PRELIMINARY DRAINAGE REPORT FOR BLACKBERRY GROVE ENERGY STORAGE PROJECT

Prepared for:

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APRIL 2025

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1 INTRODUCTION

1.1 PROJECT OVERVIEW

This Preliminary Drainage Report (Report) was prepared by WSP on behalf of Jupiter Power LLC (Jupiter) to address pre- and post-development drainage conditions for the Blackberry Grove Battery Energy Storage System Project (Blackberry Grove BESS or Project). The Project consists of a new BESS facility on approximately 9.5 acres of privately-owned land in Washington County, Oregon (Figure 1.1).

This Report presents a conceptual drainage plan in support of Blackberry Grove BESS pad elevations and storm water management facilities.

1.2 PROJECT DESCRIPTION

The proposed Blackberry Grove BESS will have a capacity of 100 megawatts (MW). The site is located within portions of Township 1 North (T1N), Range 2 West (R2W), Section 14 (Assessor's Parcel Numbers 1N214A004300 and 1N214A004400). The site borders Northwest Bendemeer Road to the west, Northwest West Union road to the south, and Northwest Old Pass Road to the southeast. The PGE West Union substation is located just south of Northwest West Union Road. Major project equipment includes:

- BESS enclosures and controller
- Fire protection system
- Power conversion system
- Project substation
- Internal site access pathways
- Security fencing

This Project will construct the battery units, substation, new gravel interior access pathways, culverts and detention basins. BESS pads will be surfaced with an aggregate base and elevated above the 100-year flood elevation. Detention basins will be designed to store the 25-year storm runoff volume and ensure that 25-year post-development peak runoff rates do not exceed 25-year pre-development rates. Access to the site will be via Northwest Bendemeer Road to the west and Northwest West Union road to the south of the Project (see Figure 1.2).

1.3 EXISTING SITE CONDITIONS

The Project area is within the Tualatin River Basin, situated on relatively flat land. The northern portion of the Project site is primarily pasture/hay. The southern portion contains a rural residence surrounded by several buildings, including a barn that will be removed during site preparation and construction. A 5- to 7-foot-tall earthen berm covered by mature evergreen trees exists along the perimeter of the site, preventing any offsite flow from entering. The topography in the Project area is flat within the surrounding berm, with elevations ranging from approximately 246 to 257 feet above mean sea level. Several culverts located at the perimeter of the site allow the field to drain through the berm. The onsite flow generally drains south to a roadside ditch south of the Project site, which then drains southeast towards Holcomb Lake. Site conditions can be found on Figure 1.2.

1.4 PURPOSE OF REPORT

The purpose of this Report is to establish the 25-year and 100-year storm flows across the site under pre- and post-development conditions and present a preliminary stormwater management plan for the Project. This Report presents an overview of the hydrologic and hydraulic methods used, and 25-year and 100-year flood results. This report is organized as follows:

- Section 1 provides general project background;
- Section 2 presents previous studies related to the Project;
- Section 3 discusses hydrologic and hydraulic analysis;
- Section 4 presents hydrologic and hydraulic analysis results;
- Section 5 presents stormwater management recommendations;
- Section 6 provides the conclusion; and
- Section 7 lists the references.

