the system of taxonomic classification used in the United States, is based mainly on the kind and character of soil properties and the arrangement of horizons within the profile. After the soil scientists classified and named the soils in the survey area, they compared the individual soils with similar soils in the same taxonomic class in other areas so that they could confirm data and assemble additional data based on experience and research.

The objective of soil mapping is not to delineate pure map unit components; the objective is to separate the landscape into landforms or landform segments that have similar use and management requirements. Each map unit is defined by a unique combination of soil components and/or miscellaneous areas in predictable proportions. Some components may be highly contrasting to the other components of the map unit. The presence of minor components in a map unit in no way diminishes the usefulness or accuracy of the data. The delineation of such landforms and landform segments on the map provides sufficient information for the development of resource plans. If intensive use of small areas is planned, onsite investigation is needed to define and locate the soils and miscellaneous areas.

Soil scientists make many field observations in the process of producing a soil map. The frequency of observation is dependent upon several factors, including scale of mapping, intensity of mapping, design of map units, complexity of the landscape, and experience of the soil scientist. Observations are made to test and refine the soil-landscape model and predictions and to verify the classification of the soils at specific locations. Once the soil-landscape model is refined, a significantly smaller number of measurements of individual soil properties are made and recorded. These measurements may include field measurements, such as those for color, depth to bedrock, and texture, and laboratory measurements, such as those for content of sand, silt, clay, salt, and other components. Properties of each soil typically vary from one point to another across the landscape.

Observations for map unit components are aggregated to develop ranges of characteristics for the components. The aggregated values are presented. Direct measurements do not exist for every property presented for every map unit component. Values for some properties are estimated from combinations of other properties.

While a soil survey is in progress, samples of some of the soils in the area generally are collected for laboratory analyses and for engineering tests. Soil scientists interpret the data from these analyses and tests as well as the field-observed characteristics and the soil properties to determine the expected behavior of the soils under different uses. Interpretations for all of the soils are field tested through observation of the soils in different uses and under different levels of management. Some interpretations are modified to fit local conditions, and some new interpretations are developed to meet local needs. Data are assembled from other sources, such as research information, production records, and field experience of specialists. For example, data on crop yields under defined levels of management are assembled from farm records and from field or plot experiments on the same kinds of soil.

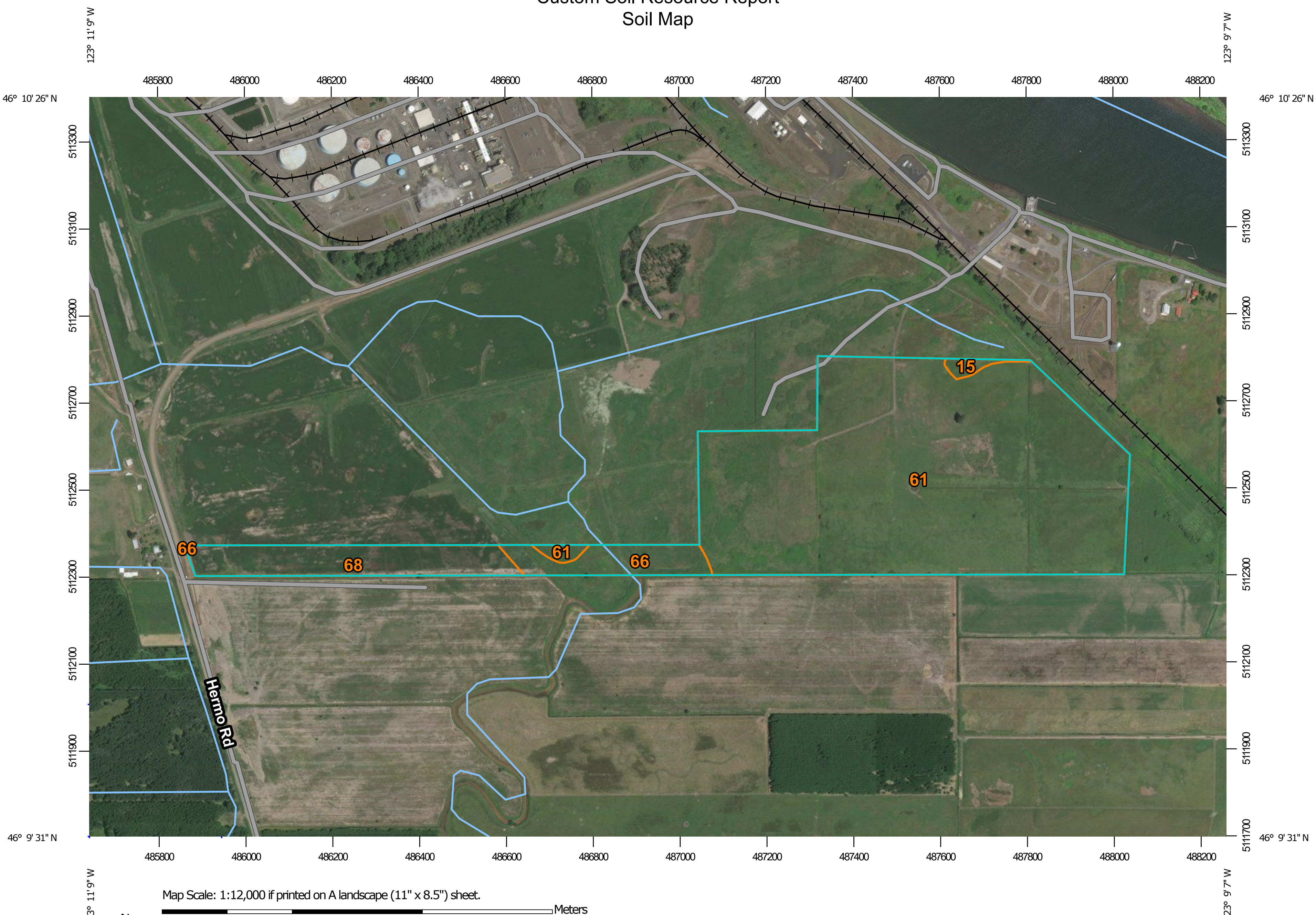
Predictions about soil behavior are based not only on soil properties but also on such variables as climate and biological activity. Soil conditions are predictable over long periods of time, but they are not predictable from year to year. For example, soil scientists can predict with a fairly high degree of accuracy that a given soil will have a high water table within certain depths in most years, but they cannot predict that a high water table will always be at a specific level in the soil on a specific date.

After soil scientists located and identified the significant natural bodies of soil in the survey area, they drew the boundaries of these bodies on aerial photographs and identified each as a specific map unit. Aerial photographs show trees, buildings, fields, roads, and rivers, all of which help in locating boundaries accurately.

Soil Map

The soil map section includes the soil map for the defined area of interest, a list of soil map units on the map and extent of each map unit, and cartographic symbols displayed on the map. Also presented are various metadata about data used to produce the map, and a description of each soil map unit.

Custom Soil Resource Report
Soil Map



Map Scale: 1:12,000 if printed on A landscape (11" x 8.5") sheet.

Meters: 0, 150, 300, 600, 900

Feet: 0, 500, 1000, 2000, 3000

N

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

MAP LEGEND

Area of Interest (AOI)
- Area of Interest (AOI)

Soils
- Soil Map Unit Polygons
- Soil Map Unit Lines
- Soil Map Unit Points

Special Point Features
- Blowout
- Borrow Pit
- Clay Spot
- Closed Depression
- Gravel Pit
- Gravelly Spot
- Landfill
- Lava Flow
- Marsh or swamp
- Mine or Quarry
- Miscellaneous Water
- Perennial Water
- Rock Outcrop
- Saline Spot
- Sandy Spot
- Severely Eroded Spot
- Sinkhole
- Slide or Slip
- Sodic Spot
- Spoil Area
- Stony Spot
- Very Stony Spot
- Wet Spot
- Other
- Special Line Features

Water Features
- Streams and Canals

Transportation
- Rails
- Interstate Highways
- US Routes
- Major Roads
- Local Roads

Background
- Aerial Photography

MAP INFORMATION

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

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

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

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

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

Soil Survey Area: Columbia County, Oregon
Survey Area Data: Version 17, Jun 11, 2020

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

Date(s) aerial images were photographed: Apr 16, 2015—Feb 12, 2017

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

Map Unit Legend

Map Unit Symbol	Map Unit Name	Acres in AOI	Percent of AOI
15	Crims silt loam, protected	1.0	0.8%
61	Udipsamments, nearly level, protected	104.1	83.4%
66	Wauna silt loam, protected	7.0	5.6%
68	Wauna-Locoda silt loams, protected	12.9	10.3%
Totals for Area of Interest		**124.9**	**100.0%**

Map Unit Descriptions

The map units delineated on the detailed soil maps in a soil survey represent the soils or miscellaneous areas in the survey area. The map unit descriptions, along with the maps, can be used to determine the composition and properties of a unit.

A map unit delineation on a soil map represents an area dominated by one or more major kinds of soil or miscellaneous areas. A map unit is identified and named according to the taxonomic classification of the dominant soils. Within a taxonomic class there are precisely defined limits for the properties of the soils. On the landscape, however, the soils are natural phenomena, and they have the characteristic variability of all natural phenomena. Thus, the range of some observed properties may extend beyond the limits defined for a taxonomic class. Areas of soils of a single taxonomic class rarely, if ever, can be mapped without including areas of other taxonomic classes. Consequently, every map unit is made up of the soils or miscellaneous areas for which it is named and some minor components that belong to taxonomic classes other than those of the major soils.

Most minor soils have properties similar to those of the dominant soil or soils in the map unit, and thus they do not affect use and management. These are called noncontrasting, or similar, components. They may or may not be mentioned in a particular map unit description. Other minor components, however, have properties and behavioral characteristics divergent enough to affect use or to require different management. These are called contrasting, or dissimilar, components. They generally are in small areas and could not be mapped separately because of the scale used. Some small areas of strongly contrasting soils or miscellaneous areas are identified by a special symbol on the maps. If included in the database for a given area, the contrasting minor components are identified in the map unit descriptions along with some characteristics of each. A few areas of minor components may not have been observed, and consequently they are not mentioned in the descriptions, especially where the pattern was so complex that it was impractical to make enough observations to identify all the soils and miscellaneous areas on the landscape.

The presence of minor components in a map unit in no way diminishes the usefulness or accuracy of the data. The objective of mapping is not to delineate pure taxonomic classes but rather to separate the landscape into landforms or landform segments that have similar use and management requirements. The delineation of such segments on the map provides sufficient information for the development of resource plans. If intensive use of small areas is planned, however, onsite investigation is needed to define and locate the soils and miscellaneous areas.

An identifying symbol precedes the map unit name in the map unit descriptions. Each description includes general facts about the unit and gives important soil properties and qualities.

Soils that have profiles that are almost alike make up a *soil series*. Except for differences in texture of the surface layer, all the soils of a series have major horizons that are similar in composition, thickness, and arrangement.

Soils of one series can differ in texture of the surface layer, slope, stoniness, salinity, degree of erosion, and other characteristics that affect their use. On the basis of such differences, a soil series is divided into *soil phases*. Most of the areas shown on the detailed soil maps are phases of soil series. The name of a soil phase commonly indicates a feature that affects use or management. For example, Alpha silt loam, 0 to 2 percent slopes, is a phase of the Alpha series.

Some map units are made up of two or more major soils or miscellaneous areas. These map units are complexes, associations, or undifferentiated groups.

A *complex* consists of two or more soils or miscellaneous areas in such an intricate pattern or in such small areas that they cannot be shown separately on the maps. The pattern and proportion of the soils or miscellaneous areas are somewhat similar in all areas. Alpha-Beta complex, 0 to 6 percent slopes, is an example.

An *association* is made up of two or more geographically associated soils or miscellaneous areas that are shown as one unit on the maps. Because of present or anticipated uses of the map units in the survey area, it was not considered practical or necessary to map the soils or miscellaneous areas separately. The pattern and relative proportion of the soils or miscellaneous areas are somewhat similar. Alpha-Beta association, 0 to 2 percent slopes, is an example.

An *undifferentiated group* is made up of two or more soils or miscellaneous areas that could be mapped individually but are mapped as one unit because similar interpretations can be made for use and management. The pattern and proportion of the soils or miscellaneous areas in a mapped area are not uniform. An area can be made up of only one of the major soils or miscellaneous areas, or it can be made up of all of them. Alpha and Beta soils, 0 to 2 percent slopes, is an example.

Some surveys include *miscellaneous areas*. Such areas have little or no soil material and support little or no vegetation. Rock outcrop is an example.

Columbia County, Oregon

15—Crims silt loam, protected

Map Unit Setting

National map unit symbol: 21f3
Elevation: 0 to 20 feet
Mean annual precipitation: 50 to 80 inches
Mean annual air temperature: 50 to 54 degrees F
Frost-free period: 165 to 210 days
Farmland classification: Farmland of unique importance

Map Unit Composition

Crims, protected, and similar soils: 95 percent
Minor components: 4 percent
Estimates are based on observations, descriptions, and transects of the mapunit.

Description of Crims, Protected

Setting

Landform: Flood plains
Landform position (three-dimensional): Tread
Down-slope shape: Linear
Across-slope shape: Linear
Parent material: Partially decomposed herbaceous plant material over silty alluvium

Typical profile

H1 - 0 to 9 inches: silt loam
Oe - 9 to 40 inches: mucky peat
H3 - 40 to 60 inches: silt loam

Properties and qualities

Slope: 0 to 3 percent
Depth to restrictive feature: More than 80 inches
Drainage class: Very poorly drained
Capacity of the most limiting layer to transmit water (Ksat): Moderately high to high (0.57 to 1.98 in/hr)
Depth to water table: About 0 to 12 inches
Frequency of flooding: RareNone
Frequency of ponding: Frequent
Available water capacity: Very high (about 22.2 inches)

Interpretive groups

Land capability classification (irrigated): None specified
Land capability classification (nonirrigated): 3w
Hydrologic Soil Group: B/D
Hydric soil rating: Yes

Minor Components

Locoda, protected

Percent of map unit: 2 percent
Landform: Flood plains
Landform position (three-dimensional): Tread
Down-slope shape: Linear

Across-slope shape: Linear
Other vegetative classification: Very Poorly Drained (G001XY009OR)
Hydric soil rating: Yes

Wauna, protected

Percent of map unit: 2 percent
Landform: Flood plains
Landform position (three-dimensional): Tread
Down-slope shape: Linear
Across-slope shape: Linear
Other vegetative classification: Poorly Drained (G001XY008OR)
Hydric soil rating: Yes

61—Udipsamments, nearly level, protected

Map Unit Setting

National map unit symbol: 21h4
Elevation: 0 to 40 feet
Mean annual precipitation: 50 to 80 inches
Mean annual air temperature: 50 to 54 degrees F
Frost-free period: 145 to 210 days
Farmland classification: Not prime farmland

Map Unit Composition

Udipsamments, protected, and similar soils: 85 percent
Minor components: 12 percent
Estimates are based on observations, descriptions, and transects of the mapunit.

Description of Udipsamments, Protected

Setting

Landform: Flood plains
Landform position (three-dimensional): Tread
Down-slope shape: Linear
Across-slope shape: Linear
Parent material: Sandy dredge spoils

Typical profile

H1 - 0 to 4 inches: loamy sand
H2 - 4 to 60 inches: fine sand

Properties and qualities

Slope: 0 to 3 percent
Depth to restrictive feature: More than 80 inches
Drainage class: Well drained
Capacity of the most limiting layer to transmit water (Ksat): High to very high (5.95 to 99.90 in/hr)
Depth to water table: More than 80 inches
Frequency of flooding: RareNone
Frequency of ponding: None
Available water capacity: Low (about 3.0 inches)

Interpretive groups

Land capability classification (irrigated): None specified
Land capability classification (nonirrigated): 6s
Hydrologic Soil Group: A
Hydric soil rating: Yes

Minor Components

Wauna, protected

Percent of map unit: 4 percent
Landform: Flood plains
Landform position (three-dimensional): Tread
Down-slope shape: Linear
Across-slope shape: Linear
Other vegetative classification: Poorly Drained (G001XY008OR)
Hydric soil rating: Yes

Locoda, protected

Percent of map unit: 4 percent
Landform: Flood plains
Landform position (three-dimensional): Tread
Down-slope shape: Linear
Across-slope shape: Linear
Other vegetative classification: Very Poorly Drained (G001XY009OR)
Hydric soil rating: Yes

Crims, protected

Percent of map unit: 4 percent
Landform: Flood plains
Hydric soil rating: Yes

66—Wauna silt loam, protected

Map Unit Setting

National map unit symbol: 21h9
Elevation: 0 to 40 feet
Mean annual precipitation: 50 to 80 inches
Mean annual air temperature: 50 to 54 degrees F
Frost-free period: 145 to 210 days
Farmland classification: Farmland of statewide importance

Map Unit Composition

Wauna, protected, and similar soils: 90 percent
Minor components: 8 percent
Estimates are based on observations, descriptions, and transects of the mapunit.

Description of Wauna, Protected

Setting

Landform: Flood plains

Landform position (three-dimensional): Tread
Down-slope shape: Linear
Across-slope shape: Linear
Parent material: Silty alluvium derived from mixed sources

Typical profile

H1 - 0 to 8 inches: silt loam
H2 - 8 to 26 inches: silt loam
H3 - 26 to 60 inches: silt loam

Properties and qualities

Slope: 0 to 3 percent
Depth to restrictive feature: More than 80 inches
Drainage class: Poorly drained
Capacity of the most limiting layer to transmit water (Ksat): Moderately high (0.20 to 0.57 in/hr)
Depth to water table: About 24 to 60 inches
Frequency of flooding: NoneRare
Frequency of ponding: None
Available water capacity: Very high (about 12.0 inches)

Interpretive groups

Land capability classification (irrigated): None specified
Land capability classification (nonirrigated): 2w
Hydrologic Soil Group: C
Forage suitability group: Poorly Drained (G002XY006OR)
Other vegetative classification: Poorly Drained (G002XY006OR)
Hydric soil rating: Yes

Minor Components

Crims, protected

Percent of map unit: 3 percent
Landform: Flood plains
Hydric soil rating: Yes

Locoda, protected

Percent of map unit: 3 percent
Landform: Flood plains
Landform position (three-dimensional): Tread
Down-slope shape: Linear
Across-slope shape: Linear
Other vegetative classification: Very Poorly Drained (G001XY009OR)
Hydric soil rating: Yes

Udipsamments, protected

Percent of map unit: 2 percent
Landform: Flood plains
Landform position (three-dimensional): Tread
Down-slope shape: Linear
Across-slope shape: Linear
Hydric soil rating: Yes

68—Wauna-Locoda silt loams, protected

Map Unit Setting

National map unit symbol: 21hc
Elevation: 0 to 40 feet
Mean annual precipitation: 50 to 80 inches
Mean annual air temperature: 50 to 54 degrees F
Frost-free period: 145 to 210 days
Farmland classification: Farmland of statewide importance

Map Unit Composition

Wauna, protected, and similar soils: 45 percent
Locoda, protected, and similar soils: 35 percent
Minor components: 14 percent
Estimates are based on observations, descriptions, and transects of the mapunit.

Description of Wauna, Protected

Setting

Landform: Flood plains
Landform position (three-dimensional): Tread
Down-slope shape: Linear
Across-slope shape: Linear
Parent material: Silty alluvium derived from mixed sources

Typical profile

H1 - 0 to 8 inches: silt loam
H2 - 8 to 26 inches: silt loam
H3 - 26 to 60 inches: silt loam

Properties and qualities

Slope: 0 to 3 percent
Depth to restrictive feature: More than 80 inches
Drainage class: Poorly drained
Capacity of the most limiting layer to transmit water (Ksat): Moderately high (0.20 to 0.57 in/hr)
Depth to water table: About 24 to 60 inches
Frequency of flooding: NoneRare
Frequency of ponding: None
Available water capacity: Very high (about 12.0 inches)

Interpretive groups

Land capability classification (irrigated): None specified
Land capability classification (nonirrigated): 2w
Hydrologic Soil Group: C
Forage suitability group: Poorly Drained (G002XY006OR)
Other vegetative classification: Poorly Drained (G002XY006OR)
Hydric soil rating: Yes

Description of Locoda, Protected

Setting

Landform: Flood plains
Landform position (three-dimensional): Tread
Down-slope shape: Linear
Across-slope shape: Linear
Parent material: Silty alluvium from mixed sources

Typical profile

H1 - 0 to 10 inches: silt loam
H2 - 10 to 60 inches: silty clay loam

Properties and qualities

Slope: 0 to 3 percent
Depth to restrictive feature: More than 80 inches
Drainage class: Very poorly drained
Capacity of the most limiting layer to transmit water (Ksat): Moderately high (0.20 to 0.57 in/hr)
Depth to water table: About 0 to 12 inches
Frequency of flooding: RareNone
Frequency of ponding: Frequent
Available water capacity: Very high (about 12.1 inches)

Interpretive groups

Land capability classification (irrigated): None specified
Land capability classification (nonirrigated): 3w
Hydrologic Soil Group: C/D
Forage suitability group: Poorly Drained (G002XY006OR)
Other vegetative classification: Poorly Drained (G002XY006OR)
Hydric soil rating: Yes

Minor Components

Udipsamments, protected

Percent of map unit: 7 percent
Landform: Flood plains
Landform position (three-dimensional): Tread
Down-slope shape: Linear
Across-slope shape: Linear
Hydric soil rating: Yes

Crims, protected

Percent of map unit: 7 percent
Landform: Flood plains
Hydric soil rating: Yes

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APPENDIX C

WATER QUALITY TREATMENT SWALE SIZING

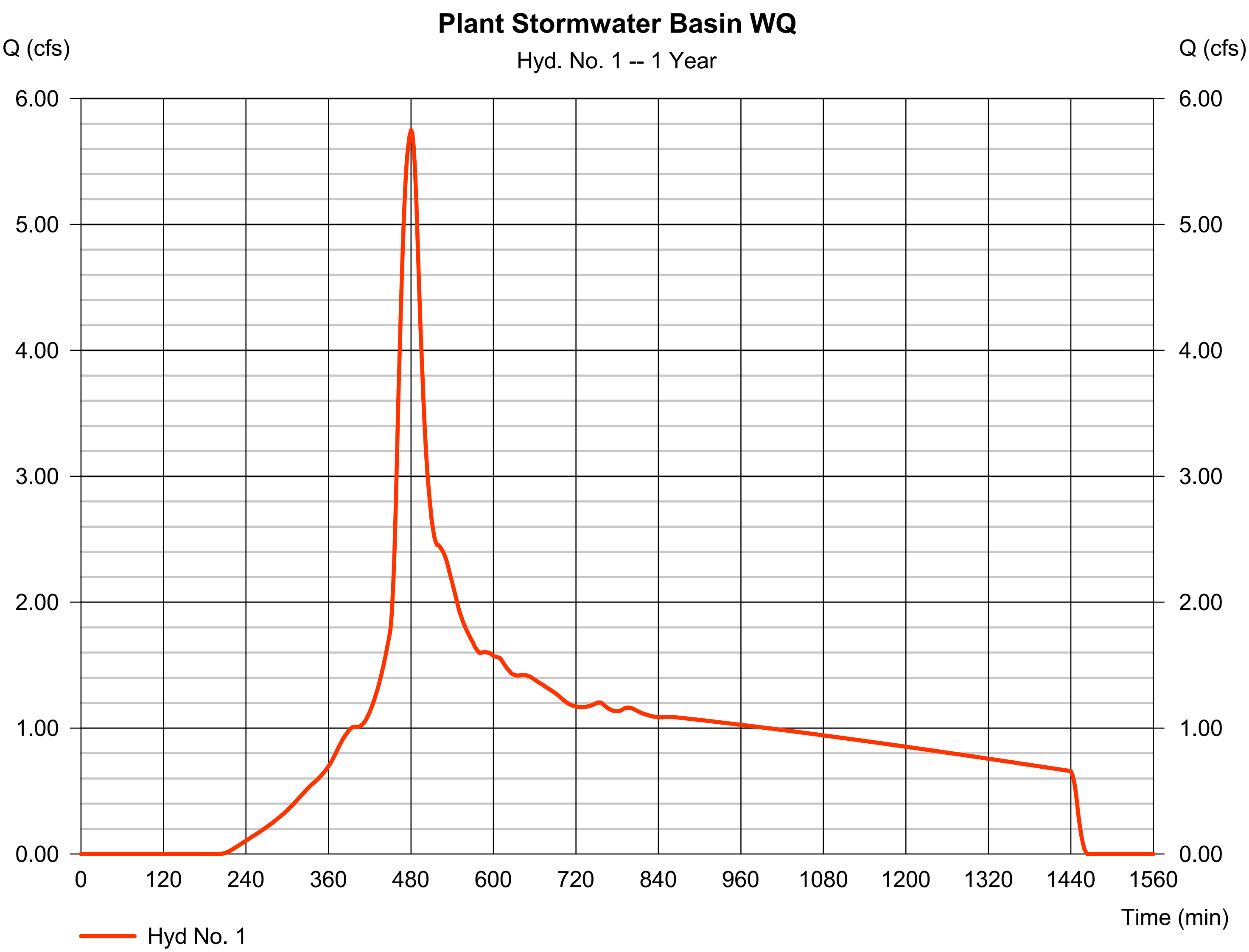
Hydrograph Report

Hyd. No. 1

Plant Stormwater Basin WQ

Hydrograph type	= SCS Runoff	Peak discharge	= 5.750 cfs
Storm frequency	= 1 yrs	Time to peak	= 480 min
Time interval	= 2 min	Hyd. volume	= 84,625 cuft
Drainage area	= 28.220 ac	Curve number	= 94*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 15.00 min
Total precip.	= 1.40 in	Distribution	= Type IA
Storm duration	= 24 hrs	Shape factor	= 484