Blackberry Grove BESS Project

N

0 0.3 0.6
Miles

Vicinity Map
Figure 1.1

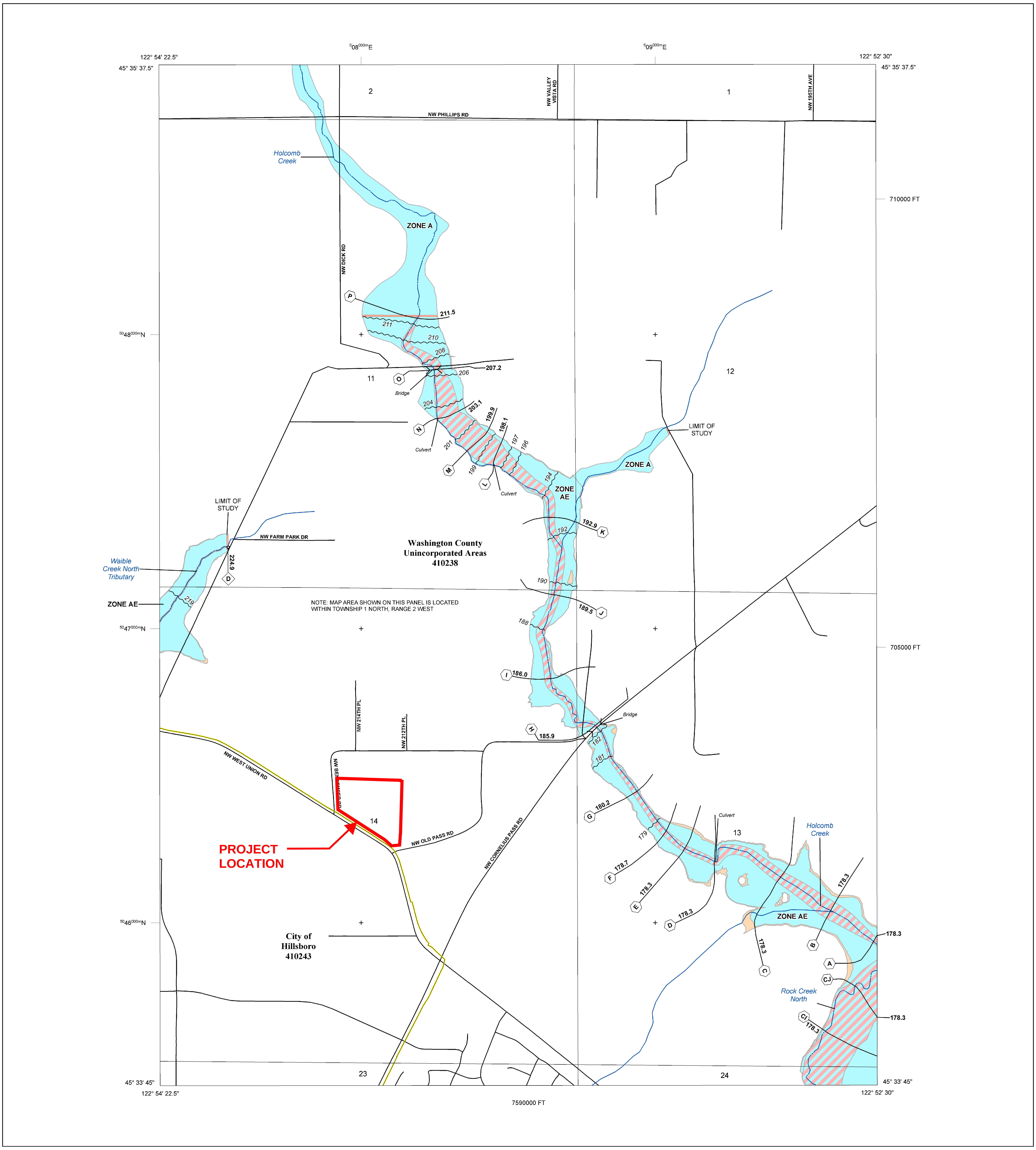
Jupiter POWER
Blackberry Grove BESS Project
Legend
Site Boundary
Codified Setbacks
Substation
Maintenance Storage Container
Existing Buildings to be Removed
Detention Basin
Interior Pathways
SMA Power Conversion System
Transformer and Distribution Panel
Battery Storage Container
Existing 18" RCP Culvert
Proposed 12" CMP Culvert
Fence
1-ft Contours
NW Bendemeer Road
NW West Union Road
NW Old Pass Road
PGE West Union Substation
N
0 50 100
Feet
wsp
Site Map
Figure 1.2

2 PREVIOUS STUDIES

2.1 FLOOD INSURANCE RATE MAPS

The Federal Emergency Management Agency (FEMA) publishes Flood Insurance Rate Maps (FIRMs) that delineate special flood hazard areas (SFHAs) subject to flooding by the 1% annual chance (100-year) storm event.

Figure 2.1 is a reproduction of the FIRM panel for Washington County, where the Project is located. According to FIRM Number 41067C0334F, Panel 334 of 650, effective date October 19, 2018, the Project is not within a SFHA and is identified as unshaded Zone X, areas outside the 0.2% annual chance floodplain.



FLOOD HAZARD INFORMATION

SEE FIS REPORT FOR ZONE DESCRIPTIONS AND INDEX MAP

THE INFORMATION DEPICTED ON THIS MAP AND SUPPORTING DOCUMENTATION ARE ALSO AVAILABLE IN DIGITAL FORMAT AT

HTTP://MSC.FEMA.GOV

SPECIAL FLOOD HAZARD AREAS	Without Base Flood Elevation (BFE) *Zone A,V, A99*
	With BFE or Depth *Zone AE, AO, AH, VE, AR*
	Regulatory Floodway
OTHER AREAS OF FLOOD HAZARD	0.2% Annual Chance Flood Hazard, Areas of 1% annual chance flood with average depth less than one foot or with drainage areas of less than one square mile *Zone X*
	Future Conditions 1% Annual Chance Flood Hazard *Zone X*
	Area with Reduced Flood Risk due to Levee See Notes. *Zone X*
OTHER AREAS	NO SCREEN Areas Determined to be Outside the 0.2% Annual Chance Floodplain *Zone X*
	Area of Undetermined Flood Hazard *Zone D*
GENERAL STRUCTURES	Channel, Culvert, or Storm Sewer
	Levee, Dike, or Floodwall
	E 18.2 17.5 Cross Sections with 1% Annual Chance Water Surface Elevation (BFE)
	8 Coastal Transect
	Coastal Transect Baseline
	Profile Baseline
	Hydrographic Feature
	513 Base Flood Elevation Line (BFE)
OTHER FEATURES	Limit of Study
	Jurisdiction Boundary

NOTES TO USERS

For information and questions about this map, available products associated with this FIRM including historic versions of this FIRM, how to order products or the National Flood Insurance Program in general, please call the FEMA Map Information eXchange at 1-877-FEMA-MAP (1-877-336-2627) or visit the FEMA Map Service Center website at http://msc.fema.gov. Available products may include previously issued Letters of Map Change, a Flood Insurance Study Report, and/or digital versions of this map. Many of these products can be ordered or obtained directly from the website. Users may determine the current map date for each FIRM panel by visiting the FEMA Map Service Center website or by calling the FEMA Map Information eXchange.

Communities annexing land on adjacent FIRM panels must obtain a current copy of the adjacent panel as well as the current FIRM Index. These may be ordered directly from the Map Service Center at the number listed above.

For community and countywide map dates refer to the Flood Insurance Study report for this jurisdiction.

To detemine if flood insurance is available in the community, contact your Insurance agent or call the National Flood Insurance Program at 1-800-638-6620.

Base map information shown on this FIRM was derived from multiple sources. Base Map files were provided in digital format by the Metro Data Resource Center. This information was compiled from many local sources and include transportation features, water features, political boundaries, and Public Land Survey System features.

SCALE

N

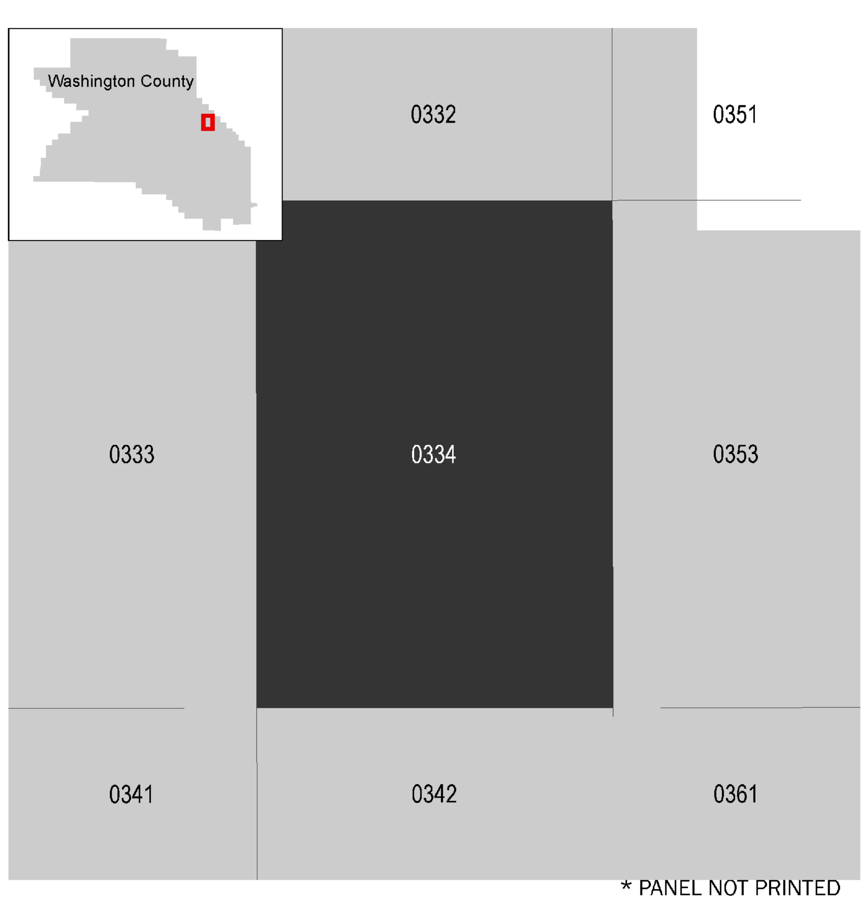
Map Projection: Universal Transverse Mercator Zone 10N; North American Datum 1983; Western Hemisphere; Vertical Datum: NAVD 88