* Composite (Area/CN) = [(19.400 x 92) + (8.820 x 98)] / 28.220



Hydrograph Report

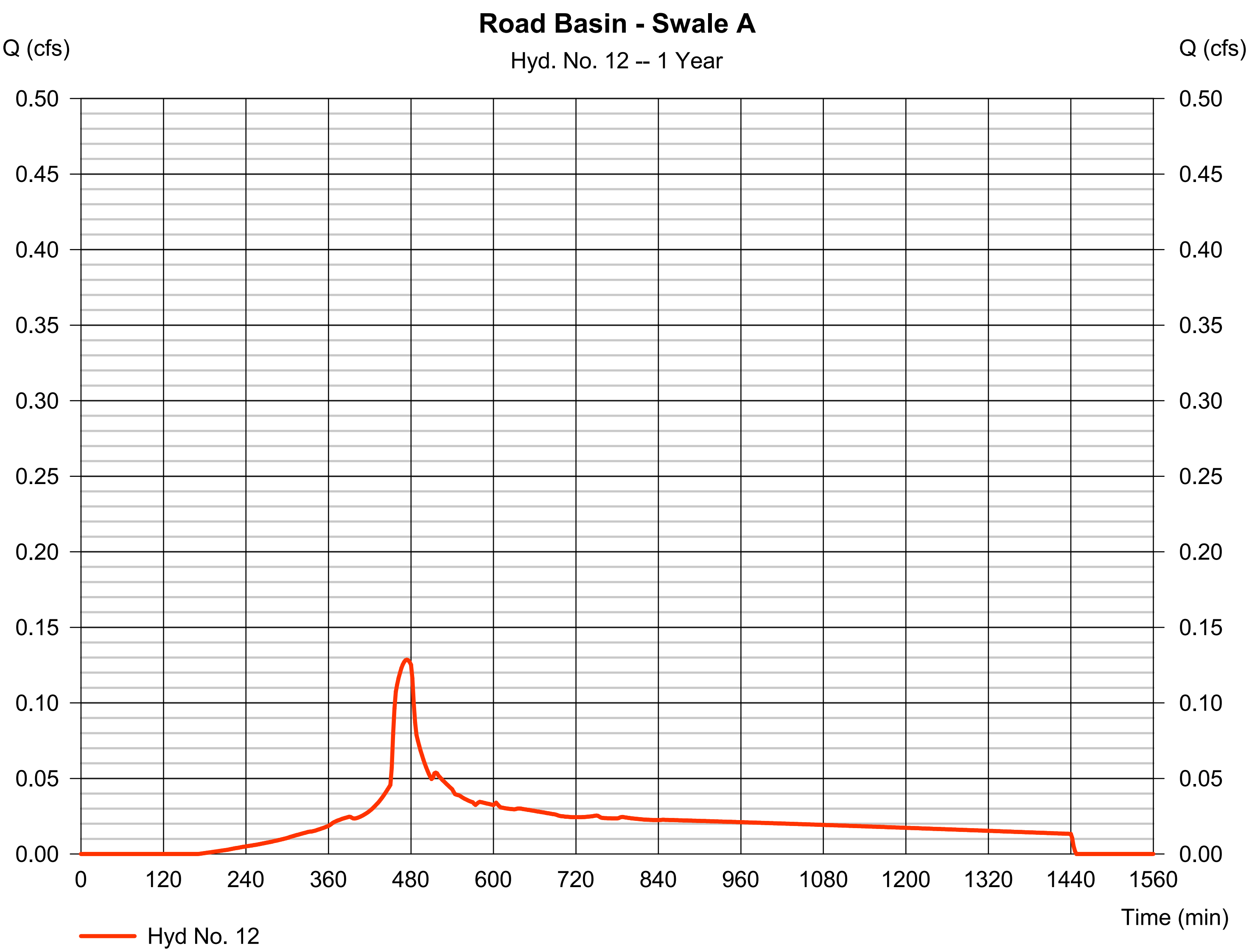
Hydraflow Hydrographs Extension for Autodesk® Civil 3D® 2019 by Autodesk, Inc. v2020 — Thursday, 01 / 7 / 2021

Hyd. No. 12

Road Basin - Swale A

Hydrograph type	= SCS Runoff	Peak discharge	= 0.129 cfs
Storm frequency	= 1 yrs	Time to peak	= 474 min
Time interval	= 2 min	Hyd. volume	= 1,817 cuft
Drainage area	= 0.580 ac	Curve number	= 95*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 5.00 min
Total precip.	= 1.40 in	Distribution	= Type IA
Storm duration	= 24 hrs	Shape factor	= 484

* Composite (Area/CN) = [(0.290 x 98) + (0.290 x 92)] / 0.580



Open Channel Design - Swales and Ditches

Project Name:	NEXT Renewables Port Westward
Project No.:	2200315.01
Channel ID:	Access Road drainage swales
Date:	1/6/2021
By:	B. Nielsen

Channel Geometry

Left Side Slope:	3 H : 1V	Channel Lining:	Grass / Vegetation
Right Side Slope:	3 H : 1V	Manning's *n* Roughness:	0.25
Bottom Width:	2.0 ft	Channel Length:	115 ft
Flowline Slope:	0.012 ft / ft	Min. Freeboard Req'd:	1.0 ft

Hydrology Summary

WQ Storm Flow:	Q_{WQ} =	0.13 cfs
2-yr Storm Flow:	Q_2 =	0.32 cfs
10-yr Storm Flow:	Q_{10} =	0.48 cfs
100-yr Storm Flow:	Q_{100} =	0.68 cfs

Uniform Flow Depth Summary

- Calculate flow characteristics using Manning's Equation assuming steady uniform flow:

$$Q = \frac{a}{n} A \cdot R^{2/3} \sqrt{S}$$

Storm Event	Flow Rate (cfs)	Flow Depth (in)	Flow Velocity (ft/s)	Residence Time (min)
WQ	0.13	2.77	0.21	9.20
2-yr	0.32	4.54	0.27	7.00
10-yr	0.48	5.55	0.30	6.30
100-yr	0.68	6.65	0.33	5.70

Design Criteria Check:

WQ Flow Depth < 0.33 ft:	**OK**
WQ Flow Velocity < 0.9 fps:	**OK**
WQ Hydraulic Residence Time > 9 min:	**OK**
Slope Between 0.5% - 1.5%:	**OK**
Conveyance Velocity < 3 fps:	**OK**

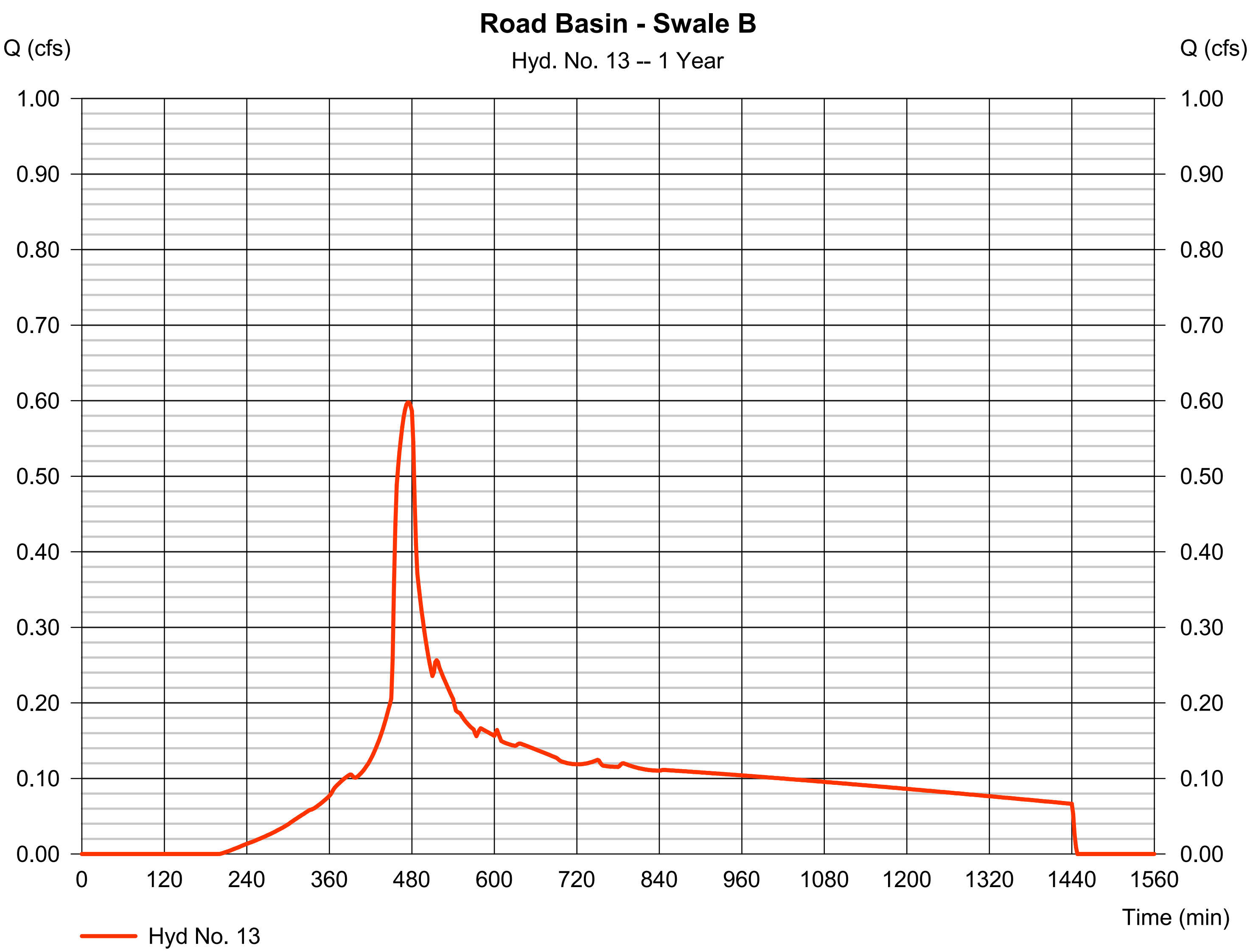
Hydrograph Report

Hyd. No. 13

Road Basin - Swale B

Hydrograph type	= SCS Runoff	Peak discharge	= 0.598 cfs
Storm frequency	= 1 yrs	Time to peak	= 474 min
Time interval	= 2 min	Hyd. volume	= 8,621 cuft
Drainage area	= 2.990 ac	Curve number	= 94*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 5.00 min
Total precip.	= 1.40 in	Distribution	= Type IA
Storm duration	= 24 hrs	Shape factor	= 484

* Composite (Area/CN) = [(0.790 x 98) + (2.200 x 92)] / 2.990



Open Channel Design - Swales and Ditches

Project Name:	NEXT Renewables Port Westward
Project No.:	2200315.01
Channel ID:	Access Road drainage - Swale B
Date:	1/6/2021
By:	B. Nielsen

Channel Geometry

Left Side Slope:	3 H : 1V	Channel Lining:	Grass / Vegetation
Right Side Slope:	3 H : 1V	Manning's *n* Roughness:	0.25
Bottom Width:	8.0 ft	Channel Length:	115 ft
Flowline Slope:	0.006 ft / ft	Min. Freeboard Req'd:	1.0 ft

Hydrology Summary

WQ Storm Flow:	Q_{WQ} =	0.60 cfs
2-yr Storm Flow:	Q_2 =	1.59 cfs
10-yr Storm Flow:	Q_{10} =	2.38 cfs
100-yr Storm Flow:	Q_{100} =	3.44 cfs

Uniform Flow Depth Summary

- Calculate flow characteristics using Manning's Equation assuming steady uniform flow:

$$Q = \frac{a}{n} A \cdot R^{2/3} \sqrt{S}$$

Storm Event	Flow Rate (cfs)	Flow Depth (in)	Flow Velocity (ft/s)	Residence Time (min)
WQ	0.60	3.94	0.20	9.40
2-yr	1.59	6.93	0.28	6.70
10-yr	2.38	8.70	0.32	5.90
100-yr	3.44	10.69	0.36	5.20

Design Criteria Check:

WQ Flow Depth < 0.33 ft:	**OK**
WQ Flow Velocity < 0.9 fps:	**OK**
WQ Hydraulic Residence Time > 9 min:	**OK**
Slope Between 0.5% - 1.5%:	**OK**
Conveyance Velocity < 3 fps:	**OK**

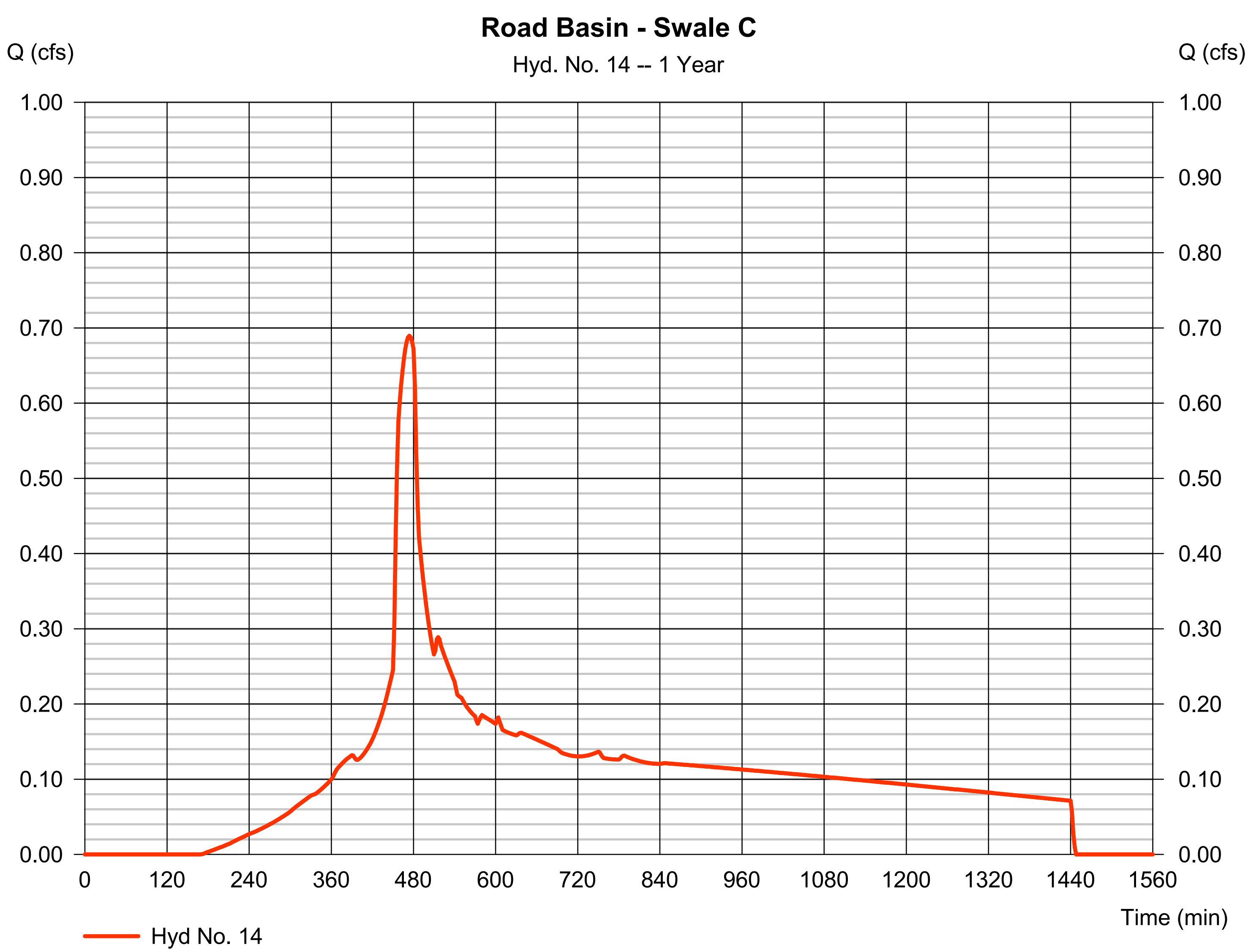
Hydrograph Report

Hyd. No. 14

Road Basin - Swale C

Hydrograph type	= SCS Runoff	Peak discharge	= 0.689 cfs
Storm frequency	= 1 yrs	Time to peak	= 474 min
Time interval	= 2 min	Hyd. volume	= 9,743 cuft
Drainage area	= 3.110 ac	Curve number	= 95*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 5.00 min
Total precip.	= 1.40 in	Distribution	= Type IA
Storm duration	= 24 hrs	Shape factor	= 484

* Composite (Area/CN) = [(1.720 x 98) + (1.390 x 92)] / 3.110



Open Channel Design - Swales and Ditches

Project Name:	NEXT Renewables Port Westward
Project No.:	2200315.01
Channel ID:	Access Road drainage - Swale C
Date:	1/6/2021
By:	B. Nielsen

Channel Geometry

Left Side Slope:	3 H : 1V	Channel Lining:	Grass / Vegetation
Right Side Slope:	3 H : 1V	Manning's *n* Roughness:	0.25
Bottom Width:	9.5 ft	Channel Length:	115 ft
Flowline Slope:	0.006 ft / ft	Min. Freeboard Req'd:	1.0 ft

Hydrology Summary

WQ Storm Flow:	Q_{WQ} =	0.69 cfs
2-yr Storm Flow:	Q_2 =	1.73 cfs
10-yr Storm Flow:	Q_{10} =	2.55 cfs
100-yr Storm Flow:	Q_{100} =	3.64 cfs

Uniform Flow Depth Summary

- Calculate flow characteristics using Manning's Equation assuming steady uniform flow:

$$Q = \frac{a}{n} A \cdot R^{2/3} \sqrt{S}$$

Storm Event	Flow Rate (cfs)	Flow Depth (in)	Flow Velocity (ft/s)	Residence Time (min)
WQ	0.69	3.88	0.20	9.40
2-yr	1.73	6.64	0.28	6.80
10-yr	2.55	8.28	0.32	6.00
100-yr	3.64	10.15	0.36	5.30

Design Criteria Check:

WQ Flow Depth < 0.33 ft:	**OK**
WQ Flow Velocity < 0.9 fps:	**OK**
WQ Hydraulic Residence Time > 9 min:	**OK**
Slope Between 0.5% - 1.5%:	**OK**
Conveyance Velocity < 3 fps:	**OK**

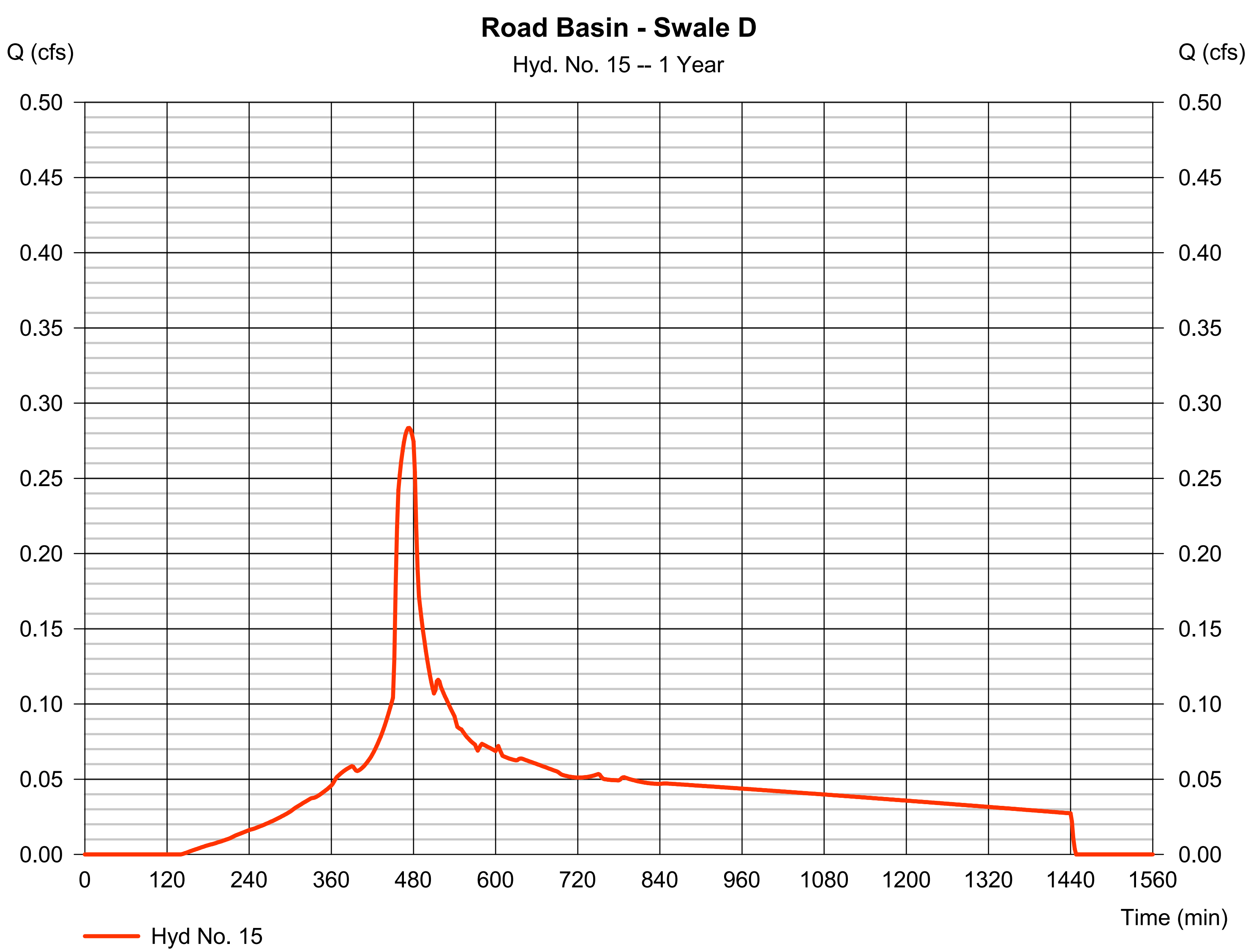
Hydrograph Report

Hyd. No. 15

Road Basin - Swale D

Hydrograph type	= SCS Runoff	Peak discharge	= 0.284 cfs
Storm frequency	= 1 yrs	Time to peak	= 474 min
Time interval	= 2 min	Hyd. volume	= 3,948 cuft
Drainage area	= 1.160 ac	Curve number	= 96*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 5.00 min
Total precip.	= 1.40 in	Distribution	= Type IA
Storm duration	= 24 hrs	Shape factor	= 484

* Composite (Area/CN) = [(0.740 x 98) + (0.420 x 92)] / 1.160



Open Channel Design - Swales and Ditches

Project Name:	NEXT Renewables Port Westward
Project No.:	2200315.01
Channel ID:	Access Road drainage - Swale D
Date:	1/6/2021
By:	B. Nielsen

Channel Geometry

Left Side Slope:	3 H : 1V	Channel Lining:	Grass / Vegetation
Right Side Slope:	3 H : 1V	Manning's *n* Roughness:	0.25
Bottom Width:	4.0 ft	Channel Length:	105 ft
Flowline Slope:	0.006 ft / ft	Min. Freeboard Req'd:	1.0 ft

Hydrology Summary

WQ Storm Flow:	Q_{WQ} =	0.28 cfs
2-yr Storm Flow:	Q_2 =	0.67 cfs
10-yr Storm Flow:	Q_{10} =	0.97 cfs
100-yr Storm Flow:	Q_{100} =	1.38 cfs

Uniform Flow Depth Summary

- Calculate flow characteristics using Manning's Equation assuming steady uniform flow:

$$Q = \frac{a}{n} A \cdot R^{2/3} \sqrt{S}$$

Storm Event	Flow Rate (cfs)	Flow Depth (in)	Flow Velocity (ft/s)	Residence Time (min)
WQ	0.28	3.72	0.19	9.40
2-yr	0.67	6.02	0.24	7.10
10-yr	0.97	7.35	0.27	6.40
100-yr	1.38	8.85	0.30	5.80

Design Criteria Check:

WQ Flow Depth < 0.33 ft:	**OK**
WQ Flow Velocity < 0.9 fps:	**OK**
WQ Hydraulic Residence Time > 9 min:	**OK**
Slope Between 0.5% - 1.5%:	**OK**
Conveyance Velocity < 3 fps:	**OK**

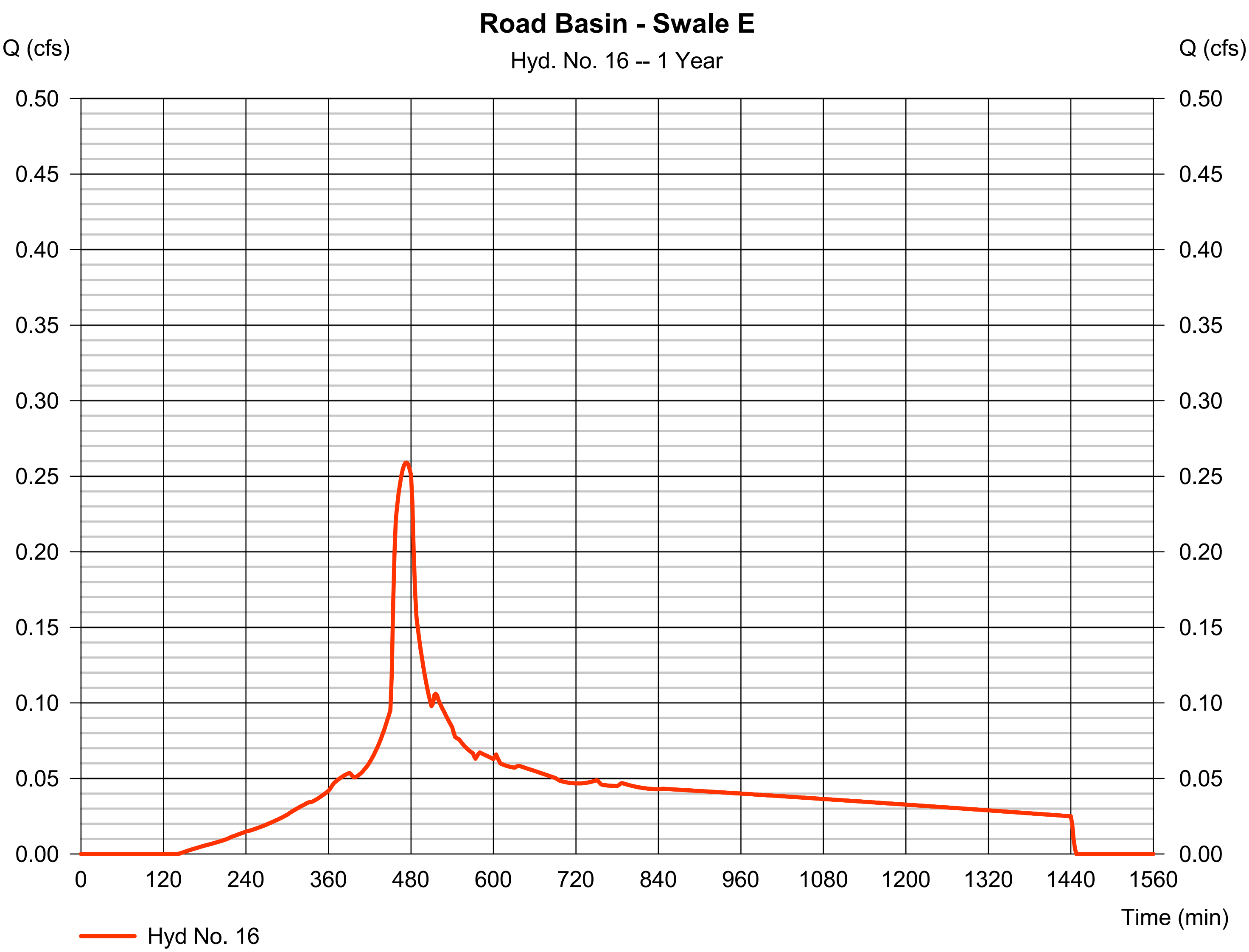
Hydrograph Report

Hyd. No. 16

Road Basin - Swale E

Hydrograph type	= SCS Runoff	Peak discharge	= 0.259 cfs
Storm frequency	= 1 yrs	Time to peak	= 474 min
Time interval	= 2 min	Hyd. volume	= 3,608 cuft
Drainage area	= 1.060 ac	Curve number	= 96*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 5.00 min
Total precip.	= 1.40 in	Distribution	= Type IA
Storm duration	= 24 hrs	Shape factor	= 484

* Composite (Area/CN) = [(0.630 x 98) + (0.430 x 92)] / 1.060



Open Channel Design - Swales and Ditches

Project Name:	NEXT Renewables Port Westward
Project No.:	2200315.01
Channel ID:	Access Road drainage - Swale E
Date:	1/6/2021
By:	B. Nielsen