1 inch = 500 feet 1:6,000

0 500 1,000 2,000 Feet

0 125 250 500 Meters

PANEL LOCATOR



* PANEL NOT PRINTED

FEMA

National Flood Insurance Program

NATIONAL FLOOD INSURANCE PROGRAM

FLOOD INSURANCE RATE MAP

WASHINGTON COUNTY, OREGON

And Incorporated Areas

PANEL 334 OF 650

FEMA

Panel Contains:

COMMUNITY	NUMBER	PANEL	SUFFIX
HILLSBORO, CITY OF	410243	0334	F
WASHINGTON COUNTY	410238	0334	F

FEMA FLOODPLAIN PLAN

FIGURE 2.1

VERSION NUMBER
2.3.3.3

MAP NUMBER
41067C0334F

MAP REVISED
OCTOBER 19, 2018

3 HYDROLOGIC AND HYDRAULIC ANALYSIS

This section addresses hydrologic and hydraulic criteria implemented for the analysis of the proposed stormwater management facilities for the Project. The analysis was conducted in accordance with the *On-Site Stormwater Disposal System (OSDS) Design and Construction Minimum Guidelines and Requirements (2007)* and *Oregon Department of Transportation (ODOT) Hydraulics Design Manual (HDM) (2014)*.

3.1 DRAINAGE CRITERIA

This study adheres to the ODOT HDM (2014) guidelines as outlined in Table 3.1. Peak runoff flow rates were determined using the rational method for the 25-year and 100-year storm events.

Table 3.1 Applicable ODOT Hydraulics Design Manual Methods and Procedures

TOPIC	DESIGN MANUAL SECTION OR SUBSECTION	COMMENTS
Rainfall Intensity-Duration-Recurrence (I-D-R) Interval Curves	Chapter 7, Appendix A	Determination of rainfall I-D-R curve zone for the Project site.
Rational Method	Chapter 7, Appendix F	Determination of pre- and post-development condition runoff peak discharge
Culvert	Chapter 9	Determination of culvert size
Storage Facility and Outlet Structure	Chapter 12	Determination of storage volume and sizing of auxiliary outlet

3.2 DATA SOURCES

Input data for the hydrologic calculations were obtained from a variety of sources. This information is summarized in Table 3.2

Table 3.2 Data Sources

DATA	PURPOSE	SOURCE	FORMAT
Elevation	Determining drainage area and points of concentration	LiDAR Survey (Provided by Jupiter)	Contour; CADD file
Land use	Estimating composite runoff coefficients and time of concentration	National Land Cover Database (NLCD), Google satellite imagery, and proposed site plan	Raster; CADD file
Project Boundary	Limits of subbasin boundary	Jupiter	CADD file

DATA	PURPOSE	SOURCE	FORMAT
FEMA Flood Zones	Reference	NFHL Data - County	png
Aerial Photography	Reference	ArcGIS Map Service	Raster

3.3 DRAINAGE BASIN

A 5- to 7-foot-tall earthen berm, covered by mature evergreen trees, exists along the site's perimeter, preventing any offsite flow from entering. The Project only has onsite flows, and the Project boundary depicts the drainage basin boundary, which is approximately 9.5 acres. The drainage basin is divided into two subbasins: ON1 and ON2. The pre-development condition subbasins were delineated primarily using site contours provided by Jupiter, while the post-development condition subbasins were delineated based on proposed site grading. In the pre-development condition, subbasin ON1 drains to the southwest corner of the site, while subbasin ON2 drains to the southeast corner.

In the post-development condition, subbasin ON1 is further divided into subbasin ON1A and subbasin ON1B. Subbasin ON1A drains west along the north side of the site access road, which then naturally drains south via a proposed culver under the site access road along the west side of the berm. Subbasin ON1B drains to the southwest corner of the site. The pre- and post-development condition subbasins are shown in Figures 3.1 and 3.2, respectively.

3.4 HYDROLOGIC ANALYSIS

The rational method was used for evaluating the 25-year and 100-year peak flow rates in both pre- and post-development conditions. The peak runoff calculations derived from the rational method are included in Appendices A.2.7 and A.2.8. The input parameters used in the rational method are described below.

3.4.1 LAND USE

According to the National Land Cover Database (NLCD), most of the Project area is located on land designated as pasture/hay with developed open space to low intensity development in some portions of the site. Existing land use for the Project site from NLCD is shown on Figure A.1.1 in Appendix A.

In the pre-development condition, land use for the onsite basin was assigned based on aerial imagery and the NLCD. In the post-development condition, land use for the onsite basin was assigned based on the proposed plan of development. Figures A.1.2 and A.1.3 in Appendix A show the pre- and post-development land use conditions, respectively.

3.4.2 RUNOFF COEFFICIENT

The composite runoff coefficients for pre- and post-development conditions were calculated based on land use types for each respective condition. According to ODOT HDM, the runoff coefficient for meadows and pasture is 0.25, for light residential areas is 0.35, and for heavy industrial areas is 0.6 on flat land (Appendix A.2.3). These coefficients apply to storms with recurrence intervals of 10-years or less. Adjustment factors of 1.1 and 1.25 are applied for the 25-year and 100-year storms, respectively, as per the ODOT HDM (Appendix A.2.4).

3.4.3 RAINFALL INTENSITY

The project was determined to be located in rainfall Zone 8 using the rainfall zone map (Appendix A.2.1). The rainfall intensity was then selected from the Zone 8 intensity-duration-recurrence interval (I-D-R) curve at a duration equal to the time of concentration (T_c) (Appendix A.2.2).

3.4.4 TIME OF CONCENTRATION

The drainage path used to calculate the time of concentration consists of two segments. The first 100 feet is considered overland sheet flow, while the remaining segment is shallow concentrated flow. The travel time for each flow segment is computed, and the time of concentration is the sum of these individual travel times, as follows:

$$T_c = T_{osf} + T_{scf}$$

where,

T_c = Time of concentration (min)

T_{osf} = Travel time for the overland sheet flow segment (min)

T_{scf} = Travel time for the shallow concentrated flow segment (min)

The kinematic wave equation is used to estimate the time of concentration associated with overland sheet flow. The equation is shown below:

$$T_{osf} = \frac{0.93(L^{0.6}n^{0.6})}{(i^{0.4}S^{0.3})}$$

where,

T_{osf} = Travel time for the overland sheet flow segment (min)

L = Length of overland sheet flow segment (ft)

n = Manning's roughness coefficient

i = Rainfall intensity (in/hr)

S = The average slope of the overland area (ft/ft)

Determining the time of concentration for overland sheet flow involves a trial-and-error process, as both the flow time and the rainfall intensity are unknown (Appendix A.2.7).

The travel time for the shallow concentrated flow segment is determined using the following formula:

$$T_{scf} = \frac{L}{60V}$$

where,

T_{scf} = Travel time for the shallow concentrated flow segment (min)

L = Length of shallow concentrated flow segment (ft)

V = Average flow velocity (ft/s)

3.4.5 PEAK DISCHARGE

The peak discharge for the 25-year and 100-year events was calculated using the following equation:

$$Q = C_f C\ i\ A$$

where,

Q = Peak flow (cfs)

C_f = Runoff coefficient adjustment factor

C = Runoff coefficient

i = Rainfall intensity (in/hr)

A = Drainage area (acre)

3.5 FLOW DEPTH CALCULATION

The 100-year flow depth along the BESS and substation pad on the north and east edges of the pad was determined using FlowMaster (see Appendix B.3). This calculation assumes a wide channel geometry, and takes into account roughness, discharge, and channel slope.

3.6 STORAGE FACILITY

3.6.1 VOLUME

Storage facilities are designed to maintain the post-development condition peak runoff rate equal to the pre-development condition peak runoff rate for a 25-year design storm event. The storage volume is determined using the simplified rational method (Appendix B.1.1). The required storage is a function of the depletion (release) rate (Q_o), inflow (runoff) rate (Q_i), and the time of detention (T). The general equation is:

$$V_S = T\ (Q_i - Q_o)$$

where,

V_S = Storage volume (cubic feet)

Q_i = Post-development condition design inflow rate when maximum storage volume is required (cfs)

Q_o = Peak design release rate (cfs)

T = Time duration to design inflow rate Q_i (s)