Channel Geometry

Left Side Slope:	3 H : 1V	Channel Lining:	Grass / Vegetation
Right Side Slope:	3 H : 1V	Manning's *n* Roughness:	0.25
Bottom Width:	4.0 ft	Channel Length:	100 ft
Flowline Slope:	0.006 ft / ft	Min. Freeboard Req'd:	1.0 ft

Hydrology Summary

WQ Storm Flow:	Q_{WQ} =	0.26 cfs
2-yr Storm Flow:	Q_2 =	0.61 cfs
10-yr Storm Flow:	Q_{10} =	0.89 cfs
100-yr Storm Flow:	Q_{100} =	1.26 cfs

Uniform Flow Depth Summary

- Calculate flow characteristics using Manning's Equation assuming steady uniform flow:

$$Q = \frac{a}{n} A \cdot R^{2/3} \sqrt{S}$$

Storm Event	Flow Rate (cfs)	Flow Depth (in)	Flow Velocity (ft/s)	Residence Time (min)
WQ	0.26	3.53	0.18	9.20
2-yr	0.61	5.72	0.24	7.00
10-yr	0.89	7.01	0.26	6.20
100-yr	1.26	8.44	0.29	5.60

Design Criteria Check:

WQ Flow Depth < 0.33 ft:	OK
WQ Flow Velocity < 0.9 fps:	OK
WQ Hydraulic Residence Time > 9 min:	OK
Slope Between 0.5% - 1.5%:	OK
Conveyance Velocity < 3 fps:	OK

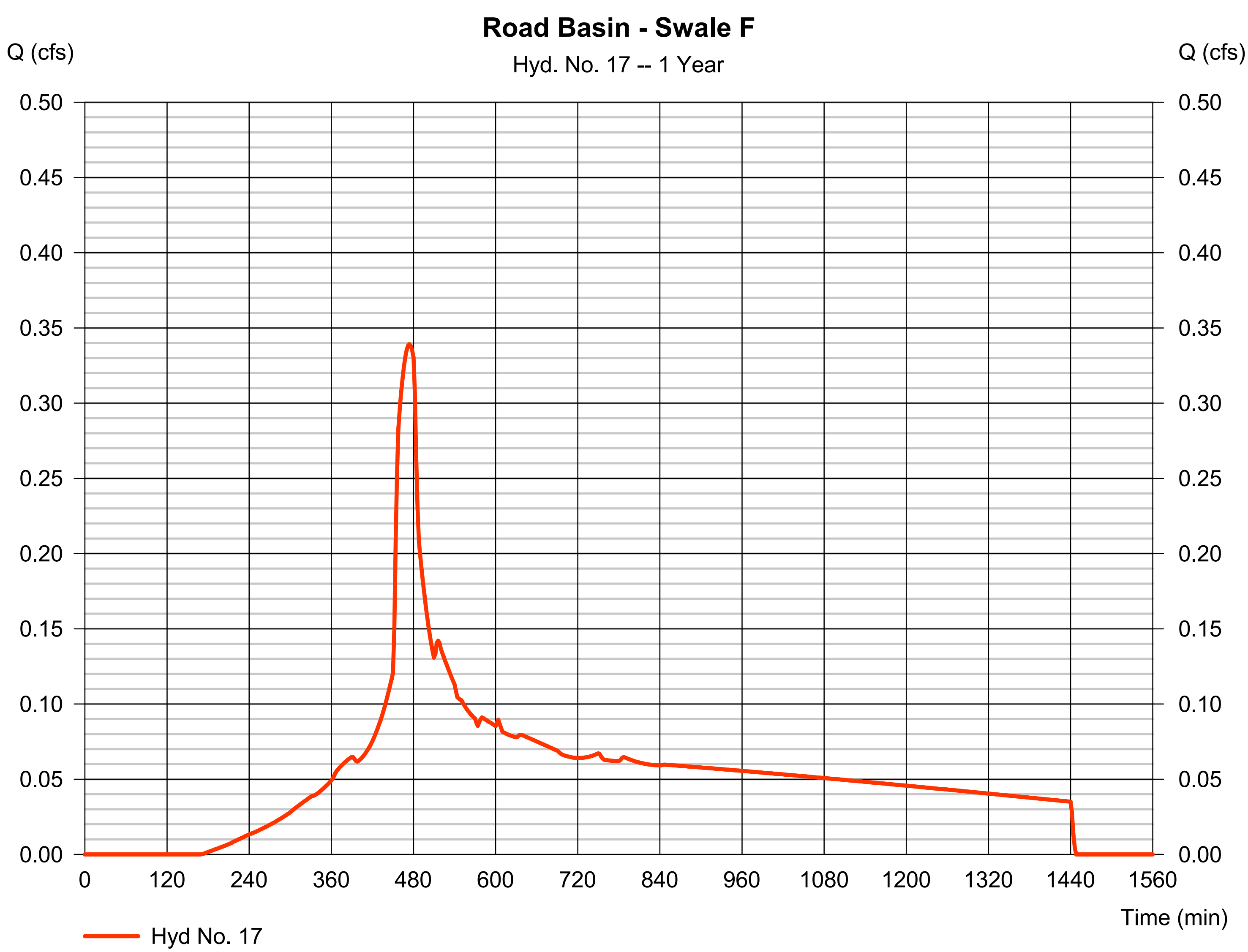
Hydrograph Report

Hyd. No. 17

Road Basin - Swale F

Hydrograph type	= SCS Runoff	Peak discharge	= 0.339 cfs
Storm frequency	= 1 yrs	Time to peak	= 474 min
Time interval	= 2 min	Hyd. volume	= 4,793 cuft
Drainage area	= 1.530 ac	Curve number	= 95*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 5.00 min
Total precip.	= 1.40 in	Distribution	= Type IA
Storm duration	= 24 hrs	Shape factor	= 484

* Composite (Area/CN) = [(0.880 x 98) + (0.650 x 92)] / 1.530



Open Channel Design - Swales and Ditches

Project Name:	NEXT Renewables Port Westward
Project No.:	2200315.01
Channel ID:	Access Road drainage - Swale F
Date:	1/6/2021
By:	B. Nielsen

Channel Geometry

Left Side Slope:	3 H : 1V	Channel Lining:	Grass / Vegetation
Right Side Slope:	3 H : 1V	Manning's *n* Roughness:	0.25
Bottom Width:	4.0 ft	Channel Length:	100 ft
Flowline Slope:	0.005 ft / ft	Min. Freeboard Req'd:	1.0 ft

Hydrology Summary

WQ Storm Flow:	Q_{WQ} =	0.34 cfs
2-yr Storm Flow:	Q_2 =	0.85 cfs
10-yr Storm Flow:	Q_{10} =	1.25 cfs
100-yr Storm Flow:	Q_{100} =	1.79 cfs

Uniform Flow Depth Summary

- Calculate flow characteristics using Manning's Equation assuming steady uniform flow:

$$Q = \frac{a}{n} A \cdot R^{2/3} \sqrt{S}$$

Storm Event	Flow Rate (cfs)	Flow Depth (in)	Flow Velocity (ft/s)	Residence Time (min)
WQ	0.18	2.96	0.15	11.00
2-yr	0.85	7.15	0.25	6.70
10-yr	1.25	8.78	0.28	6.00
100-yr	1.79	10.58	0.31	5.40

Design Criteria Check:

WQ Flow Depth < 0.33 ft:	**OK**
WQ Flow Velocity < 0.9 fps:	**OK**
WQ Hydraulic Residence Time > 9 min:	**OK**
Slope Between 0.5% - 1.5%:	**OK**
Conveyance Velocity < 3 fps:	**OK**

APPENDIX D

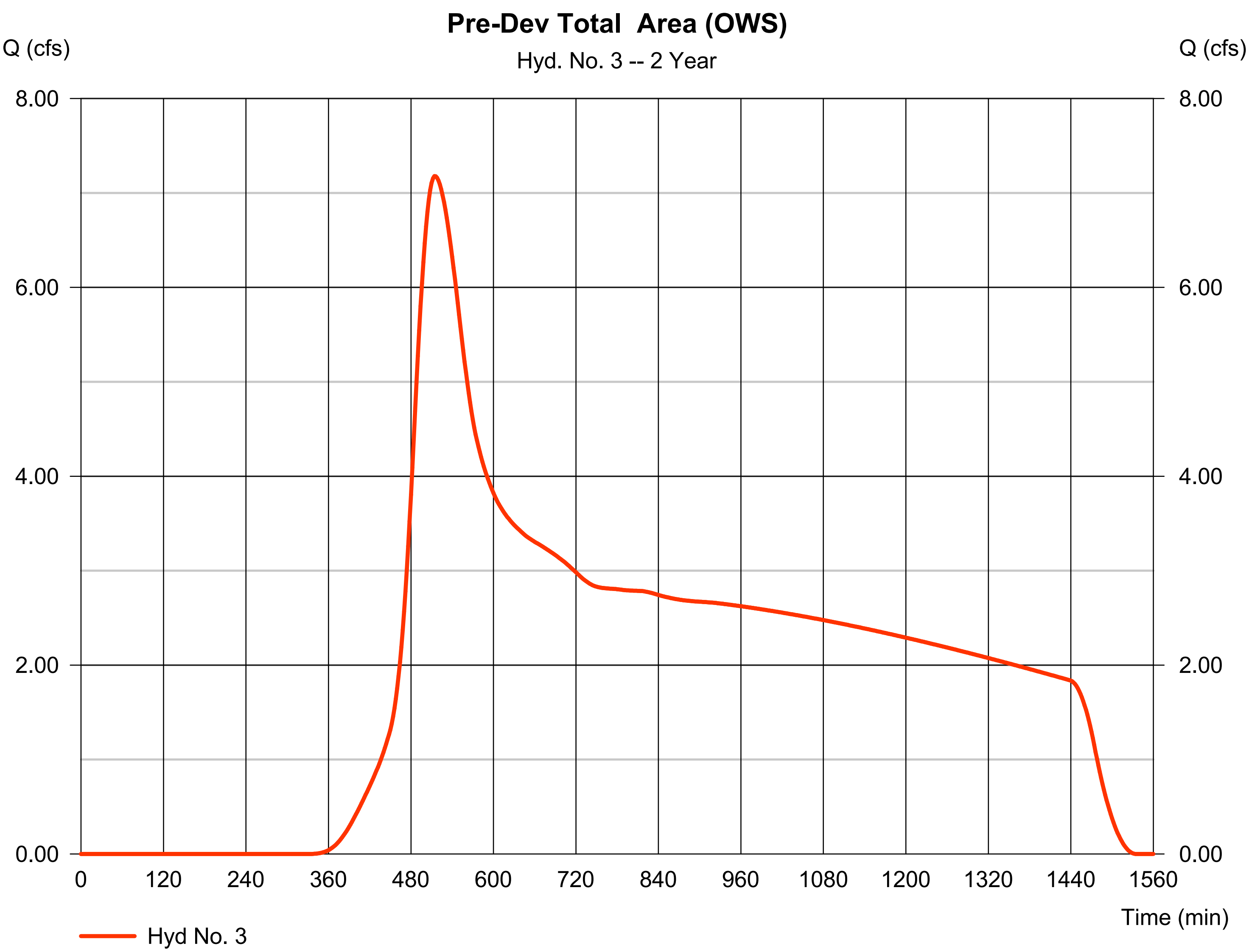
PRE-DEVELOPMENT AND DEVELOPED HYDROLOGY CALCULATIONS

Hydrograph Report

Hyd. No. 3

Pre-Dev Total Area (OWS)

Hydrograph type	= SCS Runoff	Peak discharge	= 7.178 cfs
Storm frequency	= 2 yrs	Time to peak	= 514 min
Time interval	= 2 min	Hyd. volume	= 180,665 cuft
Drainage area	= 45.160 ac	Curve number	= 80
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 60.00 min
Total precip.	= 2.80 in	Distribution	= Type IA
Storm duration	= 24 hrs	Shape factor	= 484



Hydrograph Report

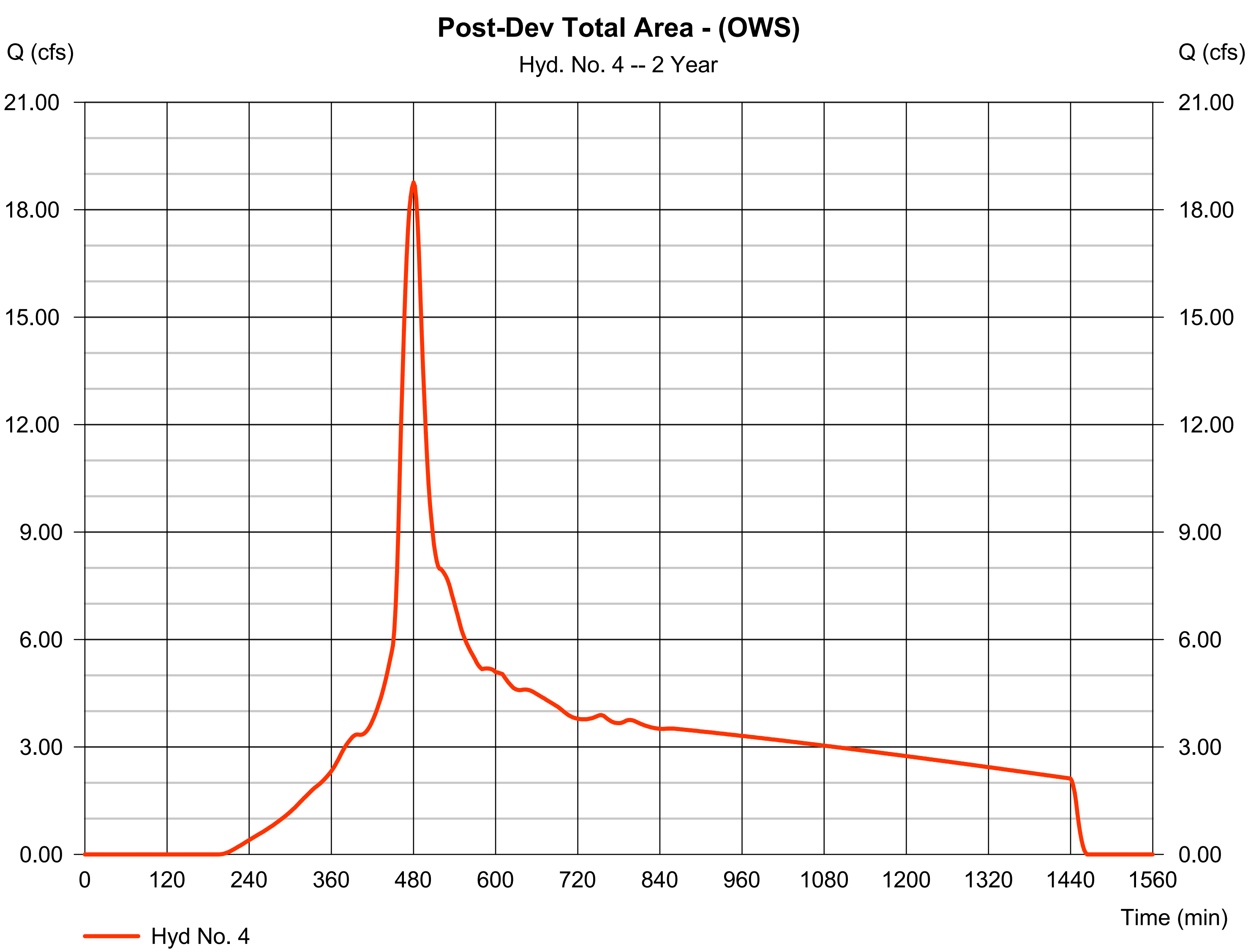
Hydraflow Hydrographs Extension for Autodesk® Civil 3D® 2019 by Autodesk, Inc. v2020 Thursday, 01 / 7 / 2021

Hyd. No. 4

Post-Dev Total Area - (OWS)

Hydrograph type	= SCS Runoff	Peak discharge	= 18.76 cfs
Storm frequency	= 2 yrs	Time to peak	= 480 min
Time interval	= 2 min	Hyd. volume	= 274,919 cuft
Drainage area	= 45.160 ac	Curve number	= 89*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 15.00 min
Total precip.	= 2.80 in	Distribution	= Type IA
Storm duration	= 24 hrs	Shape factor	= 484

* Composite (Area/CN) = [(2.020 x 98) + (4.460 x 92) + (19.110 x 98) + (19.570 x 78)] / 45.160

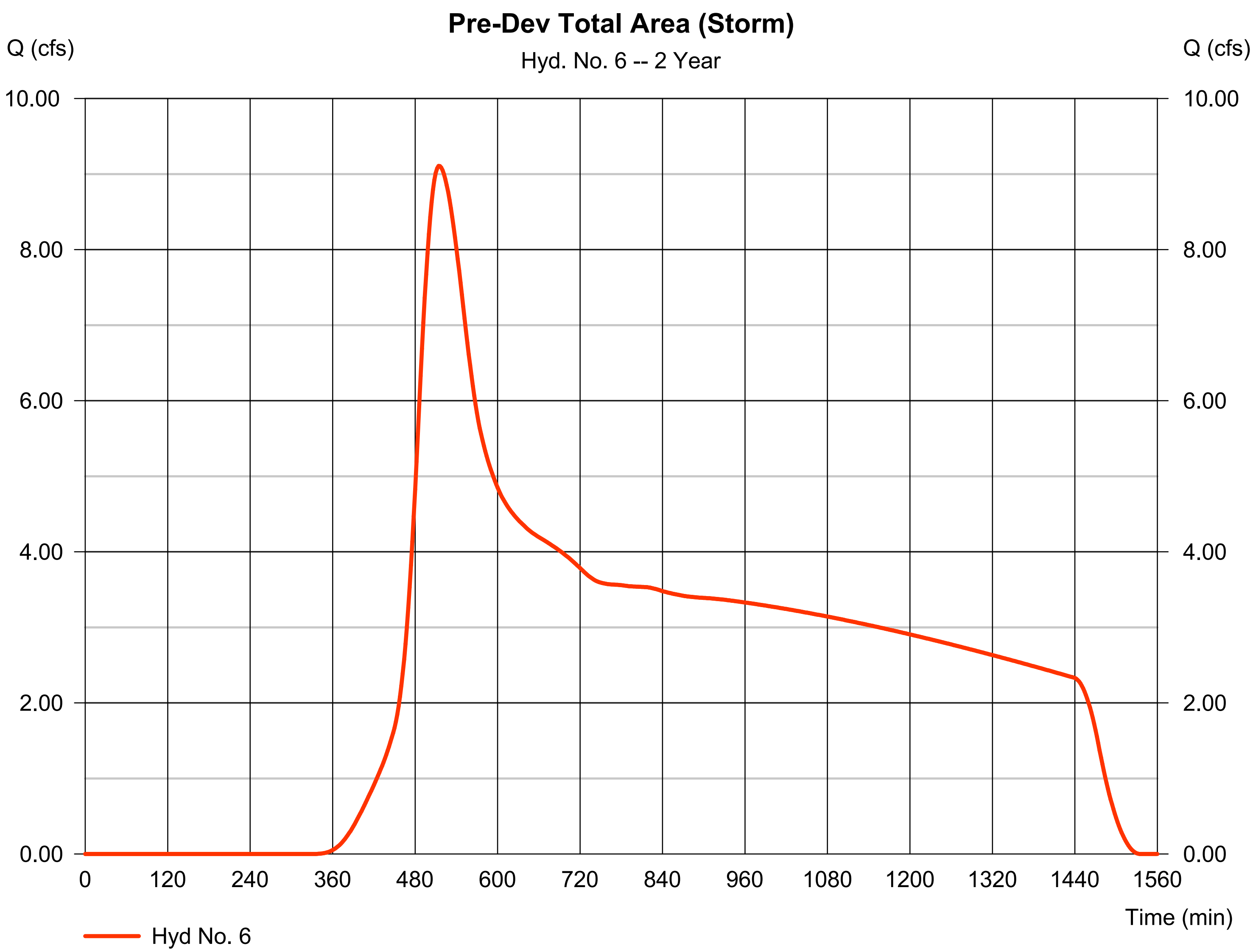


Hydrograph Report

Hyd. No. 6

Pre-Dev Total Area (Storm)

Hydrograph type	= SCS Runoff	Peak discharge	= 9.108 cfs
Storm frequency	= 2 yrs	Time to peak	= 514 min
Time interval	= 2 min	Hyd. volume	= 229,232 cuft
Drainage area	= 57.300 ac	Curve number	= 80
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 60.00 min
Total precip.	= 2.80 in	Distribution	= Type IA
Storm duration	= 24 hrs	Shape factor	= 484



Hydrograph Report

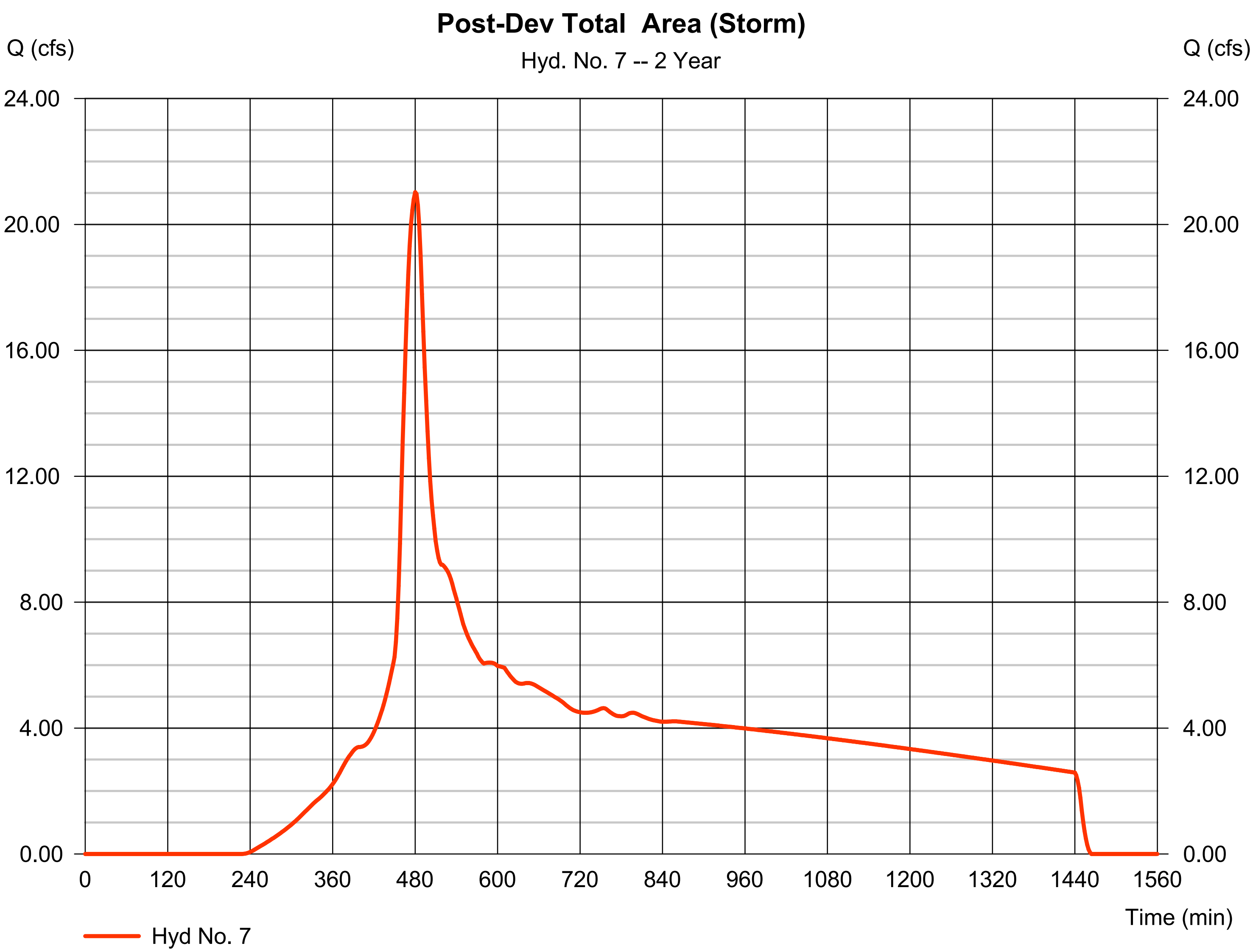
Hydraflow Hydrographs Extension for Autodesk® Civil 3D® 2019 by Autodesk, Inc. v2020 Thursday, 01 / 7 / 2021

Hyd. No. 7

Post-Dev Total Area (Storm)

Hydrograph type	= SCS Runoff	Peak discharge	= 21.03 cfs
Storm frequency	= 2 yrs	Time to peak	= 480 min
Time interval	= 2 min	Hyd. volume	= 317,530 cuft
Drainage area	= 57.300 ac	Curve number	= 87*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 15.00 min
Total precip.	= 2.80 in	Distribution	= Type IA
Storm duration	= 24 hrs	Shape factor	= 484

* Composite (Area/CN) = [(4.390 x 98) + (19.400 x 92) + (8.820 x 98) + (24.690 x 78)] / 57.300

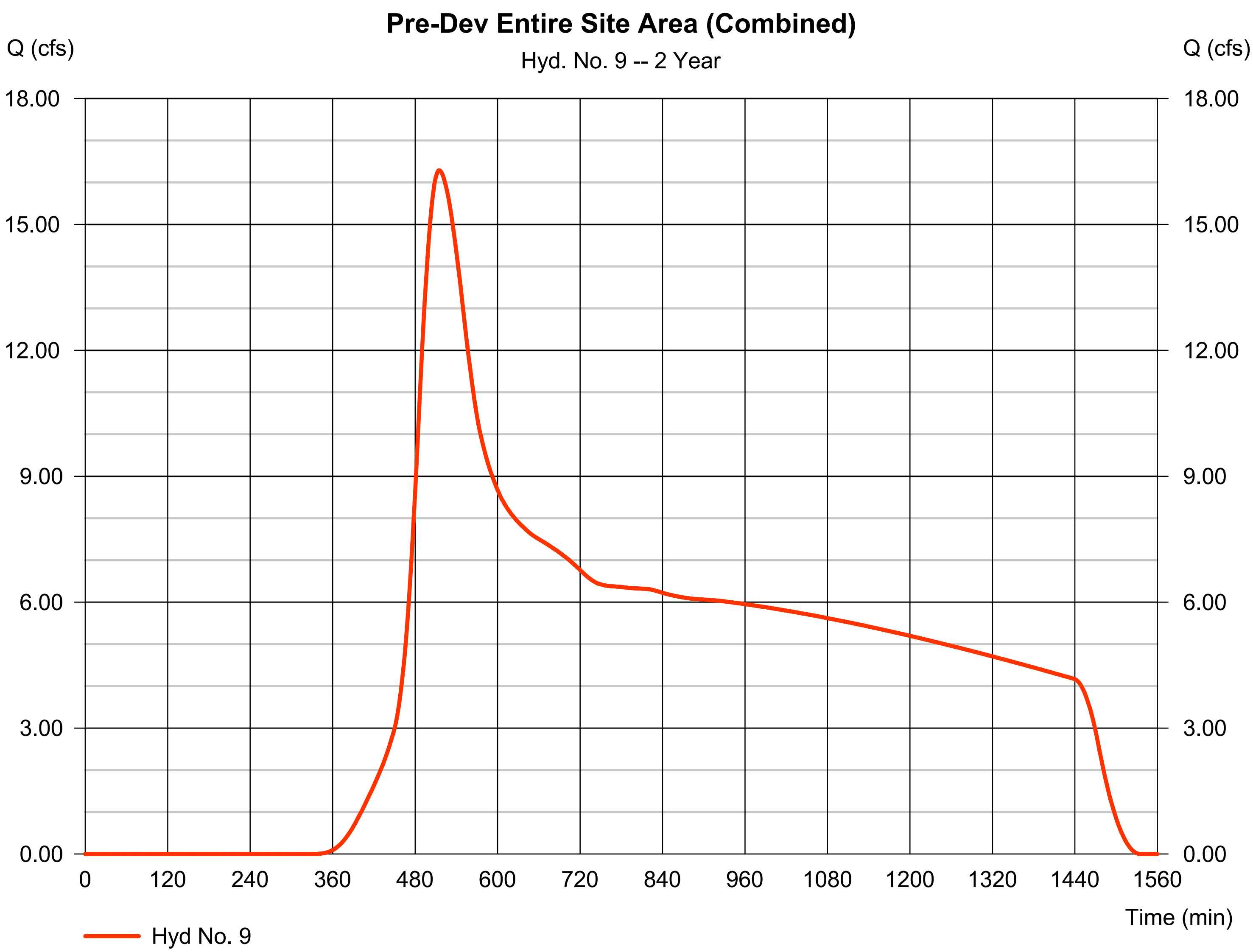


Hydrograph Report

Hyd. No. 9

Pre-Dev Entire Site Area (Combined)