The storage volume is calculated for as many time intervals as necessary to find the peak volume required as per section ODOT HDM (2014). If the post-development time of concentration is less than the time interval where peak storage occurs, impervious area used to calculate peak storage is adjusted using the following equation:

$$CA_2 = CA_1 \left(\frac{T}{T_{c'}}\right)$$

where,

$T_{c'}$ = Time of concentration in the post-development condition (min)

T = Time interval where peak storage occurs (min)

CA_1 = impervious area for post-development condition using entire contributing area in basin A_1 (acre)

CA_2 = adjusted impervious area for post-development condition using reduced contributing area A_2 corresponding to T (acre)

3.6.2 PRIMARY OUTLET

An orifice is used as a primary outlet control structure designed using the 25-year pre-development flow. The diameter of the orifice is calculated using the following equation:

$$D = \left[\frac{Q}{3.78{H_o}^{0.5}}\right]^{0.5}$$

where,

D = Diameter of orifice (feet)

Q = Discharge (cfs)

H_o = Effective head on the orifice, measured from the centroid of the orifice opening (feet)

3.6.3 AUXILIARY OUTLET

A riser pipe is used as an auxiliary outlet designed using the 100-year post-development flow. The head for the riser pipe is calculated using the following equation for a given discharge, riser pipe diameter and the sharp created weir length:

$$H = \left(\frac{Q}{3.33L}\right)^{2/3}$$

where,

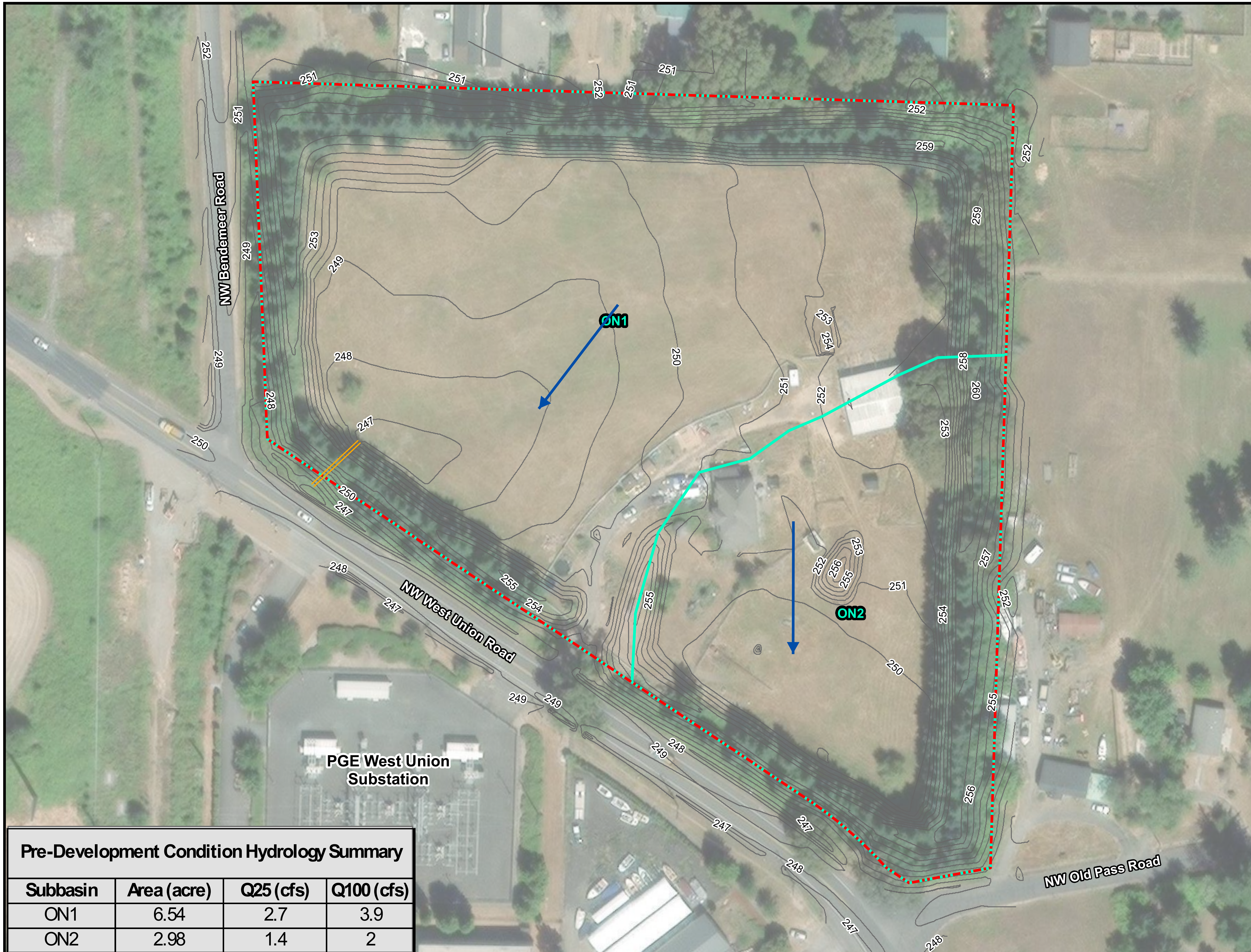
H = Head (feet)

Q = Discharge (cfs)

L = Sharp crested weir length (feet)

3.7 CULVERT

A culvert under the access road to the site from the west is sized using ODOT HDM Culvert Performance Chart, which is included in Appendix B.2.



Jupiter POWER

Blackberry Grove BESS Project

Legend

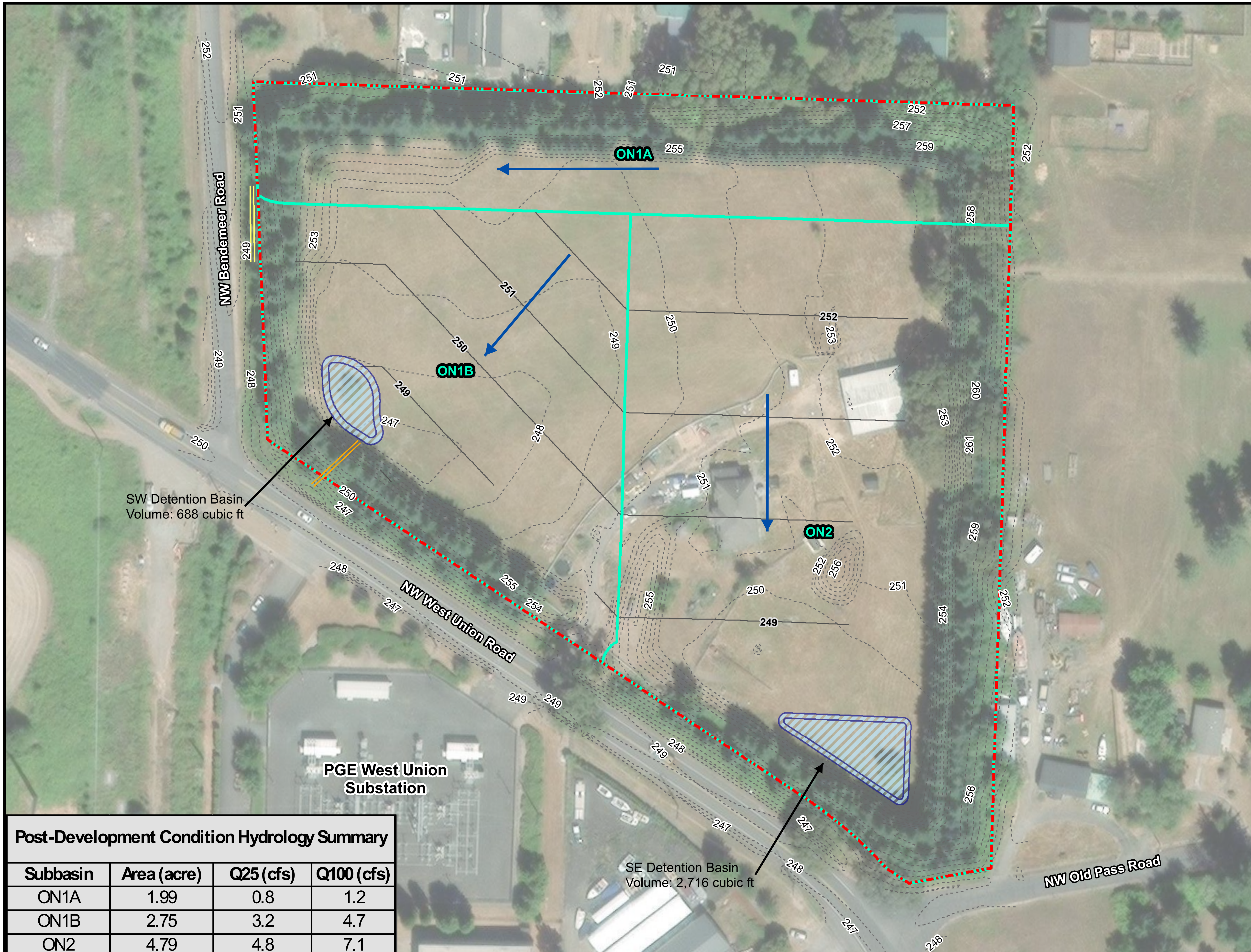
- Site Boundary
- Subbasins
- Flow Direction
- 1-ft Contours
- Existing 18" RCP Culvert