Hydrograph type	= SCS Runoff	Peak discharge	= 16.29 cfs
Storm frequency	= 2 yrs	Time to peak	= 514 min
Time interval	= 2 min	Hyd. volume	= 409,897 cuft
Drainage area	= 102.460 ac	Curve number	= 80
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 60.00 min
Total precip.	= 2.80 in	Distribution	= Type IA
Storm duration	= 24 hrs	Shape factor	= 484



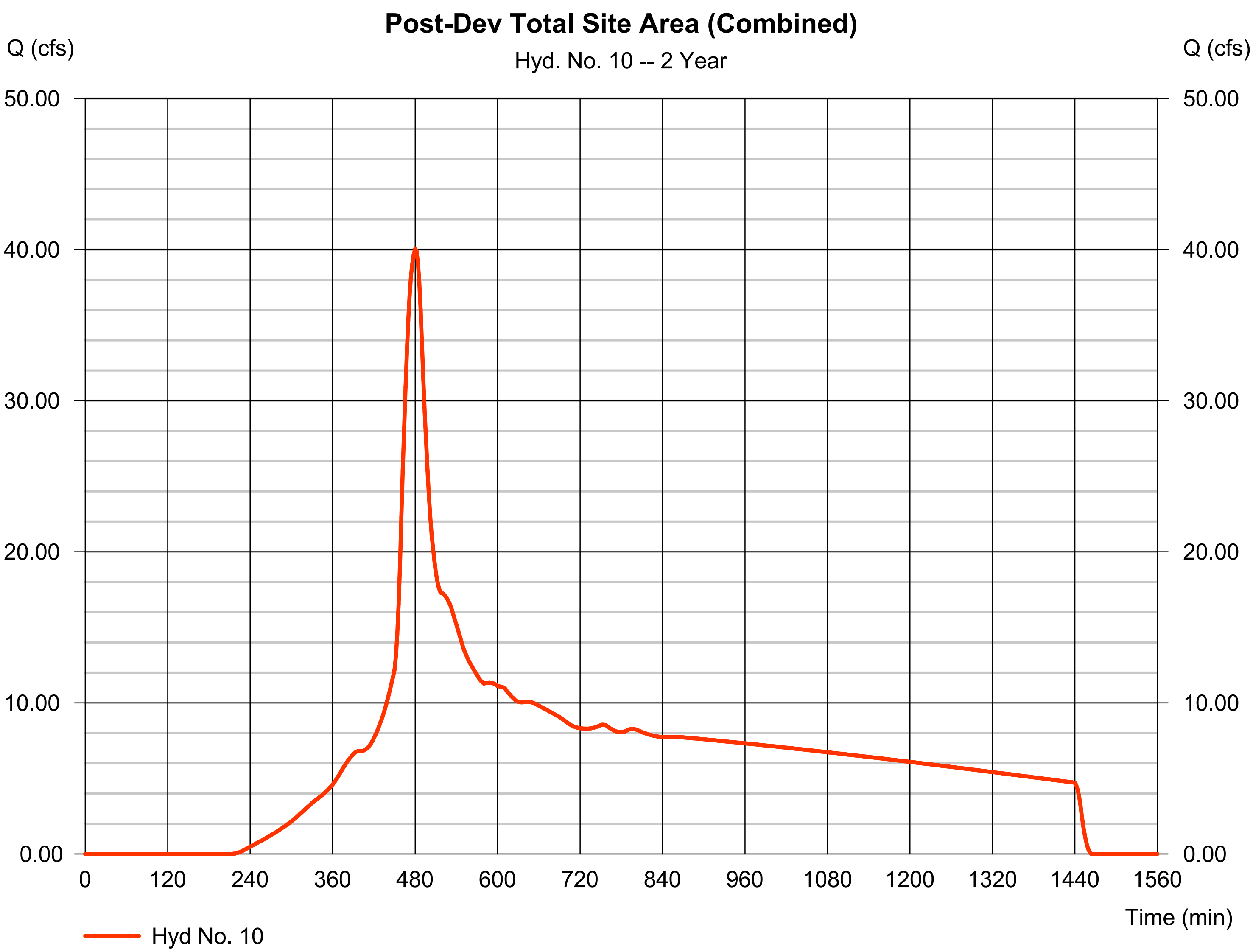
Hydrograph Report

Hyd. No. 10

Post-Dev Total Site Area (Combined)

Hydrograph type	= SCS Runoff	Peak discharge	= 40.05 cfs
Storm frequency	= 2 yrs	Time to peak	= 480 min
Time interval	= 2 min	Hyd. volume	= 595,276 cuft
Drainage area	= 102.460 ac	Curve number	= 88*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 15.00 min
Total precip.	= 2.80 in	Distribution	= Type IA
Storm duration	= 24 hrs	Shape factor	= 484

* Composite (Area/CN) = [(6.410 x 98) + (23.860 x 92) + (27.930 x 98) + (44.260 x 78)] / 102.460

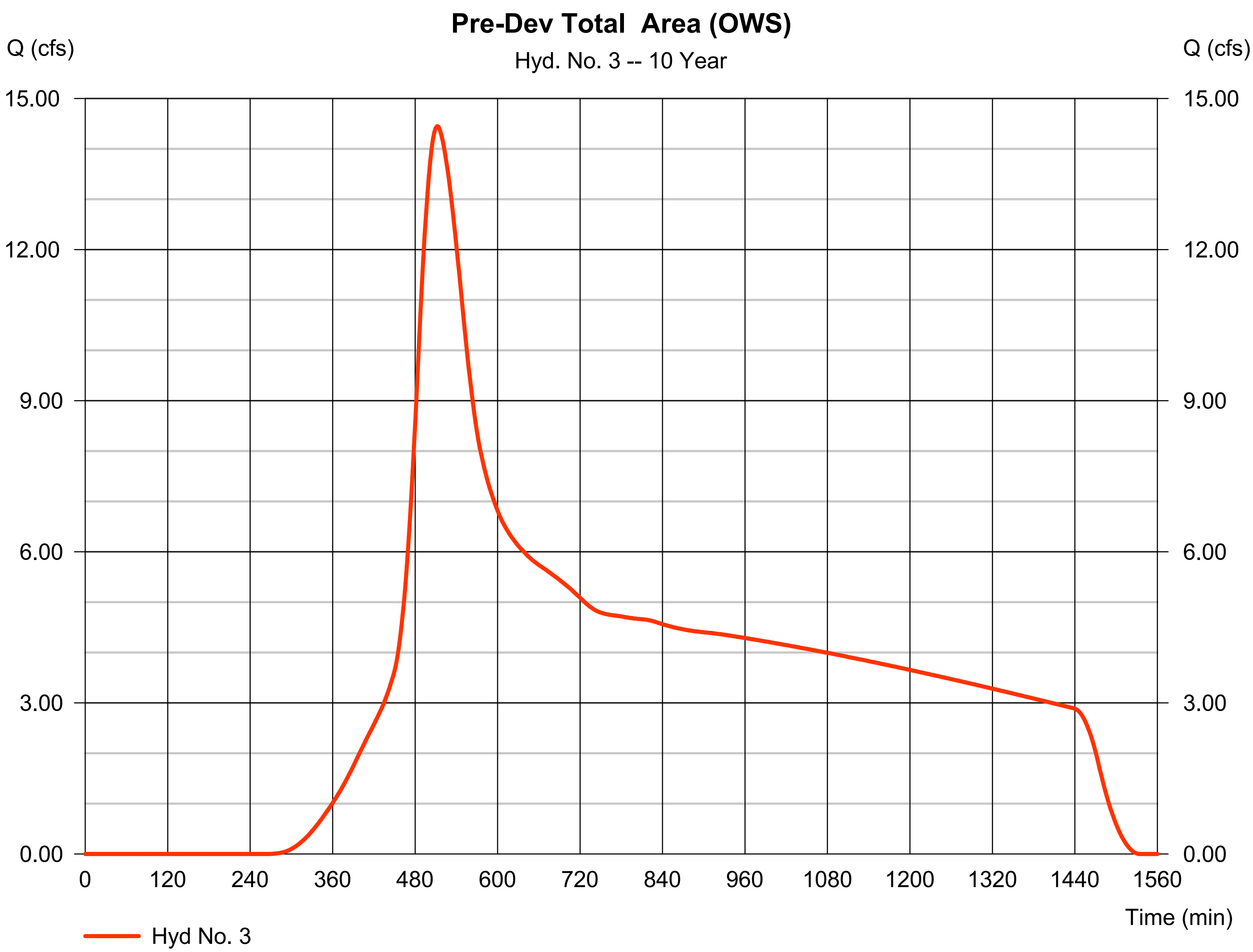


Hydrograph Report

Hyd. No. 3

Pre-Dev Total Area (OWS)

Hydrograph type	= SCS Runoff	Peak discharge	= 14.45 cfs
Storm frequency	= 10 yrs	Time to peak	= 512 min
Time interval	= 2 min	Hyd. volume	= 321,193 cuft
Drainage area	= 45.160 ac	Curve number	= 80
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 60.00 min
Total precip.	= 3.90 in	Distribution	= Type IA
Storm duration	= 24 hrs	Shape factor	= 484



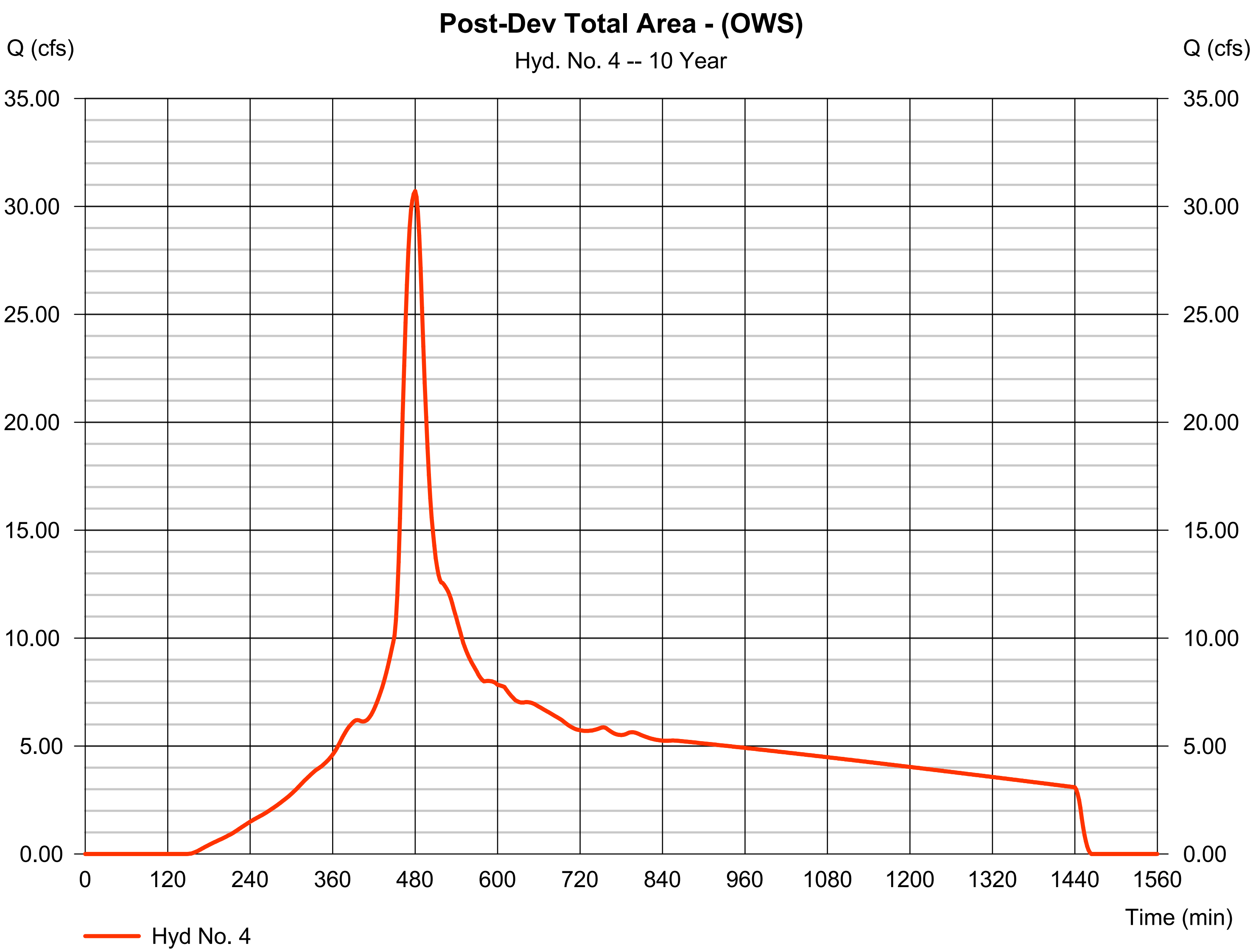
Hydrograph Report

Hyd. No. 4

Post-Dev Total Area - (OWS)

Hydrograph type	= SCS Runoff	Peak discharge	= 30.71 cfs
Storm frequency	= 10 yrs	Time to peak	= 480 min
Time interval	= 2 min	Hyd. volume	= 436,234 cuft
Drainage area	= 45.160 ac	Curve number	= 89*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 15.00 min
Total precip.	= 3.90 in	Distribution	= Type IA
Storm duration	= 24 hrs	Shape factor	= 484

* Composite (Area/CN) = [(2.020 x 98) + (4.460 x 92) + (19.110 x 98) + (19.570 x 78)] / 45.160

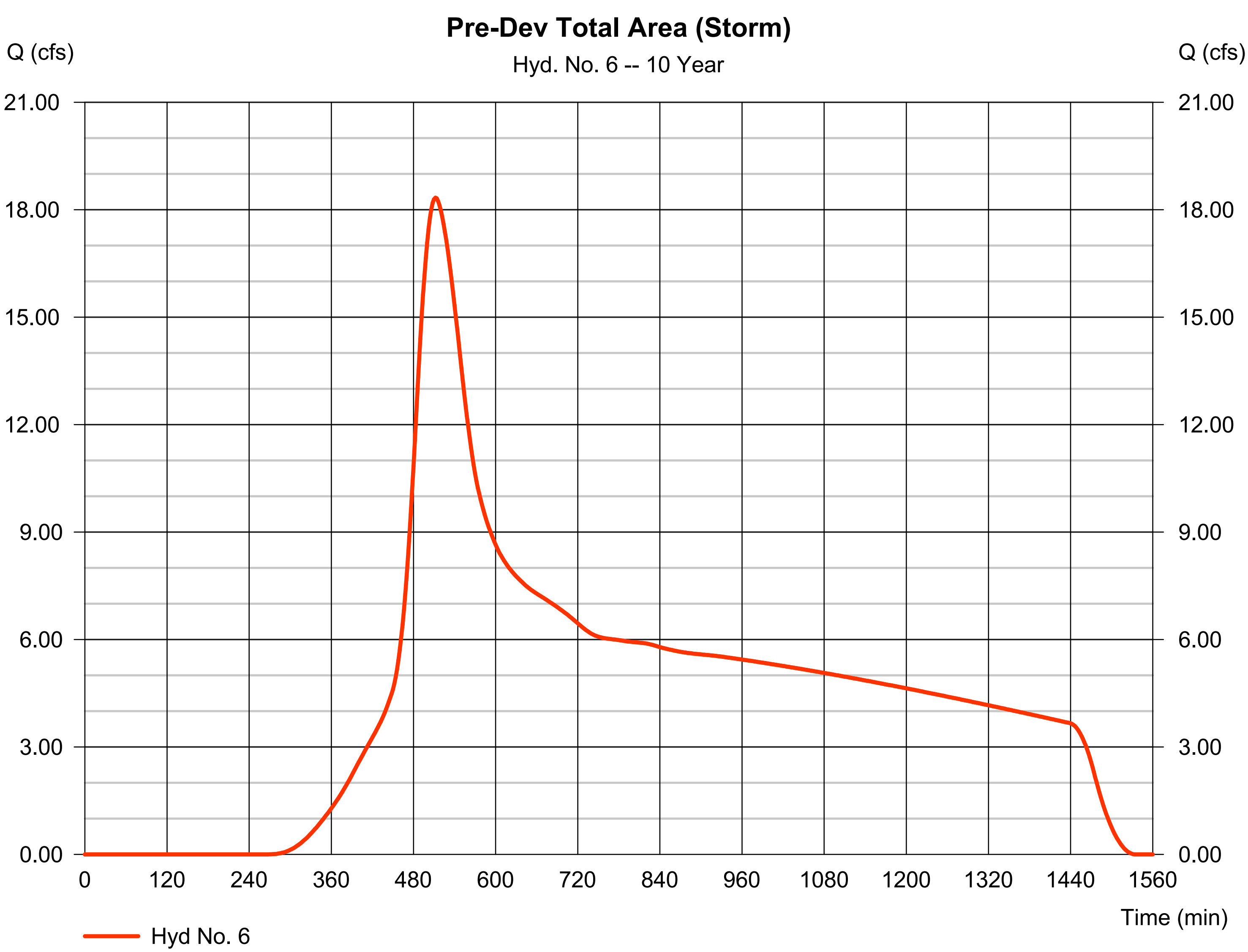


Hydrograph Report

Hyd. No. 6

Pre-Dev Total Area (Storm)

Hydrograph type	= SCS Runoff	Peak discharge	= 18.34 cfs
Storm frequency	= 10 yrs	Time to peak	= 512 min
Time interval	= 2 min	Hyd. volume	= 407,537 cuft
Drainage area	= 57.300 ac	Curve number	= 80
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 60.00 min
Total precip.	= 3.90 in	Distribution	= Type IA
Storm duration	= 24 hrs	Shape factor	= 484



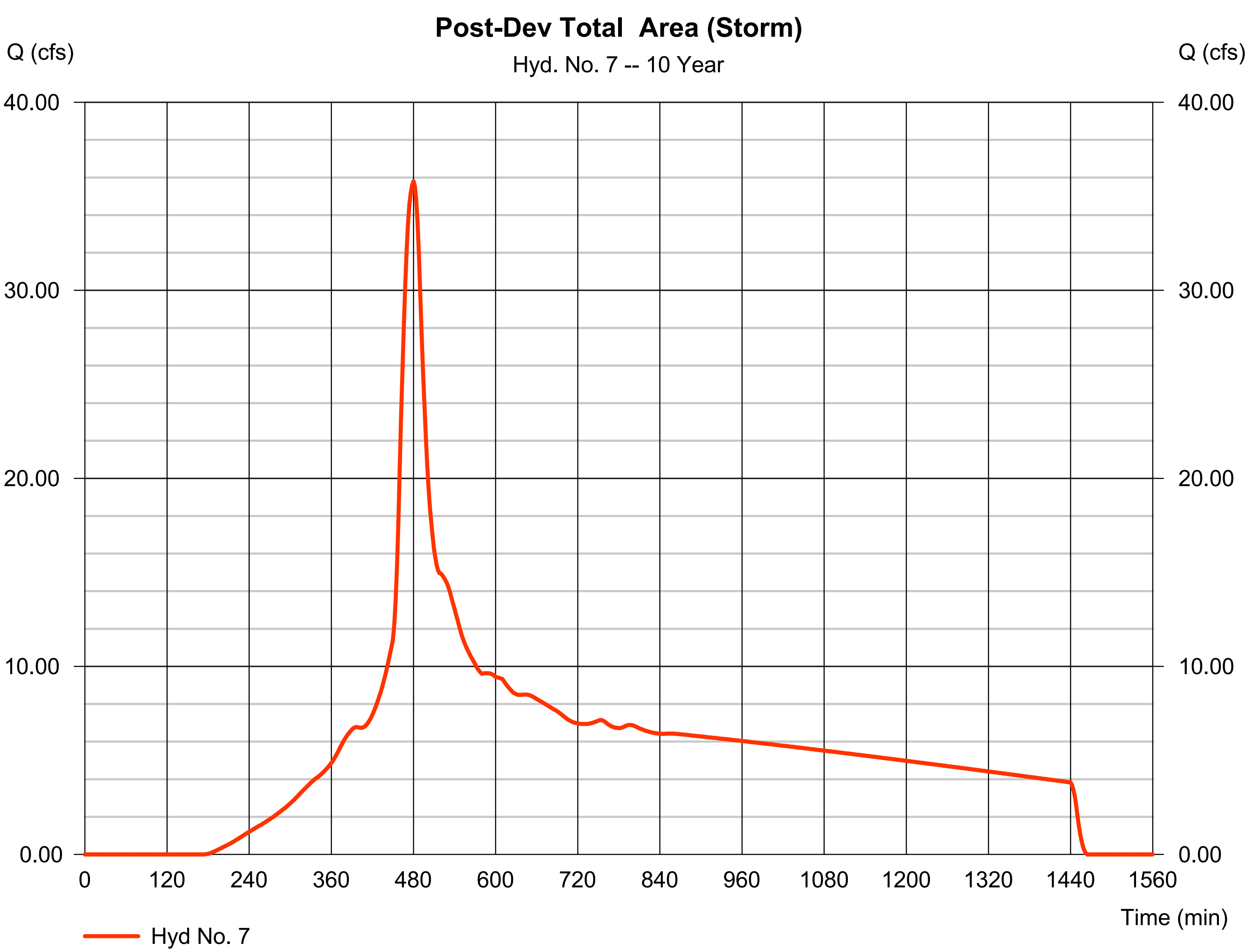
Hydrograph Report

Hyd. No. 7

Post-Dev Total Area (Storm)

Hydrograph type	= SCS Runoff	Peak discharge	= 35.80 cfs
Storm frequency	= 10 yrs	Time to peak	= 480 min
Time interval	= 2 min	Hyd. volume	= 516,142 cuft
Drainage area	= 57.300 ac	Curve number	= 87*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 15.00 min
Total precip.	= 3.90 in	Distribution	= Type IA
Storm duration	= 24 hrs	Shape factor	= 484

* Composite (Area/CN) = [(4.390 x 98) + (19.400 x 92) + (8.820 x 98) + (24.690 x 78)] / 57.300

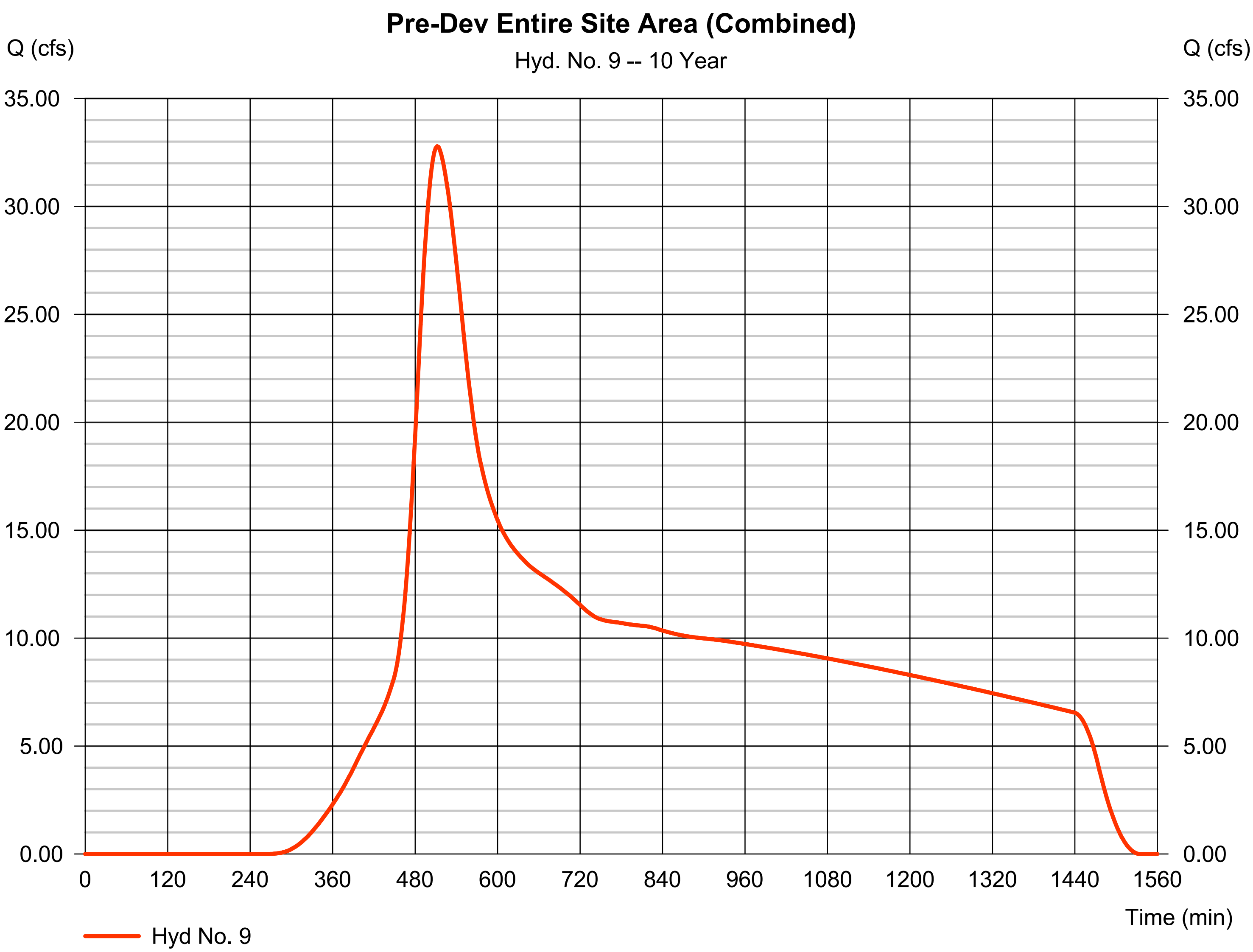


Hydrograph Report

Hyd. No. 9

Pre-Dev Entire Site Area (Combined)

Hydrograph type	= SCS Runoff	Peak discharge	= 32.79 cfs
Storm frequency	= 10 yrs	Time to peak	= 512 min
Time interval	= 2 min	Hyd. volume	= 728,731 cuft
Drainage area	= 102.460 ac	Curve number	= 80
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 60.00 min
Total precip.	= 3.90 in	Distribution	= Type IA
Storm duration	= 24 hrs	Shape factor	= 484



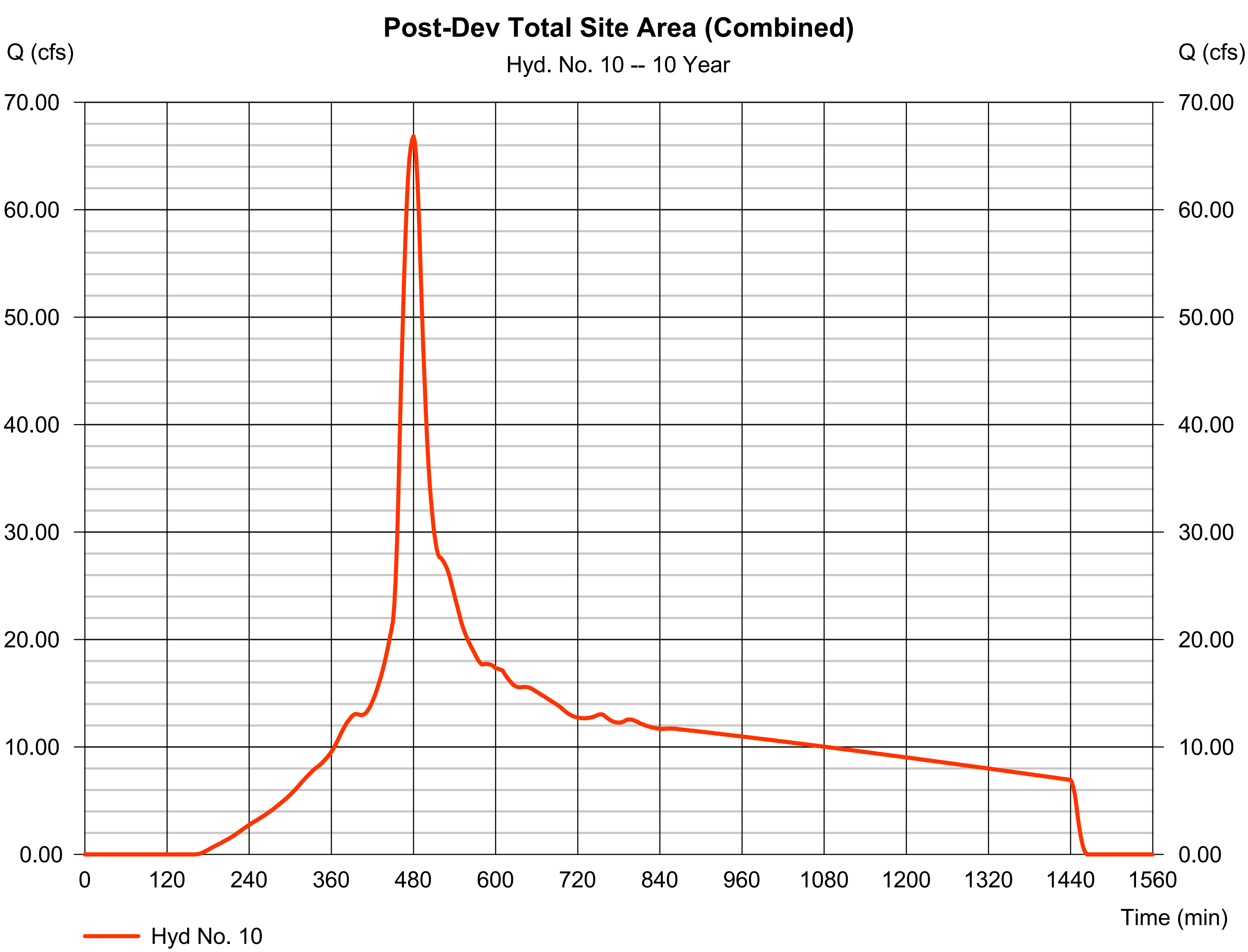
Hydrograph Report

Hyd. No. 10

Post-Dev Total Site Area (Combined)

Hydrograph type	= SCS Runoff	Peak discharge	= 66.85 cfs
Storm frequency	= 10 yrs	Time to peak	= 480 min
Time interval	= 2 min	Hyd. volume	= 955,974 cuft
Drainage area	= 102.460 ac	Curve number	= 88*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 15.00 min
Total precip.	= 3.90 in	Distribution	= Type IA
Storm duration	= 24 hrs	Shape factor	= 484

* Composite (Area/CN) = [(6.410 x 98) + (23.860 x 92) + (27.930 x 98) + (44.260 x 78)] / 102.460

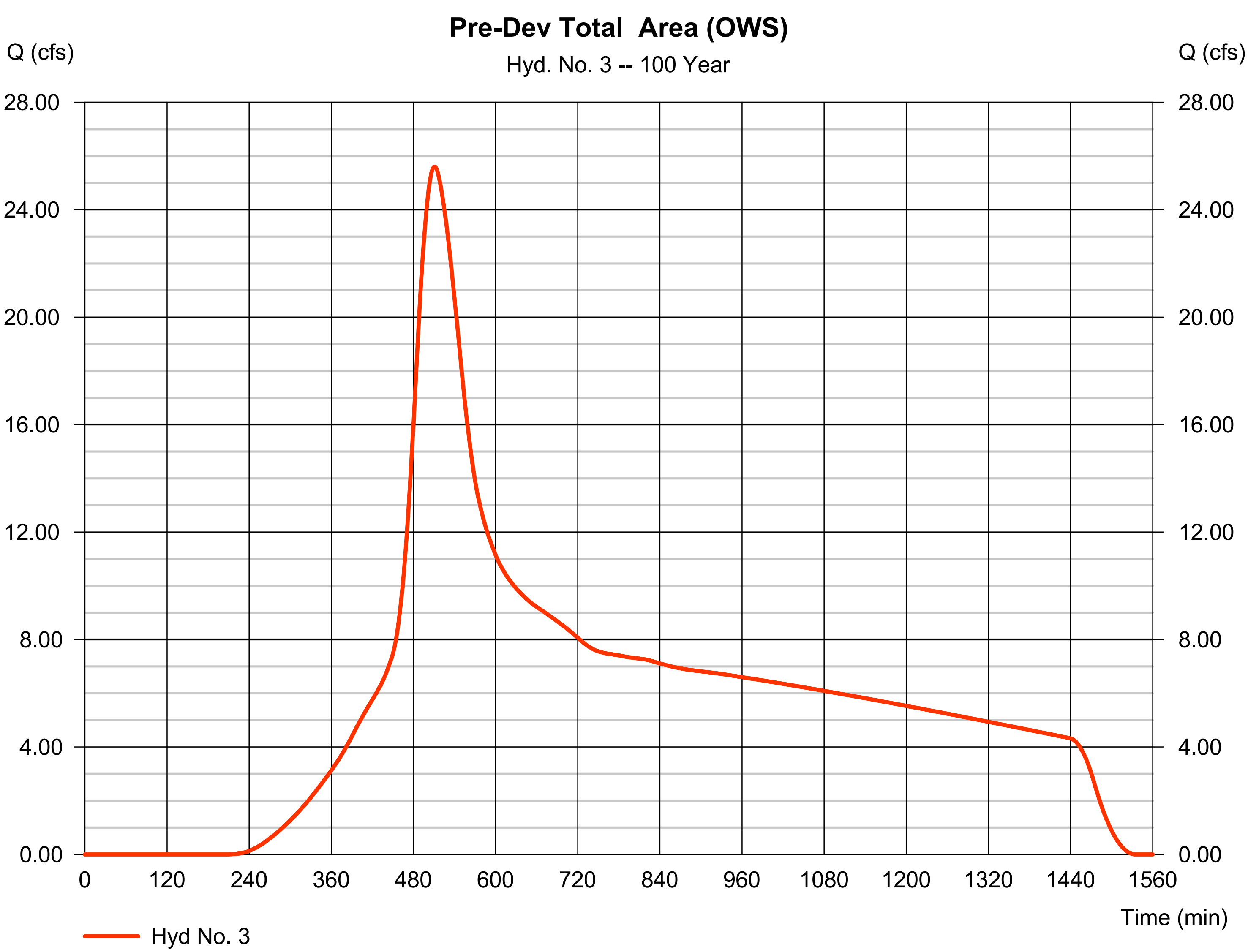


Hydrograph Report

Hyd. No. 3

Pre-Dev Total Area (OWS)

Hydrograph type	= SCS Runoff	Peak discharge	= 25.60 cfs
Storm frequency	= 100 yrs	Time to peak	= 510 min
Time interval	= 2 min	Hyd. volume	= 531,889 cuft
Drainage area	= 45.160 ac	Curve number	= 80
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 60.00 min
Total precip.	= 5.40 in	Distribution	= Type IA
Storm duration	= 24 hrs	Shape factor	= 484



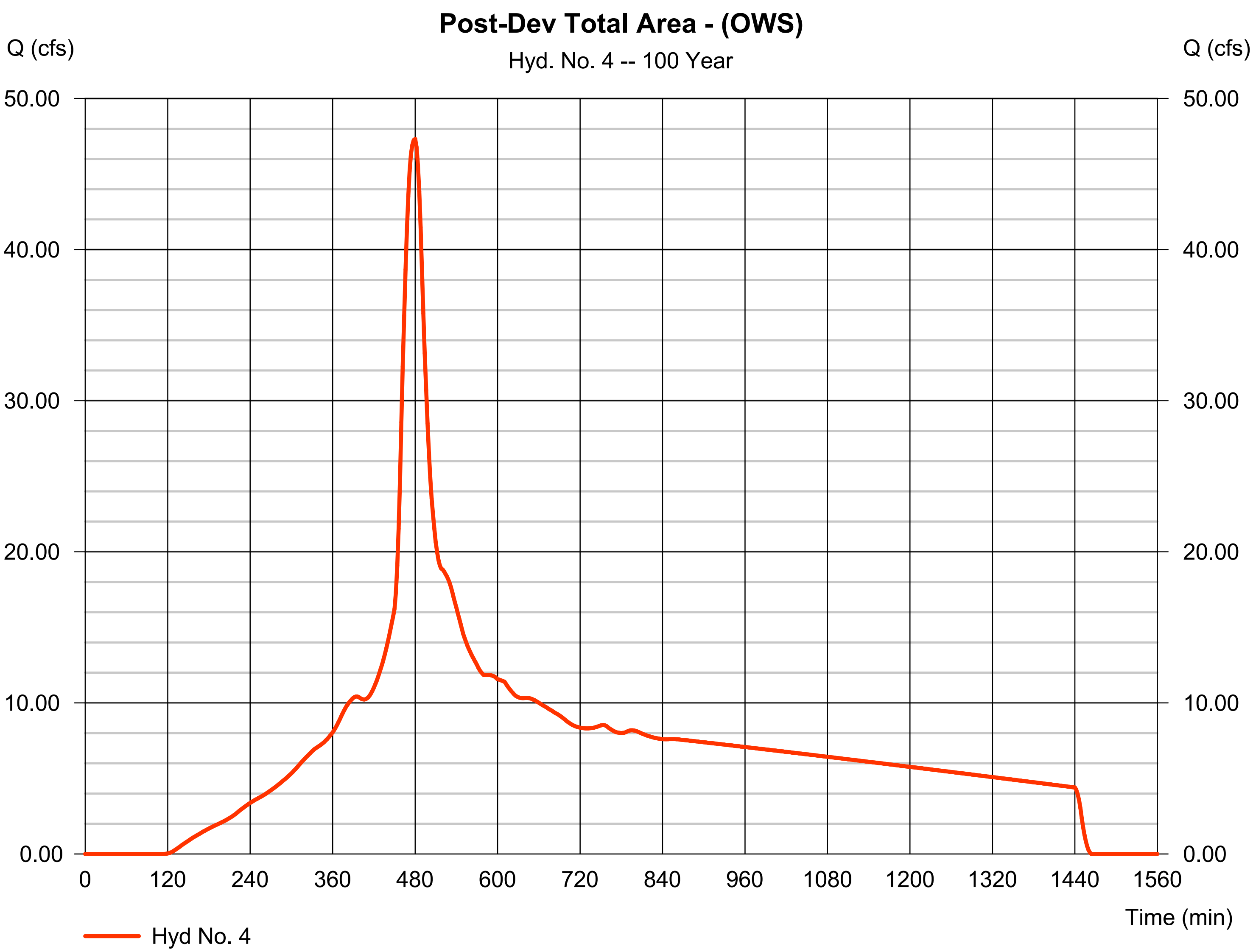
Hydrograph Report

Hyd. No. 4

Post-Dev Total Area - (OWS)

Hydrograph type	= SCS Runoff	Peak discharge	= 47.32 cfs
Storm frequency	= 100 yrs	Time to peak	= 480 min
Time interval	= 2 min	Hyd. volume	= 664,258 cuft
Drainage area	= 45.160 ac	Curve number	= 89*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 15.00 min
Total precip.	= 5.40 in	Distribution	= Type IA
Storm duration	= 24 hrs	Shape factor	= 484

* Composite (Area/CN) = [(2.020 x 98) + (4.460 x 92) + (19.110 x 98) + (19.570 x 78)] / 45.160

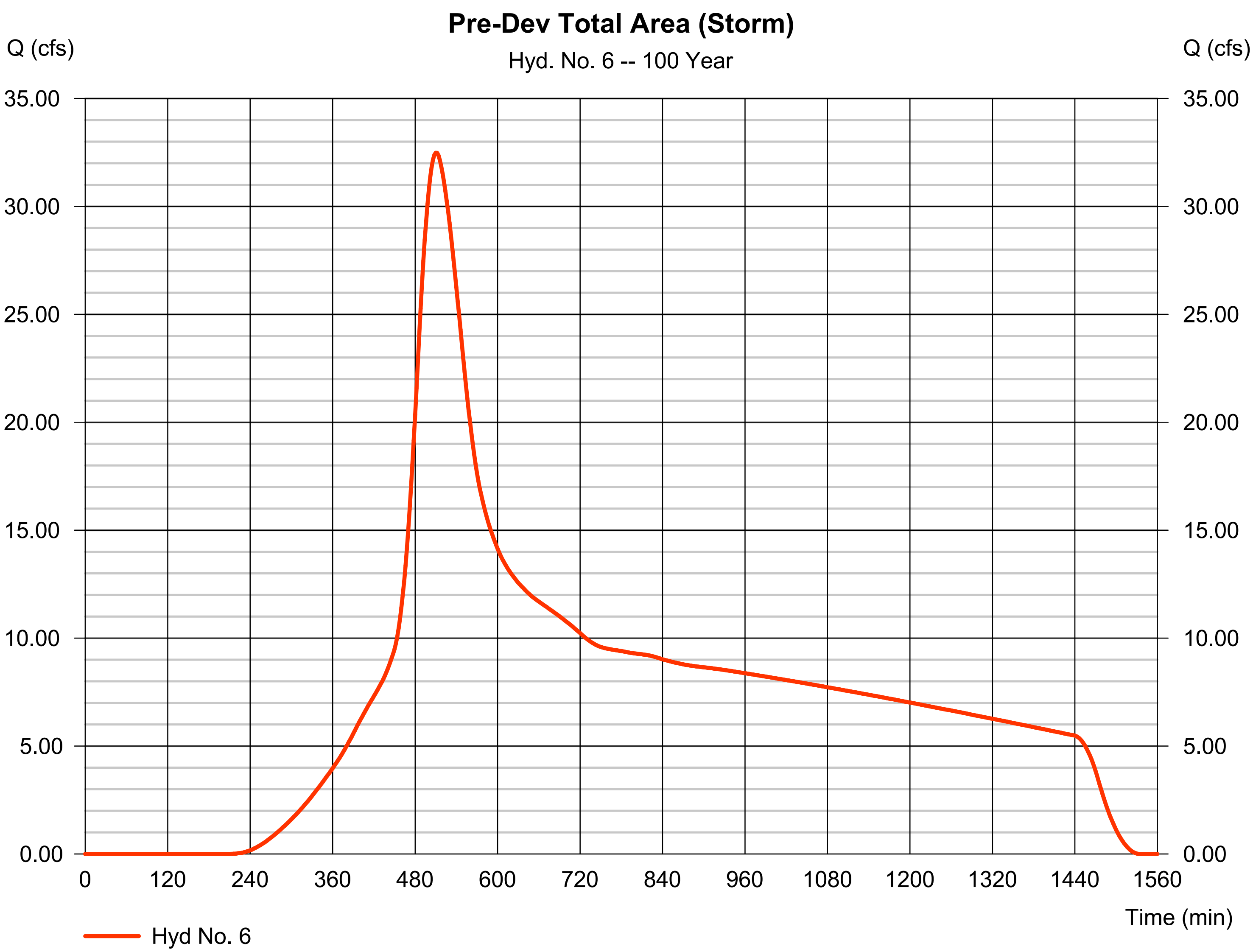


Hydrograph Report

Hyd. No. 6

Pre-Dev Total Area (Storm)

Hydrograph type	= SCS Runoff	Peak discharge	= 32.48 cfs
Storm frequency	= 100 yrs	Time to peak	= 510 min
Time interval	= 2 min	Hyd. volume	= 674,873 cuft
Drainage area	= 57.300 ac	Curve number	= 80
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 60.00 min
Total precip.	= 5.40 in	Distribution	= Type IA
Storm duration	= 24 hrs	Shape factor	= 484



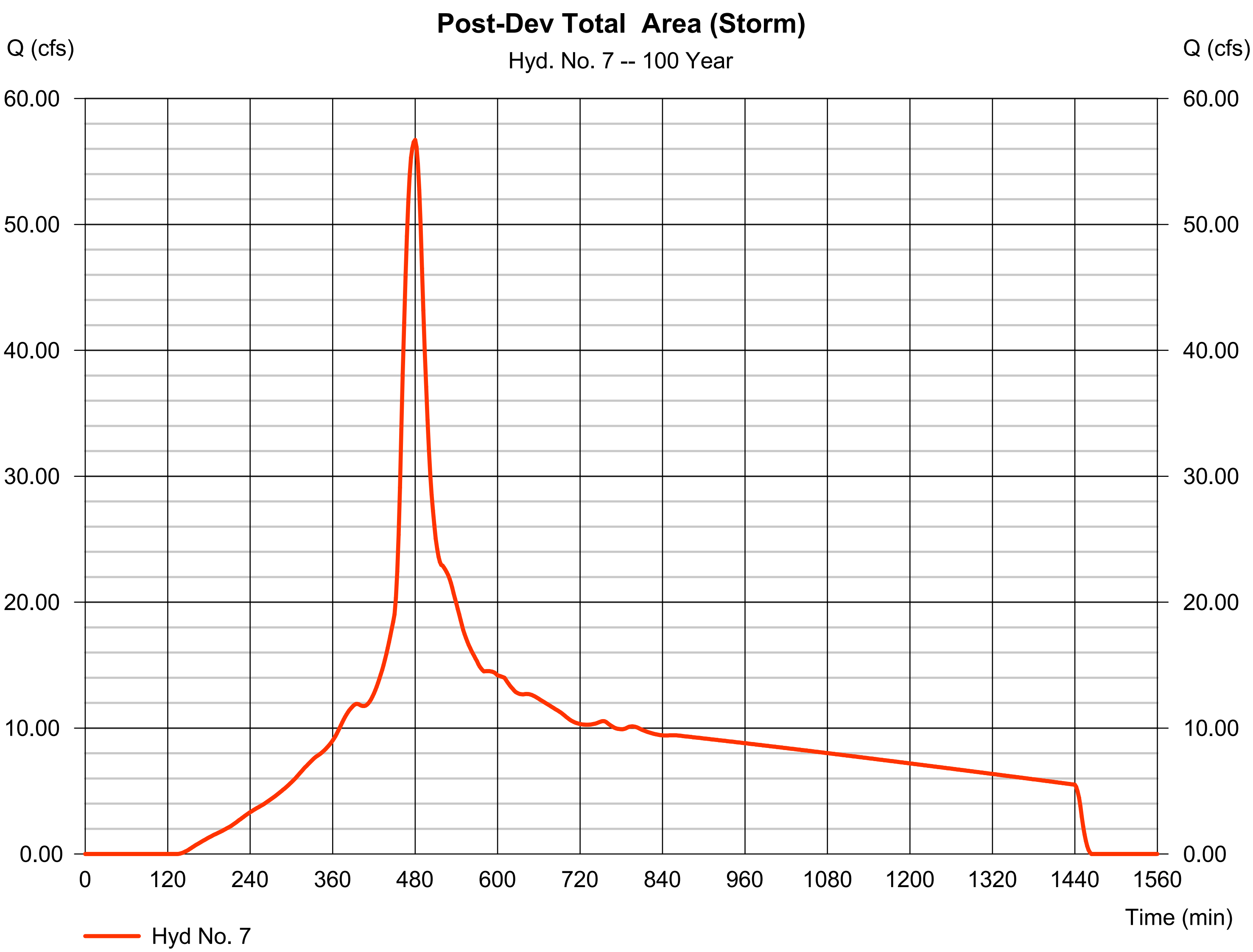
Hydrograph Report

Hyd. No. 7

Post-Dev Total Area (Storm)

Hydrograph type	= SCS Runoff	Peak discharge	= 56.71 cfs
Storm frequency	= 100 yrs	Time to peak	= 480 min
Time interval	= 2 min	Hyd. volume	= 800,130 cuft
Drainage area	= 57.300 ac	Curve number	= 87*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 15.00 min
Total precip.	= 5.40 in	Distribution	= Type IA
Storm duration	= 24 hrs	Shape factor	= 484

* Composite (Area/CN) = [(4.390 x 98) + (19.400 x 92) + (8.820 x 98) + (24.690 x 78)] / 57.300

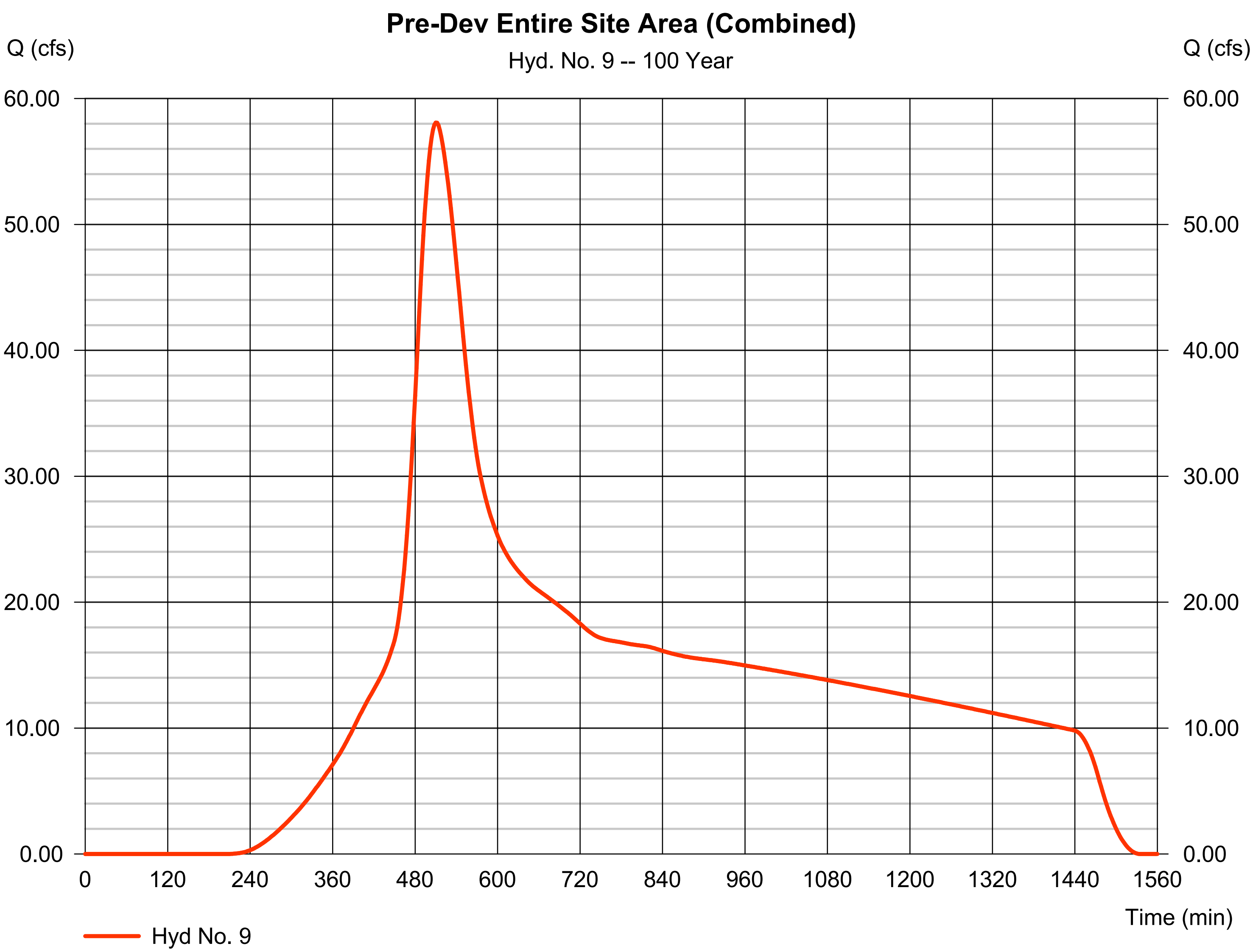


Hydrograph Report

Hyd. No. 9

Pre-Dev Entire Site Area (Combined)

Hydrograph type	= SCS Runoff	Peak discharge	= 58.08 cfs
Storm frequency	= 100 yrs	Time to peak	= 510 min
Time interval	= 2 min	Hyd. volume	= 1,206,761 cuft
Drainage area	= 102.460 ac	Curve number	= 80
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 60.00 min
Total precip.	= 5.40 in	Distribution	= Type IA
Storm duration	= 24 hrs	Shape factor	= 484



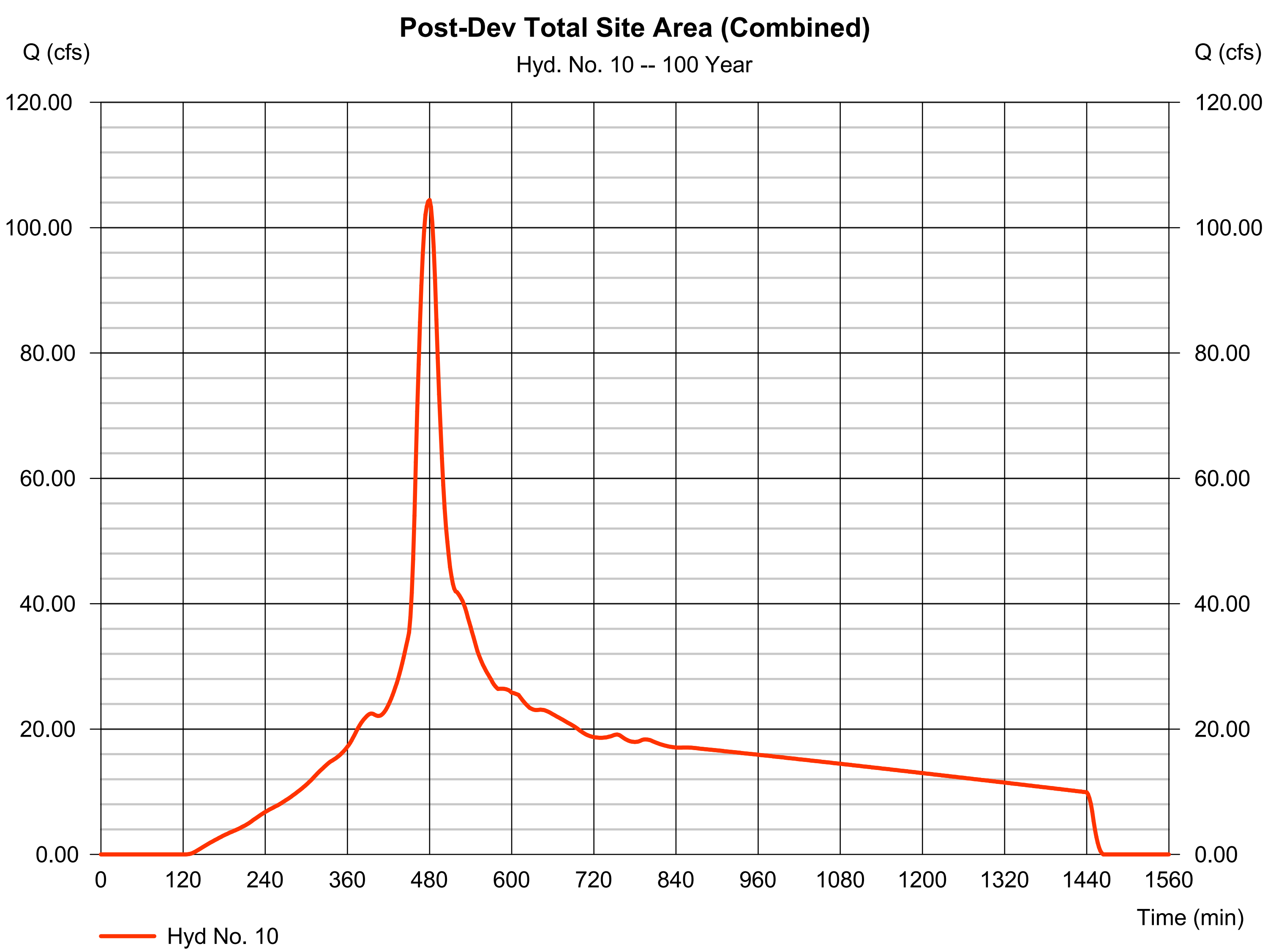
Hydrograph Report

Hyd. No. 10

Post-Dev Total Site Area (Combined)

Hydrograph type	= SCS Runoff	Peak discharge	= 104.42 cfs
Storm frequency	= 100 yrs	Time to peak	= 480 min
Time interval	= 2 min	Hyd. volume	= 1,468,698 cuft
Drainage area	= 102.460 ac	Curve number	= 88*
Basin Slope	= 0.0 %	Hydraulic length	= 0 ft
Tc method	= User	Time of conc. (Tc)	= 15.00 min
Total precip.	= 5.40 in	Distribution	= Type IA
Storm duration	= 24 hrs	Shape factor	= 484

* Composite (Area/CN) = [(6.410 x 98) + (23.860 x 92) + (27.930 x 98) + (44.260 x 78)] / 102.460

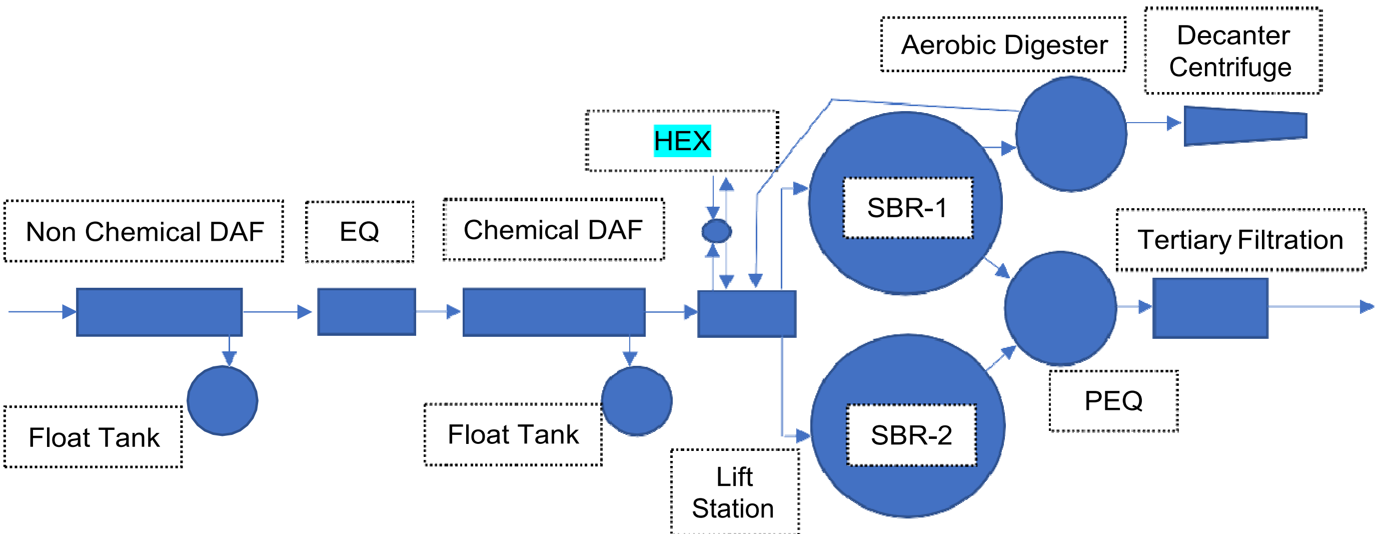


APPENDIX E

WASTEWATER TREATMENT PLANT DESIGN INFORMATION

Process Design

The following process flow diagram (PFD) shows the key unit processes involved in the recommended wastewater treatment plant for the NEXT Renewable Diesel project.



The purpose of this section is to provide the rationale for the selected design approach. Influent flow from the Alfa Laval pretreatment facility would be directed to a packaged, non-chemical dissolved air flotation unit (DAF). This DAF would be designed to treat 125 gpm containing less than 5% FOG. The non-chemical DAF reduces the FOG load to the chemical DAF and produces a float of fatty matter that could have a residual value and may even be returned for processing. Effluent from the non-chemical DAF would be equalized and pumped to a packaged, chemical DAF. In this unit chemical would be added to free emulsified oil and to provide coagulation for better flocculation and flotation/removal.

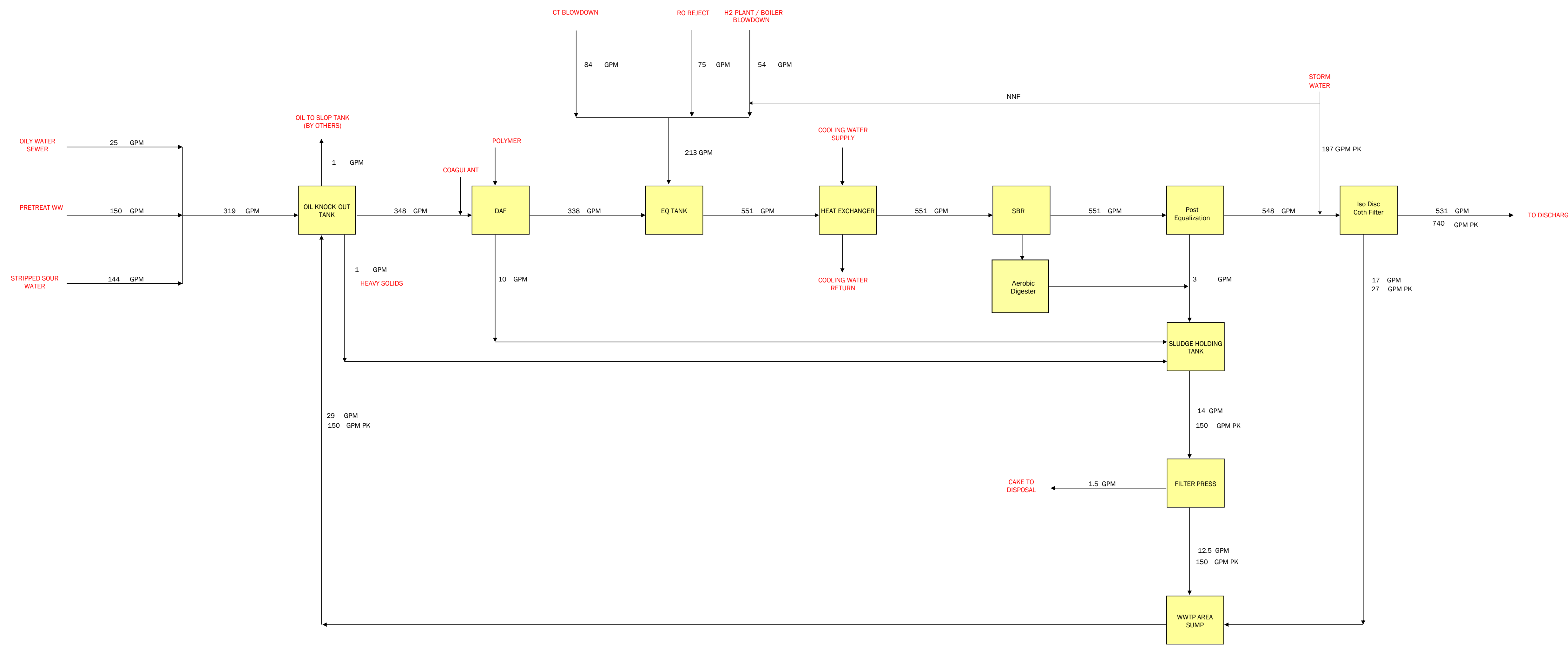
From the two-stage DAF system the flow would undergo cooling to achieve a target MLSS temperature of 35 °C and then be lifted by a pump station to the sequencing batch reactor (SBR) system. The SBR technology is very well suited for handling the variable flows and loads that would be encountered. The goal is to be able to effectively treat flows that include the majority of the higher strength stream from the pretreatment primary centrifuge, but also able to effectively treat flows where the primary centrifuge stream may be excluded from the wastewater treatment process. The cycle structure of the SBR provides optimum treatment of this type of wastewater due to 1) initial mix-only feed cycle provides for hydrolyzation of any residual fatty matter to easily biodegraded fatty acids and glycerol; 2) the cycling provides optimum conditions for biological phosphorus removal; and 3) subsequent aeration and mixing cycles provides full biological treatment to meet the organics and total nitrogen removal requirements. Another inherent benefit of the SBR technology is that the cycling of oxic and anoxic conditions favors good settling bacteria and minimizes formation of aerobic filamentous growth, which can occur in these type systems.

Gravity decant from the SBR would flow into a post equalization basin and then pumped at a steady rate to a tertiary filtration system. The Isodisk filtration system is designed to provide significant suspended solids removal insurance against process upsets that could occur due to conditions such as pH or temperature imbalances.

FOG residuals from the DAF system would be collected in tanks as shown on the PFD. The material in these tanks would be either hauled offsite for disposal, sold, and or reprocessed.

Waste activated sludge (WAS) from the SBR system would be directed by pumps to the aerobic digester for solids stabilization and thickening. The sludge from the aerobic digester could be liquid hauled for disposal or dewatered using a decanter/centrifuge. The solids from dewatering would have significantly less volume and could readily be transported offsite for disposal.

Filter backwash water would be directed to the SBR lift station for collection as WAS and directed to the aerobic digester for solids handling.



REV. NO.	ISSUE DATE	DRAWN BY	CHECKED BY	REVIEWED BY	APPROVED BY	APPROVED BY (OPTIONAL)	DESCRIPTION
	12/15/2020						
A	10/30/2018	BF					BUDGET PROPOSAL

DESIGN/DRAFTING — ENGINEERING

DRAWINGS WITH ALPHA REVISIONS ARE PRELIMINARY AND SHALL NOT BE USED FOR FINAL DESIGN WORK OR FABRICATION.

NEXT Renewable Fuels, Inc.

SCALE | DRAWN BY

TITLE

NEXT Renewable Fuels
WWT Block Flow Diagram

CONTRACT NO. 5600118065 | DWG. NO. | REV A

AS-H Iso-Disc® Cloth Media Filter

Tertiary filtration and final polishing in municipal and industrial applications for high quality effluent discharge or reuse

The Alfa Laval AS-H Iso-Disc is a cloth media polishing filter that enables continuous production of high-purity, reuse-quality filtrate from various applications. The filter removes organic and inorganic pollutants such as residual suspended solids e.g. to <10 mg/l 95%ile (<5 mg/l 30 day average).

Iso-Disc is a compact, cost efficient alternative to traditional sand filters and other disc filter technologies. It offers a robust yet simple design that can handle peak hydraulic loadings up to 15 $m^3/h/m^2$ which equates to single small footprint units capable of 5 – 800 m^3/hr. The performance of the Iso-Disc filter is second to none. The standard cloth media is capable of solids capture to less than 10 microns, with the advantage of outside-in depth filtration for handling high solids loadings.

Applications

- Tertiary filtration of municipal and industrial wastewater
- Water reuse
- Process streams
- Surface water treatment e.g. for cooling towers and process water
- Pre-treatment for technologies to produce high purity water
- All industries that require water filtration

Benefits

- Individual filter element monitoring
- Individual filter element maintenance
- Simple, robust and efficient design
- Reuse quality filtrate and California Title 22 Water Reuse Certification
- Fully automatic outside-in depth filtration
- Small footprint-to-flow ratio
- Easily expanded
- Low backwash water rates
- High hydraulic loading capacities
- Uninterrupted operation during backwashing cycle
- High hydraulic and solids loading rates
- Simple internal emergency flow bypass

The filter can be installed into a concrete structure, carbon steel tank with coating, or a stainless steel tank, and can easily be retrofitted into existing tanks.

Design features

The Alfa Laval AS-H Iso-Disc Cloth Media Filter is engineered as a continuous operating process that utilizes a fixed cloth media and an efficient linear backwashing system that cleans the media equally across the whole surface area. The cloth media is fully submerged into a tank to allow 100% use of filtration area at all times.

An Iso-Disc filter incorporates a number of hollow filter elements, designed to handle the actual flow and load conditions. The elements are mounted in a “cassette frame” within the path of the incoming water. Both square and rectangular cassettes are available to cater for different installation configurations.

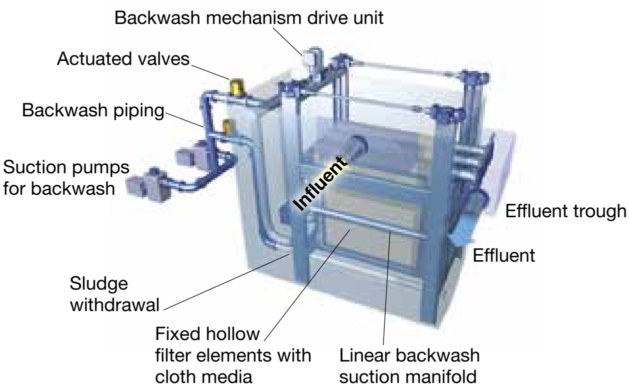
All submerged components are corrosion resistant stainless steel or non-metallic materials.

The design of the Iso-Disc allows for individual visual assessment of the effluent flow rate and quality. The filter cloth can be replaced while the filter continues to function without interruption.

As the filtration area is static, there are no rotating seals which can lead to cross contamination if worn.The simple nature of the Iso-Disc and minimal moving parts ensure that maintenance requirements are kept to a minimum.

Working principles

The Alfa Laval AS-H Iso-Disc Cloth Media Filter operates continuously in an outside-in flow pattern. The cloth media is mounted on the outsides of a hollow filter element which allows the water to pass through the cloth into the centre of the hollow element by gravity. As water passes through the cloth media, the particulate solids are captured on the outer surface of the cloth. The filtered water exits the element at a high level discharge port which directs the water into a collection trough.



Vacuum cleaning of the cloth media (Backwashing)

With time, the captured solids progressively build up on the outside of the cloth media and slowly generate resistance to the water flow, causing the water level to rise in the tank. A sensor monitors the water level, and at a preset high level, a backwash is instigated to clean the cloth media and remove the captured solids.

Unlike other fine solids filtration systems, Iso-Disc uses fixed elements with static filtration media and achieves backwashing by moving a horizontal, bi-directional backwash suction manifold up and down each element. A centrifugal pump generates vacuum at the backwash suction manifold/cloth media surface, which gently relieves the cloth of its solids load via the backwash manifold. As a result, the resistance to water flow is removed, and water level falls in the tank as filtration continues.

When the water level within the tank reaches a predetermined high water level, a simple PLC control system will intiate a backwash event. Actuated valves between the backwash manifolds and the backwash pump open and close in a programmed sequence to facilitate backwashing of the individual filter elements, one at a time. The manifold is driven up and down the elements using a single electric motor drive and four corner mounted gearboxes. This ensures complete vacuum cleaning of the filter cloth while minimizing the rate at which backwash water is returned to the treatment facility. The efficient cleaning system ensures the minimal number of backwashes per hour.

At the end of each backwashing cycle, the pump, valves and manifolds are parked until the next backwash is required at high water level. After an operator selected number of operation hours or a set number of backwash events, a sludge withdrawal valve will open and remove sediment from the bottom of the tank.

The Iso-Disc operation is self-managing; as flow and load increases or decreases, the backwashing frequency naturally compensates to maintain steady state filtration conditions. At times of high flow and load, the backwash frequency will increase. Reduced flow and load will result in longer periods between backwashing. In all cases, the cloth media polishing filter remains in operation during backwashing.

Standard filter element sizes and flow capacities

Filter element dimension

Model		Number of elements per filter cassette	Average flow capacity
0.6 x 0.6 m	(24” x 24”)	2 and 3	5.5 m^3 / hr / element
0.9 x 0.9 m	(36” x 36”)	3, 5 and 7	12.3 m^3 / hr / element
1.5 x 1.5 m	(60” x 60”)	3, 4, 5, 6, 7, 8 and 9	34.1 m^3 / hr / element
1.5 x 2.4 m	(60” x 96”)	7	54.5 m^3 / hr / element
1.8 x 2.4 m	(72” x 96”)	7	65.4 m^3 / hr / element

Alfa Laval reserves the right to change specifications without prior notification.

How to contact Alfa Laval
Up-to-date Alfa Laval contact details for all countries are always available on our website at www.alfalaval.com

APPENDIX F

GEOTECHNICAL ENGINEERING REPORT

Harza Engineering Company
Fossil Power Business Unit
Two Honey Creek Corporate Center
115 South 84th Street, Suite 200
Milwaukee, Wisconsin

GEOTECHNICAL EVALUATION
SUMMIT/WESTWARD ENERGY PROJECT
CLATSKANIE, OREGON

May 2001

REGISTERED PROFESSIONAL ENGINEER
58371PE
OREGON
JULY 13, 1999
MITCHELL F. SCHAUB
Expires: 06/30/02

Mitchell F. Schaub, P.E.
Project Engineer

REGISTERED PROFESSIONAL ENGINEER
9057
OREGON
DEC. 17, 1976
GERALD L. JACKSHA
EXPIRES 06/30/02

Jerry L. Jacksha, P.E.
Principal Engineer

Arlan H. Rippe, P.E.
President

201578

Squier Associates, Inc.
4260 Galewood Street
Lake Oswego, Oregon 97035
Phone: (503) 635-4419
Fax: (503) 635-1430

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GEOTECHNICAL EVALUATION
SUMMIT/WESTWARD ENERGY PROJECT
CLATSKANIE, OREGON

1.0 INTRODUCTION

This geotechnical evaluation report has been completed for the proposed Summit/Westward Energy Project, which includes a new natural gas-fired combined-cycle combustion turbine generation facility located near Clatskanie, Oregon. The project site is located on Port of St. Helens property located in Columbia County approximately seven miles northeast of Clatskanie, Oregon. The Vicinity Map, Figure 1, shows the location of the project site.

The purpose of this evaluation was to present findings regarding the geologic and seismic setting of the project site; assess the nature of the subsurface conditions and materials which underlie the project site including site specific seismic evaluation; develop preliminary conclusions concerning the key geotechnical aspects of the project, such as foundations for the turbines/generators and other settlement sensitive facilities; seismic design considerations; and related site geotechnical issues. This report also contains "site specific geological and soil stability assessment" information pertinent to site certificate application, Exhibit H, requirements by Oregon Department of Energy, Energy Facility Siting Council.

2.0 LIMITATIONS

The scope of the geotechnical evaluation presented herein is limited to the assessment of geologic site-specific conditions and evaluation of the subsurface conditions related to the proposed facilities for the Summit/Westward Energy Project near Clatskanie, Oregon. This report has been prepared to aid Harza Engineering Company, Milwaukee, Wisconsin and the project owner in the evaluation of the site and application for site certificate for the proposed facility in accordance with generally accepted engineering geologic and geotechnical engineering practices. No other warranty, based on the contents of this report is intended, and none shall be inferred from the statements or opinions expressed herein.

Our description of the project represents our understanding of the significant aspects of the project relevant to the general arrangement of the project and the proposed site layout provided by Harza Engineering. In the event that any changes in the proposed locations of the structures as outlined in this report are planned or occur, we recommend that a geotechnical review of the changes be made to affirm in writing the conclusions of this report.

The scope of our services reported herein included environmental field screening of the near surface soils to a depth of 15 feet below the current ground surface for the presence of certain soil contaminants. Any statement in this report or on the boring logs regarding odors noted or unusual or suspicious items or conditions observed are solely for the information of our client.

The analyses and conclusions represented in this report are based on the data obtained from the borings made at the locations indicated on the Boring Location Site Map, (Figure 2) and from other information discussed herein. This report is based on the assumption that the subsurface conditions across the site are not significantly different from those revealed by the borings. However, variations in soil conditions may exist between the borings locations. The nature and extent of the variations may not become evident until further investigations are made at the site during the design phase or during construction.

The exploratory activities, laboratory testing, and preliminary analysis are consistent with those normally used in conceptual or preliminary geotechnical evaluations and for site characterizations to develop budgets for future design and construction. When concepts have been better defined, additional explorations and analyses will be necessary to complete the geotechnical analysis and to provide design recommendations.

3.0 BACKGROUND INFORMATION

3.1 Topography

The site is located in the Oak Point 7½-minute quadrangle (U.S. Geological Survey, 1985). The proposed generation facilities site is a relatively flat, vegetation covered pasture land with shallow drainage ditches containing water generally to the south and east of the proposed main plant facilities. The greatest relief on the site is related to the existing drainage ditches, which are less than 10 feet deep with associated spoil piles from the ditch excavations. The ground surface on the site varies between elevations 5 and 10 feet, based on North American Vertical Datum (NAVD) 1988, according to the contours shown on Figure 2. The topography north of the site remains flat for a distance of approximately 2000 feet to a levee that bounds Bradbury Slough, a side channel of the Columbia River.

3.2 Project Description

The proposed project layout map is shown on Figure 2. Along the northeast border of the property are existing gas lines and power lines and a railroad spur. We understand that the generation facility will contain the following major components:

- Combustion Turbines and Generators;
- Heat Recovery Steam Generators (HRSGs);
- A Steam Turbine, Condenser and Generator;
- Main Power Transformers;
- Miscellaneous Buildings;
- Multi-cell Cooling Tower Complex;
- Water Storage Tanks;
- In-plant Substation and Switch Yard; and
- Pipes, Conduits, and Pipe Racks.

We understand there also will be numerous buried utilities and associated underground vaults constructed across the proposed plant site to depths up to 20 feet. Large diameter underground pipelines will be installed between the cooling tower and the steam generator. We understand the orientation of the structures shown on Figure 2 could change, but the general spacing or relative location will remain similar.

4.0 GEOLOGIC SETTING

The information in this section represents a summary of the geologic setting information presented in Appendix D.

4.1 Regional and Site Geology

The Summit/Westward Energy Project site and its related/supporting facilities are located on the Columbia River alluvial valley within the Coast Range physiographic province of northern Oregon and southern Washington. A physiographic province is a region of similar geologic history and composition. The Coast Range province is broadly upwarped, forming a low mountain range located between the Pacific Ocean and coastline on the west and Willamette Valley-Puget Sound Lowlands on the east. The general geology in the vicinity of the project area is shown in Appendix D, Geology Map, Figure H3 (Walsh and others, 1987 and Walker and MacLeod 1991). The region is underlained from oldest to youngest: basement rock of

Eocene epoch age volcanic sea floor basalt and island volcanic centers; a thick marine sedimentary sequence of younger Oligocene to Miocene; Miocene epoch Columbia River Basalt lava flows; and local younger alluvial deposits along the Columbia River, coastal rivers and bays. The Eocene volcanic rock basement is estimated to be about 20 miles thick under the Oregon Coast Range (Orr and Orr, 1996). The overlying marine sedimentary sequence is at least 5,000 feet thick and the Columbia River Basalt 1,400 feet thick in the northern Oregon Coast Range (Beaulieu, 1973). The alluvial sediments may be about 350 feet thick.

Following the cessation of Columbia River Basalt volcanism, the Coast Range began to uplift. Concurrently, the eastern and western margins began to subside and sedimentation resumed along the eastern and western margin of the uplift. As the uplift continued, the erosive power of the Columbia River was able to maintain its course through the growing mountain range.

During the Pleistocene (2 million years) (Orr and Orr, 1996), major continental glaciers periodically formed over much of Canada and Europe. At glacial maximums, vast quantity of water was locked up in glacial ice, which caused 300 to 450 feet lowering of sea level (Balwin, 1964). During these times, the Columbia River eroded a deep channel. The eroded Pleistocene Columbia River channel was probably greater than 350 feet deep at the project site.

During glacial maximums, glacial ice advance blocked the Clark Fork River in northern Idaho and northwestern Montana. Water backed up behind the ice-dam until the dam became unstable and failed, releasing a vast flood of water (Trimble, 1963). These floods are known as the Pleistocene floods or "Bretz Floods". These floods scoured and redeposited sands and gravels in the Pleistocene river channel. At the site, the Pleistocene channel at the time of the floods was probably greater than 350 feet below the present day ground surface. Consequently, the Pleistocene flood deposits are not exposed at the surface in the lower Columbia River valley but are probably present at depths below 300 feet.

At the end of each glacial period, including the latest, sea level rose rapidly as the glacial ice melted. This rise in sea level caused a general flooding and formation of an estuary environment in the lower Columbia River. The base level of the Columbia River rose concurrently, resulting in rapid sedimentation of alluvium along the river. This alluvium consists of sand deposit along the river channel and silt, clay, and organic soils in the overbank (flood plain) deposit.

The geologic structure within the vicinity of the project area is complex. Overall, the area is dominated by the broad north-south upwarp of the Coast Range. The amount of upwarping is uneven, with both the Tillamook highlands to the south and Willapa Hills to the north, uplifted higher than the area in between along the lower Columbia River. Geologic mapping shows the older rocks exposed in the core of the uplifted areas are extensively faulted (Walker and MacLeod, 1991 and Walsh and others, 1987). Faults are generally oriented northwest-southeast and northeast-southwest. Most of these faults, however, appear to be restricted to the older rocks suggesting that they are related to the older tectonism and were not active after the deposition of the younger sedimentary rocks. Therefore, they are not active now.

Superimposed on the broad uplift are numerous small secondary folds. In the vicinity of the project, these secondary folds are oriented northwest-southeast (Walker and MacLeod, 1991 and Walsh and others, 1987). The nearest mapped secondary fold is a syncline that trends through Quincy, beneath the project site and into the state of Washington.

4.2 Seismic Setting

The site is located in the seismic region known as the Cascadia Subduction Zone (CSZ), which extends from Northern California to British Columbia. A more in depth discussion of the CSZ is presented in Appendix D. In the CSZ, just off the coast of Oregon and Washington, the oceanic Juan de Fuca Plate is being forced under the North American Plate. Much of the Pacific Northwest's topographic relief, including the Coast Ranges and Cascade Mountains and the region's seismicity, can be attributed to the plate tectonics of the region. Three types of earthquakes are known to occur within the CSZ: shallow crustal, deeper subcrustal intraplate, and the large interface. The most seismically active area occurs in the Puget Sound region, 60 miles to north.

Earthquakes are sized using two fundamentally different scales: Modified Mercalli scale and magnitude scales. The following definitions are based on Rogers, Walsh, Kockelman, and Priest (1996) definitions. The Modified Mercalli scale was developed before the advent of mechanical means of measuring earthquakes. It is a subjective numerical index describing the severity of an earthquake in terms of its effects on the Earth's surface and on humans and their structures. The index scale spans from Roman Number I, felt by few, to XII, total destruction. Unless specifically stated, Modified Mercalli intensity is the maximum observed at the epicenter of an earthquake.

Magnitude scale is a measured number that characterizes the relative size of an earthquake. It is based on measurement of the maximum motion recorded by a seismograph corrected for attenuation to a standardized distance. Several magnitude scales have been defined, but the most commonly used are 1) local magnitude (M_L), commonly referred to as "Richter magnitude," 2) surface-wave magnitude (MS), 3) body-wave magnitude (m_b), and movement magnitude (M_W). The first three scales have limited range and applicability and do not satisfactorily measure the largest earthquakes. The moment magnitude (M_W) scale is based on the concept of seismic moment, and is uniformly applicable to all sizes of earthquakes. Conceptually, all magnitude scales can be cross-calibrated to yield the same value for any given earthquake. In practice, however, this has only been proved to be approximately true. For engineering purposes, the scales are similar enough that the differences are not significant. Historically, most of the earthquakes recorded in the Pacific Northwest were reported in local magnitude M_L scale. For this report, magnitudes are expressed as **M** without attempting to convert between the various scales.

Shallow crustal earthquakes take place typically between depths of 10 km and 20 km. Several earthquakes between estimated **M**4 and **M**5 have occurred within 31 miles (50 km) of the site over the past 150 years. The most significant event is the estimated **M**5.2, 1962 Portland-Vancouver earthquake located approximately 46 miles east-southeast of the site. Earthquake recurrence relationship suggests a magnitude **M**6.0 event with about a 500-year recurrence and a magnitude **M**6.5 event with about a 5000-year recurrence.

The second major type of earthquake that could affect the site is a deeper subcrustal intraplate earthquake occurring within the subducting Juan de Fuca Plate at depths between 40 km to 60 km. The 1949 Olympia and the 2001 Nisqually earthquakes were deep subcrustal events. An intraplate earthquake could potentially occur directly below the site (depth 50 km). The maximum expected magnitude for an intraplate earthquake is between **M**7.0 and **M**7.5. An earthquake recurrence relationship extrapolated to large magnitudes based on smaller magnitude subcrustal earthquakes suggests that an **M**7.0 event may occur in the region once in 1000 years. The distance that this possible event could have ranges between 0 to more than 30 miles (0 to 50 km). For hazard analysis purposes, a **M**7.0 occurring directly beneath the site (distance 0 km, depth 50 km) and a larger **M**7.5 event occurring at a distance of 30 miles (50 km) were considered.

The third major type of earthquake that potentially could affect the site is an interface, or subduction zone, earthquake, which could take place at the boundary of the Juan de Fuca and the North American plates. Although a subduction zone earthquake has not been historically recorded off the coast of Oregon or Washington, geologic data suggests that a **M9+** earthquake is possible from an interface event. The best estimate for the most likely size ranges between magnitudes **M**8 to **M**9 depending upon the length that ruptures. Recurrence for a subduction zone interface earthquake ranges from 350 to 600 years, with a mean recurrence of about 450 years. The last event occurred 300 years ago. The nearest approach of a CSZ interface earthquake would be about 30 miles (50 km) west of the site.

A literature review was also conducted to identify known geologically active or potentially active faults within 62 miles (100 km) of the site. The results are presented in Appendix D. Primary reference sources reviewed include Seismic Design Mapping: State of Oregon (Geomatrix Consultants, 1995), National Seismic Hazard Maps (Frankel, et al., 1996) and Wong and others (2000). The review shows that there are at least eleven geologic faults or fault zones with or suspected with greater than 50 percent probability of having Quaternary movement (movement within the last two million years). In addition, the CSZ is active and underlies the site at depth.

4.3 Geologic Hazards

Potential geologic hazards for the site were evaluated. The results are presented in Geologic and Soil Stability Assessment, Appendix D. Based on the geologic history, the alluvial soil is assumed to extend down to about 350 feet below sea level. Deep alluvial soils at the site strongly affect seismic ground response at the surface. The assessment identified the primary geologic and soil stability issues are associated with seismic hazards: primarily strong ground shaking, the potential for liquefaction of some of the subsurface materials, and seismically induced settlement. The analysis indicates that seismic waves would be significantly dampened and deamplified as they traverse up through the deep soil column. In addition, the analysis suggests that some of the loose sandy silt and sand strata may be susceptible to liquefaction during a subduction zone earthquake event. The occurrence of liquefaction could result in loss of foundation bearing capacity of the near surface soils and/or settlement. Consequently, heavy structures and structures sensitive to settlement probably will be founded on deep piles driven to below identified liquefiable zones to provide adequate support.

Other geologic hazards, in our current opinion, are not significant at the site. The site is flat and there are no landslide or slope stability issues. Also, there is little risk of fault displacement at the site. In addition, the site is located behind flood control levees that provide 100-year flood protection with 4.7 feet of freeboard. Since the site is level and over 2000 feet from Bradbury

Slough, the potential for lateral spreading is not considered a hazard. Also, the site is too far from the ocean to be affected by tsunami.

5.0 FIELD EXPLORATIONS AND LABORATORY TESTING

The subsurface conditions beneath the site were investigated with eight borings that were advanced between April 16 and April 25, 2001. Laboratory and field soil tests consisting, of among others, photoionization, soil classification, seismic compression and shear wave, and soil resistivity tests were performed. Presented in the following sections is a discussion of tests performed at the site during the field exploration and laboratory testing that were performed on the samples returned to our office.

5.1 Field Explorations

The locations of the borings, designated B-1 through B-8, are shown on the Borehole Location Map, Figure 2. The borings were advanced to between 80 and 150 feet from the ground surface using a combination of track and truck-mounted drill rigs owned and operated by Geo-Tech Explorations of Tualatin, Oregon. A total of 852 feet lineal feet was drilled, sampled, and logged.

During the drilling, disturbed samples were obtained at about every 2.5 feet in the upper 25 feet, and about 5 feet thereafter using the Standard Penetration Test (SPT) ASTM D1586. During the Standard Penetration Test, the N-value blow counts required to advance the sampler with a 140-pound weight dropped 30 inches was recorded. The N-value, expressed as blows per foot, is used to provide a measure of the relative density of granular soils such as sand, and the consistency of cohesive soils such as silt and clay. In addition, thin-wall Shelby tube samples of relatively undisturbed soil were obtained at selected depths.

Two piezometers, consisting of a slotted PVC pipe backfilled with clean free draining sand were installed in Borings B-4 and B-7 at the site to allow for future measurements of a ground water level. At the ground surface, each piezometer pipe was placed inside a flush mounted monument cover set in concrete. All the other borings were backfilled with bentonite up to the ground surface at the completion of drilling, except for B-3 that also contained the downhole testing PVC pipe, described below.

Presented in Appendix A is a description of the procedures used in making the borings, including the details of the piezometer installations and the techniques utilized in obtaining the various types of soil samples. Table A1 in Appendix A presents the terminology used to describe the soils. Presented on Figure A1 of Appendix A is information related to the symbols, soil and well material graphics, and soil property data presented on the boring logs. The logs of the borings are presented in figures A2 through A9.

5.2 Photoionization Testing

Environmental screening for the presence of volatile vapors in the upper 15 feet of each boring was analyzed by use of a Photoionization detector (PID). The PID measures vapors released from chemical volatilization of organic compounds in parts per million (ppm). For the purpose of environmental screening, a lower limit threshold was set to 10 ppm for this project based on typical industry standards, before further environmental analysis was considered necessary. Additional information on this testing is contained in Appendix A.

5.3 Laboratory Testing

Laboratory tests were performed on selected soils returned to our laboratory to evaluate the soil index properties and provide data related to the strength and settlement characteristics of the soil. The testing program adopted for this investigation includes soil visual examinations, moisture content, grain-size analyses, Atterberg limits, and unit weight measurements. In addition, two unconfined compressive strength and a soil consolidation test were also performed. Presented in Appendix B of this report is a description of the laboratory tests that were performed and the testing results.

5.4 Downhole Seismic Tests

A downhole seismic wave velocity survey for S and P waves was conducted at the project site in Boring B-3 on April 22, 2001. The test was performed by Northwest Geophysical Associates, Corvallis, Oregon, and the results are presented in Appendix C. In general, the test measures the time required for shear (S) and compression (P) waves propagation through soils over a range of distances from a surface energy source. By measuring the arrival time of shear waves at incremental depths in the borehole, a profile of shear wave velocity is developed. Changes in shear wave velocity with depth in the borehole were used to predict differences in soil types, soil properties and soil behavior. Shear wave velocity in the soils was used in the seismic analyses of the site and an evaluation of the range of the level of ground shaking during the controlling earthquake event.

5.5 Soil Resistivity

Soil resistivity measurements were made at the site on May 3, 2001 to determine the soil resistance to an electric current. We understand this information will be used to evaluate the grounding potential of the soils at the site. The resistivity of the soil was measured using the four-point Wenner method with tests performed by Northwest Geophysical Associates. The results of the test are presented in Appendix E.

6.0 DISCUSSION OF SUBSURFACE CONDITIONS

6.1 Soils

Figure 3 through Figure 5 present general geologic cross sections, which show in a generalized manner, the interpreted subsurface conditions disclosed by the borings at various locations at the site. The Cross Sections are designated A-A', B-B', and C-C' and their location and orientation are shown on the Site Plan, Figure 2. The geologic Cross Sections are interpretive in nature and the contacts between soil units may be gradational. Further, variations in soil conditions may exist between the locations of the borings.

As shown on the geologic Cross Sections, the subsurface materials encountered at the site can be divided into two general soil units within the depth of our explorations, based on their engineering characteristics and stratigraphic position. The subsections that follow present a description of the two soil units, including the subsurface conditions and materials present across the site. A more detailed description of the soils is described on the Boring Logs, Figures A2 through A9 (Appendix A).

6.1.1 Upper Fine-Grained Alluvium

An upper fine-grained alluvium unit was encountered in all the borings and consists generally of very soft silt with various minor amounts of fine sand. The upper alluvium was encountered up to depths between 25 to 60 feet from the ground surface. Blow counts or N-values, observed during the Standard Penetration Test (SPT) varied from 0 to 11 blows per foot. In general, the predominantly silt soils, which constituted a majority of the unit, had N-values between 0 and 2. Higher N-values between 5 and 11 were observed in the silt soils containing, in general, a higher percentage of sand. Organics, including isolated pieces of plant and wood fiber, were generally observed in estimated amounts between 5 to about 15 percent (based on volume) of the soil samples. The moisture content of the unit ranged between 40 to 70 percent. Some higher moisture contents were observed within the soils containing a larger percentage of organic matter.

The plasticity characteristics of the soil unit, as measured in Atterberg Limits Tests, indicate a Liquid Limit (LL) between 53 and 73 percent, and a Plastic Limit (PL) between 35 and 41 percent. These values are influenced, in our opinion, because of the presence of organic matter, as described previously. The Plasticity Index (PI) ranged between 0 percent (non-plastic) to 34 percent, with a majority of the test results below 15 percent. Locally within the unit, some minor amounts of clay were apparent, up to estimates of about 5 percent, by weight of the sample. Classification tests performed on the silt, including dry strength, dilatancy and toughness, performed in general accordance with ASTM D-2488, indicate a range of plasticity between non-plastic to medium plasticity, with a majority of the results ranging from non-plastic to low plasticity.

In general, as indicated by a majority of the "N"-values between 0 and 2, the silty soil was classified as either "very loose" or as "soft", depending upon its apparent plasticity. The condition of the silt, together with a high ground water level at the site, and the presence of organic matter, in our opinion, contributes to a moderate to high potential of settlement within the unit. A consolidation test was performed on a sample of the upper fine-grained alluvium with results discussed under Section 7.3.

Measurements of shear strength were performed on selected samples of the soil unit and consisted of unconfined compressive strength test, pocket penetrometer, and torvane strength tests. The results of the unconfined tests indicate undrained shear strength of between .18 and .25 ton per square foot (tsf), correlating to very soft. Pocket penetrometer tests and torvane tests performed on Shelby tube samples returned to our laboratory indicate a range of undrained shear strength between 0 and .25 tsf.

6.1.2 Lower Sandy Alluvium

Below the upper fine-grained alluvium, we encountered a lower sandy alluvium unit consisting mostly of fine-grained poorly graded sand with varying amounts of silt. All of the borings were terminated in this soil unit. N-values varied between 4 to 60 blows per foot, with most of the values between 20 to 35 blows per foot. The lower N-values within this unit were generally observed in the sand soils that contained a higher percentage of silt. The moisture content of the unit ranged between about 30 to 50 percent. Organics, although observed in this unit, were generally less abundant than observed in the upper fine-grained alluvium.

6.2 Ground Water

Ground water was measured at depths between 2 to 4 feet from the ground surface in Borings B-3, B-4 and B-5 during and immediately after drilling. A ground water level was not observed in the other borings and is in general, difficult to measure when a mud-rotary system is used. Based on our analyses and our experience, we believe that the ground water level at the site should be expected at elevations closely related to the surface water level in the Columbia River, located to the north of the site.

6.3 Photoionization Results

Photoionization results on soil samples in the upper 15 feet of each boring ranged from 0 to 8 ppm. Boring B-3 at 10 feet registered 8 ppm, while all other results in the other seven borings registered no more than 0.1 ppm. Since all results were below the minimum threshold, 10 ppm, previously described, no samples required additional analytical analysis.

7.0 PRELIMINARY CONCLUSIONS AND RECOMMENDATIONS

7.1 General Findings

The field explorations disclosed that deep soft alluvial sediments exist across the site. The conditions observed in the borings suggest that the upper 50 feet of soils is relatively loose to very soft, and potentially liquefiable during the design earthquake. In addition, ground water occurs at a relatively shallow depth. During periods of flooding, water level in the river is higher than the ground surface. High ground water is currently controlled by a drainage ditch system managed by the Beaver Drainage District. In our opinion, the upper relatively soft soils in their existing condition are not suitable for the support of settlement sensitive equipment, heavily loaded mat foundations, and building foundations. Pile supporting structures or ground modification techniques will be discussed in later sections.

7.2 Site Preparation/Earthwork/Ground Water Control

The following issues are considerations for future design and construction activities.

7.2.1 Clearing and Stripping

There are scattered trees that will need to be cleared and grubbed. The pasture land vegetation cover and topsoil should be stripped under settlement sensitive facilities and other areas where organics left in-place would be a detriment to long-term performance.

7.2.2 Well Abandonment

Regarding subsurface features, we became aware of an existing shallow water well that would need to be abandoned by a State of Oregon licensed water well driller. Similarly, the two soil borings containing the standpipe piezometers and the one boring containing the grouted pipe for the downhole seismic tests will need to abandoned according to Oregon Department of Water Resources regulations.

7.2.3 Working Pad (Site Fill)

Due to the relatively very loose and soft nature of the shallow subsurface materials and the high ground water levels, working pads or mats are advisable for the construction period. Typically, a pad constructed of imported granular material, preferably well-graded, free-draining crushed rock placed on a heavy non-woven geotextile would be used. The material specifications, thickness, and placement methods would depend on how the working pad would be incorporated into the design of the various foundation systems, roadway subgrade preparation, and buried piping. Based on discussions with the site grading consultant, we understand that site filling throughout most of the area will be less than 3 feet. The exception would be areas requiring special treatment. Since site filling would cause some settlement, we have assumed a site fill thickness of 3 feet in our analysis discussed in Section 7.4.

7.2.4 Drainage Ditches

There are at least two fairly deep drainage ditches that intersect the footprint of the plant facilities that will need to be dealt with during site preparation. We understand that these ditches are part of the Beaver Drainage District.

7.2.5 Softer Surface Areas

In the southwest portion of the site in the vicinity of the existing barn, we noticed that the ground surface was generally softer than the rest of the plant site area. Additional stripping or other treatment may be required if facilities area placed in this area.

7.2.6 Ground and Subsurface Modification

To decrease the long-term settlement of the deep, soft and loose subsurface materials for static and seismic loading conditions, various ground improvement methods may be needed as part of the overall site preparation. More discussion related to this is mentioned in sections below.

7.2.7 Earthwork and Ground Water Control

For the various earthwork activities, heavy earthwork equipment and loaded dump trucks most likely will have difficulty operating on the existing ground surface. During our explorations, truck-mounted soil exploration drill rigs were breaking through the vegetative cover and were stuck several times. For the excavations that extend below the shallow ground water, we anticipate that lowering ground water levels with positive control dewatering systems would be needed. Use of sump systems is generally not feasible for these types of soil. The use of excavated material from above and below the ground water levels for structural fill or backfill most likely is not feasible. Potential uses of the excavation spoils may be for landscaping or grading for surface drainage improvements. Grading this material with its high moisture content will be difficult.

7.2.8 Other Related Issues

There other site preparation issues adjacent to the site, such as construction of an access roadway embankment to change grade from the existing road on the levee adjacent to the slough, crossing of the raised grade railroad tracks, and preparation of subsurface for utilities coming into or leaving the site.

7.3 Soil Parameters for the Site

Soil parameters are provided for the project site to assist in the preliminary project site evaluation. Based on the subsurface conditions and the laboratory testing, the recommended soil parameters are presented in Table 1, below. Descriptions of the various parameters follow Table 1.

Table 1
Soil Parameters for the Site

Soil parameter	Very loose silt to sand	Very soft silt	Lower Sandy Alluvium
Poisson's ratio	0.2	0.3	0.25
Modulus of elasticity	100,000 psf	10,000 psf	250,000 psf
Shear modulus	300,000 psf	340,000 psf	900,000 psf
Subgrade modulus	30 pci	25 pci	100 pci
Moist unit weight	105 pcf	100 pcf	120 pcf

psf = pounds per square foot
pcf = pounds per cubic foot
pci = pounds per square inch per inch

7.3.1 Poisson's Ratio

Poisson's ratio, μ, is defined as the ratio of axial compression to lateral expansion strains. Poisson's ratio is both nonlinear and stress-dependent. The range of Poisson's ratio is relatively small for the same types of soil at the site; therefore, we estimated Poisson's ratio based on the soil classifications. The estimated Poisson's ratio values are presented on Table 1. The Poisson's ratio for the very soft silt is estimated for drained condition.

7.3.2 Modulus of Elasticity

The modulus of elasticity, E_0, is the initial slope of soil stress-strain curve. It is often estimated by correlation from field tests, such as the Standard Penetration Test (SPT) and Cone Penetration Test (CPT). For this project, we used the field SPT N-values and laboratory test results to estimate the Modulus of Elasticity for both the very loose silt to sand and very soft silt. The modulus of elasticity of the very soft silt is estimated for drained condition. The estimated modulus of elasticity values are shown in Table 1. Estimates of E_o were based on information from EPRI, 1990.

7.3.3 Shear Modulus

The shear modulus, G, is defined as the slope of the shear stress-strain curve. For soil seismic evaluation purposes, the shear modulus is often estimated by using shear wave velocity measurements, v_s. The relationship between shear modulus and shear wave velocity is: $G = \rho\, v_s^2$, where ρ is the mass density of the soil. The shear modulus estimated using the above method is a low-strain shear modulus. The shear modulus for the project site were estimated by using the measured shear wave velocity data obtained using a downhole technique in Boring B-3.

Appendix C provides additional background data related to the downhole shear wave velocity values. The estimated shear modulus values are shown in Table 1.

7.3.4 Subgrade Modulus

The subgrade modulus, k_{s1}, is defined as the ratio of stress to deformation for a 1-foot by 1-foot square plate or 1-foot wide beam resting on the subgrade. The subgrade modulus is generally dependent on the relative density of the native soil and the thickness of the compacted foundation structural fill above the native material. The estimated subgrade modulus for the native soils is shown in Table 1. The estimated subgrade modulus values in Table 1 are based on an assumption that footings directly are founded on the native soils. Therefore, in the final design phase, the subgrade modulus should be modified based on the thickness of the compacted working pad and foundation structural fill above the native soils.

7.3.5 Consolidation Settlement Parameters

A one-dimensional consolidation test was performed on a sample of the upper fine-grained alluvium layer, specifically from boring B-6, at a depth of 15 feet. The test sample was classified as soft silt (ML) with trace fine sand and scattered organics. An Atterberg Limits Test resulted LL = 53.6%, PL = 40.8 %, and PI = 13.9%.

The percent strain in the sample was plotted versus the applied test load. Since the interpreted apparent pre-consolidation pressure was slightly above the present overburden pressure, the sample was judged to be essentially normally consolidated. From the strained based consolidation test, soil was judged to normally consolidated based on a reconstructed curve to adjust for potential sample disturbance. The following parameters were estimated based on the results of the consolidation test and our experience:

$C_{c\varepsilon}$	=	0.12
$C_{r\varepsilon}$	=	0.0008
$C_{\alpha\varepsilon}$	=	0.002
Pre-consolidation pressure	=	1,700 psf
OCR	=	slightly over 1

where $C_{c\varepsilon} = \dfrac{C_c}{1 + e_o}$.

For definition of terms, we recommend referring to Holtz and Kovacs, 1981. In our experience with silty soil with organics along the Columbia River, we have seen $C_{C\varepsilon}$ values range from approximately 0.10 to 0.20, depending on the soil consistency and amount of organics.

7.3.6 Coefficient of Sliding Resistance

The lateral loads on the various power facilities, including lateral earth pressures, earthquakes, and wind can be resisted by sliding resistance of the foundation and partial soil passive pressure, which should be estimated in the final design. The coefficient of sliding resistance for concrete on granular materials generally ranges between 0.3 to 0.4. For this site, it is not feasible to place concrete foundations directly on the native soil.

7.3.7 CBR and Resilient Modulus

The native soil subgrade at the plant site is predominately very low strength non-plastic silt to sand with relatively high natural moisture content. For design of flexible pavement sections, we estimate a California Bearing Ratio (CBR) of 1 percent. Also, for use in design of flexible pavement sections, we estimate a resilient modulus (M_R) value of 1,500 psi. The CBR value was estimated by past experience on these types of soils, and use of the soil classification tests performed on the near surface soils. The M_R value was estimated by the commonly used expression (1500 x CBR) presented in AASHTO Guide for Design of Pavement Structures (1993).

7.3.8 Hydraulic Conductivity of Native Soil

Hydraulic conductivity tests have not been conducted on the native soils. However, based on visual soil classification, experience in similar soils along the Columbia River, and comparison to the consolidation test time rates, hydraulic conductivity is expected to be low. The upper silt and silty fine sand is estimated to have a hydraulic conductivity of about 10^{-5} to 10^{-3} cm/sec. The hydraulic conductivity of the underlying very soft silt is estimated to be in the range of 10^{-6} to 10^{-4} cm/sec.

7.3.9 Seismic Soil Profile Type

The seismic soil profile type represents the average condition of the upper 100 feet beneath the site. The Uniform Building Code, 1997 Edition (UBC-97) Soil Profile Type for the site is S_F because the soil is vulnerable to potential failure due to liquefaction occurring in the medium dense silty sand. The designation S_F means that a site-specific evaluation must be conducted.

From our site evaluation, the site is underlain by about 50 feet of loose sandy silt and medium dense silty sand that is susceptible to liquefaction and 20 to 30 feet of very soft silt (PI <20).

7.3.10 Site Response

Site response spectra for the site is presented in Appendix D. The site is classified as a seismically soft site with potential for soil liquefaction to occur above elevation -50 feet. The foundation support system should consider this risk.

7.4 Foundation Alternative Evaluation

To compare foundation support alternatives for the non-heavily loaded structures planned for the site, we have completed a preliminary evaluation of two different support alternatives using two site soil models. These consist of 1) shallow mat foundations, and 2) pile-supported deep foundations. The two different soil models and types of planned structures are:

- Main Plant Area – Typical water tanks planned for construction in the north central portion of the site.
- Cooling Tower Area – A series of multi-cell cooling towers planned near the southeast corner of the site.

Presented below is an estimate of static settlement and seismically induced post-liquefaction settlement for the shallow foundation system. With large amounts of settlements anticipated for these structures, piles for most of the structures may be warranted. A discussion of estimated pile capacities is presented in a later section. Also discussed are possible mitigation measures to reduce settlement.

We have assumed the heavily loaded structures such as turbines, generators, HRSGs, and other settlement sensitive structures would be placed on pile-supported foundations.

7.4.1 Shallow Foundations Main Plant Area

To analyze a typical shallow foundation support alternative, we have assumed a mat foundation with a plan area of 40 feet by 40 feet and a static dead and sustained live load of 500, 1000, 2000, and 3000 psf. A preliminary soil analytical model was developed for this area based on the interpreted subsurface soil conditions, and the results of laboratory tests. A detail of the soil model for the main plant area is presented in Figure 6. For these settlement estimates, the lower sandy alluvium is considered non-compressible.

For static dead load and sustained live loads, estimates of total settlement, including estimates of secondary settlement, are:

For 500 psf:	1 to 2 inches
For 1,000 psf:	3 to 6 inches
For 2,000 psf:	6 to 10 inches
For 3,000 psf:	10 to 15 inches

Settlement at the site may also occur due to earthquake induced post-liquefaction settlement. The extent and level of liquefaction in general, will depend on the severity of ground shaking at the site. Figure 6 shows approximated soil zones that would liquefy during the design level magnitude earthquake that was selected based on the site-specific earthquake and hazard analyses described in Appendix D. We estimate that between 10 and 15 inches of post-liquefaction induced settlement may occur.

Based on these estimates of static and seismic induced settlement, settlement mitigation will be necessary to prevent damage to the structures. For mitigation of static and seismically induced settlement, we suggest supporting the structures on piles. Preloading could mitigate excessive static settlement; however, in our opinion, typical schedule constraints for fast-track power plant projects cannot accommodate the time necessary for conventional preloading approaches. Based on our analysis and experience, we estimate that a preload fill without installing vertical drains in the subsurface should remain in place a minimum of 3 to 4 months to induce the consolidation settlement. Installing vertical wick drains could substantially speed up the time for settlement to occur. Since preloads generally cannot mitigate for seismically induced liquefaction settlement, ground modification construction techniques should be evaluated to densify the sandy liquefiable materials.

7.4.2 Shallow Foundations Cooling Tower Area

To analyze the shallow foundation support alternative for the cooling tower area, we have assumed a mat foundation with a plan area of 40 feet by 450 feet and a static dead load and sustained live load of 500, 1,000, 2,000, and 3,000 psf. A soil analytical model was developed for this area based on the interpreted subsurface soil conditions and the results of laboratory tests. A detail of the soil model for the cooling tower area is presented in Figure 7. For these settlement estimates, lower sandy alluvium is considered non-compressible.

For static dead load and sustained live loads, estimates of total settlement, including estimates of secondary settlement, are:

For 500 psf:	4 to 6 inches
For 1,000 psf:	8 to 12 inches
For 2,000 psf:	12 to 18 inches
For 3,000 psf:	18 to 24 inches

Figure 7 shows our estimate of the soil zones that would liquefy under the same seismic event described in Appendix D. We estimate that between 12 and 18 inches of soil liquefaction induced settlement may occur.

Settlement mitigation will again be necessary to prevent structural damage to the structures. The settlement mitigation measures described above also apply to this area.

7.4.3 Deep Foundations for the Site

As previously discussed, the preliminary analytical soil models presented on Figures 6 and 7 show a layer of very soft compressive silt, and layers of very loose to medium dense liquefiable sandy silt to sand up to a depth of 60 feet below the existing ground surface. Since this surface condition results in very large estimated settlements, pile-supported foundations should be considered for all the settlement sensitive plant facilities or the seismically designed facilities. We recommend that the minimum pile embedment be 80 feet which includes at least 20 feet below the bottom of the potentially liquefiable layers to account for variability of subsurface conditions at the site. We recommend additional subsurface explorations including use of the Cone Penetration Test (CPT) to better define the thickness of the compressible soil layers.

For preliminary evaluation, we analyzed piles consisting of 12¾-inch and 16-inch diameter driven closed-end, steel pipe piles. Pipe piles should conform to the requirements of ASTM A252, Specifications for Welded and Seamless Steel Pipe Piles. We assumed the pipe piles would be fitted with a welded flat plate.

The allowable compressive and uplift capacities of the driven closed-end, steel pipe piles were evaluated under both static and seismic conditions with capacity estimates in Table 2. For the static compression condition, a nominal soil shaft friction was used for the upper 60-foot compressible zone. The allowable compressive values have a factor of safety equal to or slightly greater than 3. For the seismic compression condition, the upper 60-foot compressible zone was assumed to provide no soil shaft friction resistance and apply no downdrag or negative skin friction to the pile. The allowable seismic compressive values have a factor of safety equal to or slightly above 2. For the allowable static uplift capacities shown in Table 3, the 60-foot compressible zone was treated in the same manner as for compression. The factor of safety for the static allowable uplift condition is equal to or greater than 3. The factor of safety for the seismic allowable condition is equal to or greater than 1.

Table 2
Allowable Compressive Pile Capacities

Pile Depth (ft)	12¾-inch Dia. (kips)		16-inch Diameter (kips)	
	Static	Seismic	Static	Seismic
70	80	65	120	100
80	100	85	150	130
90	125	110	190	170

Table 3
Allowable Uplift Pile Capacities

Pile Depth (ft)	12¾-inch Dia. (kips)		16-inch Diameter (kips)	
	Static	Seismic	Static	Seismic
70	50	30	75	55
80	65	45	95	75
90	85	65	120	100

The above compressive and uplift capacities with the pile embedment lengths shown should result in less than ½-inch settlement. The allowable capacities assume no reduction for group effects and that all piles are driven no closer than 3 pile diameters center-to-center. Also, to maintain spacing, we assume piles would be driven with a maximum deviation from vertical of not more than 3 percent (1.5 inches in 4 feet).

The proposed structures will be subject to lateral loads due to wind and earthquake forces. The lateral load capacities of these pipe piles were evaluated for both static and seismic loading conditions. The laterally loaded pipe pile analyses were performed with the aid of the computer program "LPILE". Two pile sections, PP12¾ X 0.375 and PP16 X 0.375, under a free-pile head condition were evaluated. For these values a reduction for group action was not considered and no lateral resistance was assumed form passive resistance from an embedded pile cap. Based upon our evaluation, the single pipe piles, PP12¾ X 0.375 and PP16 X 0.375, can provide 4 kips and 6 kips, allowable lateral capacities, respectively, under static loading condition and horizontal deflection of approximately ½-inch. Included is a factor of safety equal to about 2.0. Under seismic loading conditions, the allowable lateral capacities of the piles should be reduced to about 50 percent of the static condition. The results of the computer analyses showed an approximate depth to fixity below the top of the pile as follows:

PP12¾ X 0.375	25 feet
PP16 X 0.375	30 feet

7.4.4 Settlement Sensitive Pipes, Pipe Racks, and Conduits

We estimate that differential static settlement between pipe racks, utility conduits and pipelines (i.e., linear facilities) may occur between structures with different foundation support systems. In addition, seismic induced liquefaction settlement could have a significant impact on settlement sensitive linear facilities. If these facilities cannot tolerate the settlement magnitudes estimated, we suggest deep foundation be considered. If linear facilities are allowed to settle, we recommend evaluating special pipe joints and connections, sleeves, shorter pipe lengths, and other methods to help mitigate such settlement and possible infrastructure damage. Also, we recommend that settlement analyses based on the type, depth, and difference in settlement tolerance between the planned structures be completed to evaluate the impact on these type of structures.

7.4.5 Lateral Earth Pressure

Lateral earth pressure on retaining walls depend on the type of wall (i.e., yielding or non-yielding), the type and method of placement of backfill against the wall, the magnitude of surcharge during construction or permanent loads on the ground surface adjacent to the wall, the slope of the backfill, location of the ground water level, use of positive drainage systems behind wall, and the design criteria such as static or seismic condition, and combination loading conditions. Based on the nature of the native soil at the site, it is our opinion that the native soil should not be used for backfill, and backfill material should be imported. For retaining wall backfill, import material consisting of free-draining, crushed rock would be the most desirable.

7.4.6 Roadways

Construction staging areas, roadways, and parking areas constructed on these loose and soft subsurface materials will require special consideration for subgrade stabilization. The subgrade bearing values for the native materials are estimated to be extremely low; therefore the use of geotextile, geogrids, and free-draining imported crushed rock should be considered to develop an adequate zone of subbase strength. Also, the consideration of maintaining drained subbase base material should also be considered.

8.0 REFERENCES

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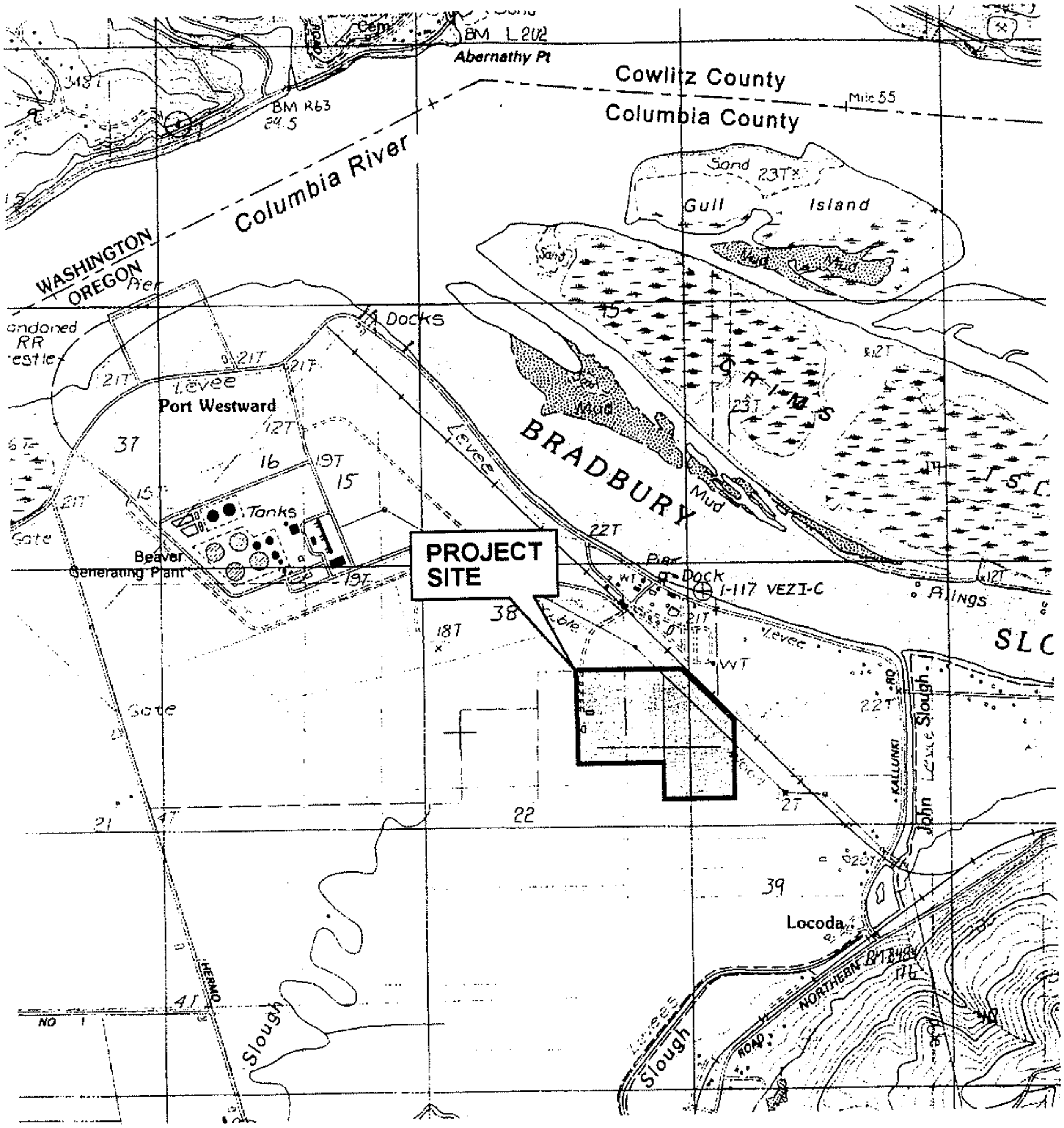
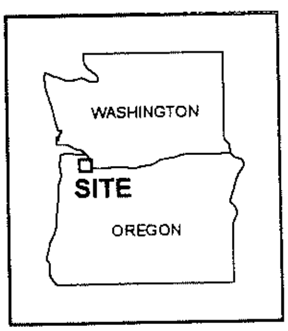
Note: A more comprehensive list of references for the Site Specific Geological and Soil Stability Assessment is contained in Appendix D.





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SCALE IN FEET

0 2000 4000

Source:
USGS 7.5 minute quadrangle map.
"OAK POINT, WASH,-OREG."

SUMMIT/WESTWARD ENERGY PROJECT
CLATSKANIE, OREGON

VICINITY MAP

SQUIER ASSOCIATES

FIGURE 1

NOTE: Features shown are for illustrative purposes only and are approximate.

Drawn by: GCD	Checked by:	Graphics Check: AHR	File: GRAPHICS\FIG1.FH8	Date: 4/24/01	Job. No. 201578

TABLE 2

DC Resistivity Models
Summit/Westward Energy Project
Clatskanie, Oregon

Models	Layer 1		Layer 2		Layer 3		Layer 4	Model
Sounding	Resistivity (ohm-m)	Depth (Feet)	Resistivity (ohm-m)	Depth (Feet)	Resistivity (ohm-m)	Depth (Feet)	Resistivity (ohm-m)	Misfit % Error
R-1	204	1.5	105	15.4	18	40	52	3.1%
R-2	161	4.7	67	18.1	19	46	57	5.2%
R-3	122	2.9	98	14.3	18	39	57	1.9%
R-4	102	3.2	57	14.5	17	36	51	2.3%
R-5	148	1.5	87	7.3	35	32	49	0.7%
R-6	213	3.4	72	15.8	21	40	54	1.9%

TABLE 1

a-spacing (feet)	Apparent Resistivity (Ohm-m)	Apparent Resistivity (Ohm-ft)	V/I (Ohms)	Error	Current (mA)
Sounding R-5					
Boring B-6	E-W Sounding offset 20 feet north of B-6(new)				
3.0	111.3	319.9	1.70E+01	0.1%	100
4.0	99.2	285.1	1.13E+01	0.0%	100
5.0	91.0	261.5	8.32E+00	0.0%	100
7.0	79.8	229.2	5.21E+00	0.0%	100
10.0	67.2	193.2	3.08E+00	0.0%	100
15.0	52.6	151.0	1.60E+00	0.0%	100
20.0	45.0	129.2	1.03E+00	0.0%	100
25.0	42.8	123.0	7.83E-01	0.0%	100
30.0	42.0	120.8	6.41E-01	0.1%	100
40.0	41.4	119.0	4.74E-01	0.0%	100
50.0	42.3	121.5	3.87E-01	0.1%	100
70.0	43.3	124.4	2.83E-01	0.0%	100
100.0	45.3	130.1	2.07E-01	0.0%	100
130.0	46.6	134.0	1.64E-01	0.0%	100
160.0	47.4	136.3	1.36E-01	0.0%	100
Sounding R-6					
Boring B-4	E-W Sounding offset 10 feet north of B-4				
3.0	181.2	520.8	2.76E+01	0.0%	100
4.0	170.2	489.0	1.95E+01	0.0%	100
5.0	147.0	422.5	1.34E+01	0.0%	100
7.0	115.8	332.9	7.57E+00	0.1%	100
10.0	87.4	251.1	4.00E+00	0.0%	100
15.0	66.4	190.8	2.02E+00	0.0%	100
20.0	52.4	150.5	1.20E+00	0.0%	100
25.0	46.1	132.5	8.43E-01	0.1%	100
30.0	40.5	116.3	6.17E-01	0.0%	100
40.0	35.1	100.7	4.01E-01	0.1%	100
50.0	34.5	99.1	3.15E-01	0.1%	100
70.0	36.4	104.6	2.38E-01	0.0%	100
100.0	41.6	119.6	1.90E-01	0.0%	100
130.0	44.6	128.0	1.57E-01	0.0%	100
160.0	47.7	137.2	1.36E-01	0.6%	100
200.0	46.2	132.8	1.06E-01	0.0%	100

END

a-spacing (feet)	Apparent Resistivity (Ohm-m)	Apparent Resistivity (Ohm-ft)	V/I (Ohms)	Error	Current (mA)
Sounding R-3					
B-7 offset	E-W Sounding offset 100 feet east of B-7				
3.0	120.2	345.4	1.83E+01	0.3%	100
4.0	113.3	325.6	1.30E+01	0.0%	100
5.0	107.3	308.3	9.81E+00	0.0%	100
7.0	98.2	282.3	6.42E+00	0.0%	100
10.0	91.4	262.7	4.18E+00	0.0%	100
15.0	77.2	221.8	2.35E+00	0.1%	100
20.0	60.8	174.6	1.39E+00	0.0%	100
25.0	48.4	139.1	8.86E-01	0.0%	100
30.0	41.9	120.3	6.38E-01	0.0%	100
40.0	33.9	97.4	3.88E-01	0.0%	100
50.0	32.6	93.6	2.98E-01	0.0%	100
70.0	35.5	101.9	2.32E-01	0.0%	100
100.0	40.1	115.2	1.83E-01	0.0%	100
130.0	43.6	125.2	1.53E-01	0.0%	100
160.0	45.4	130.5	1.30E-01	0.2%	100
Sounding R-4					
	E-W Sounding offset 300 feet east of B-7				
3.0	92.0	264.4	1.40E+01	0.0%	100
4.0	86.3	248.0	9.87E+00	0.0%	100
5.0	82.8	238.0	7.57E+00	0.0%	100
7.0	69.5	199.8	4.54E+00	0.0%	100
10.0	58.8	169.0	2.69E+00	0.0%	100
15.0	48.7	139.9	1.48E+00	0.0%	100
20.0	40.5	116.3	9.26E-01	0.1%	100
25.0	35.8	103.0	6.55E-01	0.0%	100
30.0	31.9	91.7	4.87E-01	0.0%	100
40.0	28.5	81.9	3.26E-01	0.0%	100
50.0	31.7	91.2	2.90E-01	0.1%	100
70.0	32.3	93.0	2.11E-01	0.0%	100
100.0	37.5	107.8	1.72E-01	0.0%	100
130.0	41.3	118.8	1.45E-01	0.0%	100
160.0	42.0	120.7	1.20E-01	0.0%	100

TABLE 1

DC Resistivity Soundings
Wenner Array
Summit/Westward Energy Project
Clatskanie, Oregon

a-spacing (feet)	Apparent Resistivity (Ohm-m)	Apparent Resistivity (Ohm-ft)	V/I (Ohms)	Error	Current (mA)
Sounding R-1					
Boring B-5	N-S Sounding offset 10 feet west of B-5				
3.0	138.2	397.1	2.11E+01	0.0%	20
4.0	135.7	389.8	1.55E+01	0.0%	20
5.0	112.9	324.4	1.03E+01	0.0%	20
7.0	103.5	297.4	6.76E+00	0.0%	20
10.0	96.2	276.3	4.40E+00	0.0%	20
15.0	84.6	243.0	2.58E+00	0.0%	20
20.0	69.1	198.5	1.58E+00	0.0%	20
25.0	54.3	156.1	9.94E-01	0.0%	20
30.0	44.5	127.9	6.79E-01	0.0%	50
40.0	35.5	102.1	4.06E-01	0.0%	50
50.0	33.4	96.0	3.06E-01	0.3%	50
70.0	34.4	98.8	2.25E-01	0.0%	50
100.0	38.0	109.3	1.74E-01	0.0%	100
150.0	42.2	121.3	1.29E-01	0.0%	100
Sounding R-2					
Boring B-3	N-S Sounding offset 10 feet east of B-3				
3.0	139.4	400.7	2.13E+01	0.7%	100
4.0	136.0	390.7	1.55E+01	0.8%	100
5.0	146.6	421.2	1.34E+01	1.0%	100
7.0	113.3	325.5	7.40E+00	0.2%	100
10.0	84.7	243.5	3.87E+00	0.1%	100
15.0	70.6	203.0	2.15E+00	0.1%	100
20.0	59.0	169.4	1.35E+00	0.0%	100
25.0	47.2	135.5	8.63E-01	0.2%	100
30.0	41.0	117.8	6.25E-01	0.0%	100
40.0	34.2	98.2	3.91E-01	0.1%	100
50.0	33.0	95.0	3.02E-01	0.0%	100
70.0	34.5	99.3	2.26E-01	0.0%	100
100.0	39.5	113.5	1.81E-01	0.1%	100
130.0	42.7	122.6	1.50E-01	0.2%	100
160.0	44.6	128.2	1.28E-01	0.0%	100

D.C. Resisitivity

INTRODUCTION

D.C. resistivity (electrical resistivity) techniques measure earth resistivity by driving a direct current (D.C.) signal into the ground and measuring the resulting potentials (voltages) created in the earth. From the data the electrical properties of the earth (the geoelectric section) can be derived. In turn, from those electrical properties we can infer geologic properties of the earth.

In geophysical and geotechnical literature, the terms "electrical resistivity" and "D.C. resistivity" are used synonymously. The term "vertical electric sounding" (VES) is also used to refer to soundings using the D.C resistivity method. The terms "resistivity" or "electrical" are often used to refer to the same methods or techniques, although "electrical" is sometimes used to encompass a broader range of techniques including the electromagnetic methods.

APPLICATIONS

Electrical resistivity of soils and rocks correlates with other soil/ rock properties which are of interest to the geologist, hydrogeologist, geotechnical engineer and/or quarry operator. Several geologic parameters which affect earth resistivity (and its reciprocal, conductivity) include:

- clay content,
- groundwater conductivity,
- soil or formation porosity, and
- degree of water saturation.

D.C. resistivity techniques may be used in the profiling mode (dipole-dipole surveys) to map lateral changes and identify near-vertical features (e.g., fracture zones), or they may be used in the sounding mode (e.g., Schlumberger soundings) to determine depths to geoelectric horizons (e.g., depth to saline groundwater).

Common applications of the D.C. resistivity method include:

- delineation of aggregate deposits for quarry operations
- measuring earth impedance or resistance for electrical grounding circuits or for cathodic protection,
- estimating depth to bedrock, to the water table, or to other geoelectric boundaries, and
- mapping and/or detecting other geologic features.

D.C. resistivity and electromagnetic (EM) techniques both measure electrical properties of the earth, and hence both are used for many of the same applications. Conductivity, which is often reported by EM instruments, is the reciprocal of resistivity.

THEORY OF OPERATION

Figure 2 is a schematic diagram showing the basic principle of D.C. resistivity measurements. Two short metallic stakes (electrodes) are driven about 1 foot into the earth to apply the current to the ground. Two additional electrodes are used to measure the earth voltage (or electrical potential) generated by the current.

Depth of investigation is a function of the electrode spacing. The greater the spacing between the outer current electrodes, the deeper the electrical currents will flow in the earth, hence the greater the depth of exploration. The depth of investigation is generally 20% to 40% of the outer electrode spacing, depending on the earth resistivity structure.

(Continued Next Page)

Figure 1 - D.C. Resistivity Crew In Operation In The Willamette Valley of Oregon

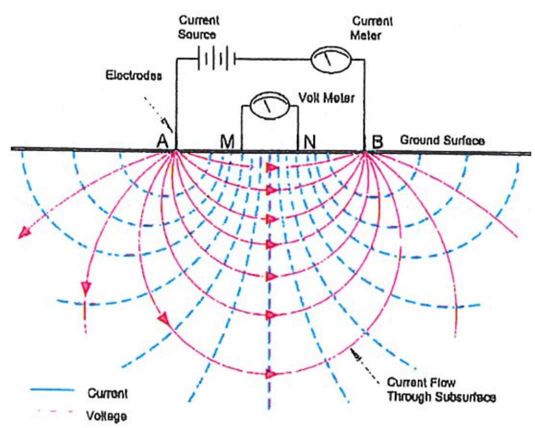

Figure 2 - Schematic Illustrating Basic Concept Of Electrical Resistivity Measurement

thwest Geophysical Associates, Inc.
. Box 1063, Corvallis, OR 97339-1063
(541) 757-7231 Fax: (541) 757-7331
www.nga.com info@nga.com

ATA ANALYSIS & INTERPRETATION

Apparent Resistivity:

Instrument readings (current and voltage) are generally reduced to "apparent resistivity" values. The apparent resistivity is the resistivity of the homogeneous half-space which would produce the observed instrument response for a given electrode spacing. Apparent resistivity is a weighted average of soil resistivities over the depth of investigation.

For soundings a log-log plot of apparent resistivity versus electrode separation is obtained. This is sometimes referred to as the "sounding curve."

Modeling:

Resistivity data is generally interpreted using the "modeling" process: A hypothetical model of the earth and its resistivity structure (geoelectric sections) is generated. The theoretical electrical resistivity response over that model is then calculated. The theoretical response is then compared with the observed field response and differences between observed and calculated are noted. The hypothetical earth model is then adjusted to create a response which more nearly fits the observed data. When this iterative process is automated it is referred to as "iterative inversion" or "optimization."

iqueness

esistivity models are generally not unique; i.e., a large number of earth models can produce the same observed data or sounding curve. In general, resistivity methods determine the "conductance" of a given stratigraphic layer or unit. The conductance is the product of the resistivity and the thickness of a unit. Hence that layer could be thinner and more conductive or thicker and less conductive, and produce essentially the same results. Hence constraints on the model, from borehole data or assumed unit resistivities, can greatly enhance the interpretation.

Deliverables

The end product from a D.C. resistivity survey is generally a "geoelectric" cross section showing thicknesses and resistivities of all the geoelectric units or layers. If borehole data or a conceptual geologic model is available, then a geologic identity can be assigned to the geoelectric units.

A two-dimensional geoelectric section may be made up of a series of one-dimensional soundings joined together to form a two-dimensional section, or it may be a continual two-dimensional cross section. The type of section produced depends on the acquisition parameters and the type of processing applied to the data.

Figure 3 is a two dimensional geoelectric section from a dipole-dipole survey in Alaska. The resistivity survey, part of a water resources investigation, was conducted in order to identify fracture zones with increased porosity. The geophysical objective was to locate conductive fracture zones in the more resistive bedrock. The zone with lower resistivities (1500 to 2000 ohm-meters), which is seen in Figure 3 between 90m and 100m, is indicative of increased water content due to higher fracture porosity in that region.

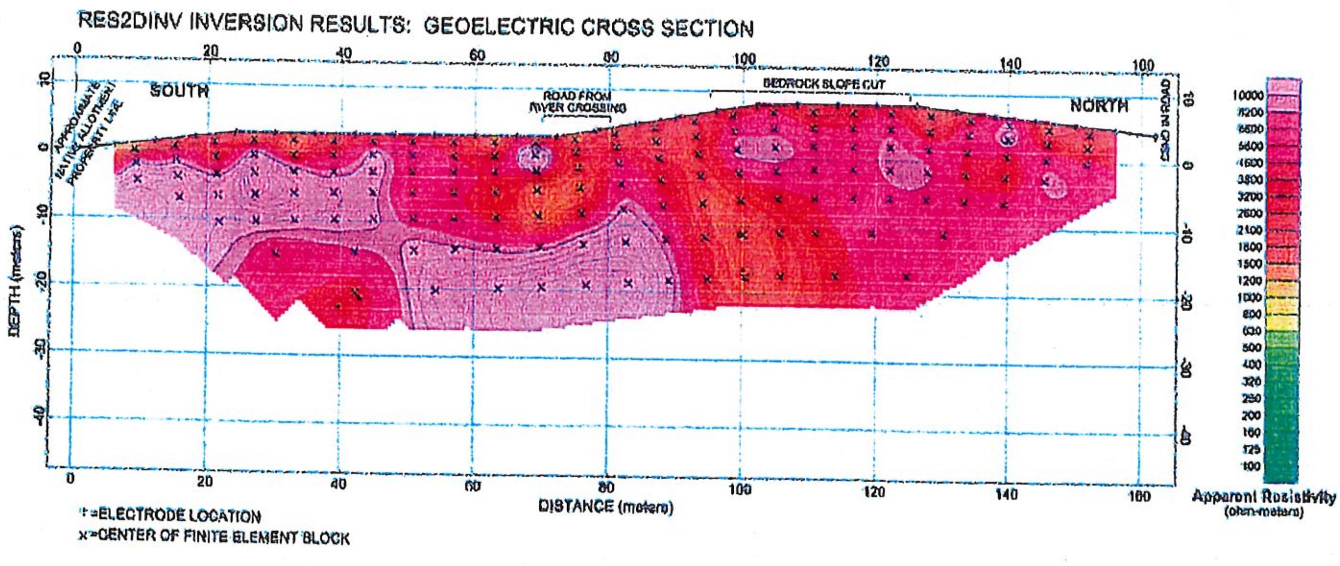

Figure 3 - Geoelectric Model From Dipole-Dipole Resistivity Survey