Pre-Development Condition Hydrology Summary

Subbasin	Area (acre)	Q25 (cfs)	Q100 (cfs)
ON1	6.54	2.7	3.9
ON2	2.98	1.4	2

wsp

Pre-Development Condition
Subbasin Map
Figure 3.1



Post-Development Condition Hydrology Summary			
Subbasin	Area (acre)	Q25 (cfs)	Q100 (cfs)
ON1A	1.99	0.8	1.2
ON1B	2.75	3.2	4.7
ON2	4.79	4.8	7.1

Jupiter POWER

Blackberry Grove BESS Project

Legend

- Existing 18" RCP Culvert
- Detention Basin
- Site Boundary
- Subbasins
- Flow Direction
- Proposed 12" CMP Culvert
- Existing 1-ft Contours
- Proposed 1-ft Contours

N

0 50 100

Feet

wsp

Post-Development Condition Subbasin Map
Figure 3.2

4 HYDROLOGIC AND HYDRAULIC ANALYSIS RESULTS

4.1 PRE-DEVELOPMENT CONDITION HYDROLOGIC ANALYSIS RESULTS

The 25-year peak flows from subbasins ON1 and ON2 in pre-development condition, calculated using the rational method, are 2.7 cfs and 1.4 cfs, respectively. Similarly, the 100-year peak flows from subbasins ON1 and ON2 in pre-development condition, calculated using the rational method, are 3.9 cfs and 2 cfs, respectively. The peak flows are shown in Figure 3.1, and the rational method calculations are included in Appendix A.2.7.

4.2 POST-DEVELOPMENT CONDITION HYDROLOGIC ANALYSIS RESULTS

The 25-year peak flows from subbasins ON1A, ON1B and ON2 in post-development condition, calculated using the rational method, are 0.8 cfs, 3.2 cfs and 4.8 cfs, respectively. Similarly, the 100-year peak flows from subbasins ON1A, ON1B and ON2 in post-development condition, calculated using the rational method, are 1.2 cfs, 4.7 cfs and 7.1 cfs, respectively. The peak flows are shown in Figure 3.2, and the rational method calculations are included in Appendix A.2.8.

4.3 FLOW DEPTH RESULTS

The normal flow depth during 100-year event against the BESS and substation pad on both the north and east edges of the pad was calculated to be 0.1 feet (See Appendix B.3).

4.4 STORAGE FACILITY SIZING

Two detention basin storage facilities, also referred to herein as detention basins, are designed to manage the increased flow due to proposed development, ensuring that the peak runoff rate after development remains the same as the pre-development rate during a 25-year design storm event, as described in Section 3.5. Runoff from the proposed development will be captured in these detention basins and released into the natural flow path at the pre-development flow rate. One basin, referred to herein as SW Detention Basin, will be located at the site's southwest corner, while the other, referred to herein as SE Detention Basin, will be at the southeast corner. The SW Detention Basin will capture runoff from subbasin ON1B and release it at a rate equal to the pre-development rate for ON1, minus the flow rate for ON1A. This ensures that the total flow from ON1A and ON1B matches the pre-development rate for ON1. The SE Detention Basin will capture runoff from subbasin ON2. The required storage volumes for SW and SE Detention Basins are calculated to be 688 cubic feet and 2,716 cubic feet, respectively. Storage volume calculations are included in Appendix B.1.1.

4.5 OUTLET STRUCTURE

The diameter of the orifice for the SW Detention Basin is determined to be 7 inches, with a design head of 2 feet. For the auxiliary riser pipe, the diameter is calculated to be 18 inches, corresponding to a design head of 0.5 feet. Detailed calculations can be found in Appendix B.1.2.

The diameter of the orifice for the SE Detention Basin is determined to be 8 inches, with a design head of 0.5 feet. For the auxiliary riser pipe, the diameter is calculated to be 24 inches, corresponding to a design head of 0.5 feet. Detailed calculations can be found in Appendix B.1.2.

5 STORMWATER MANAGEMENT RECOMMENDATIONS

Figure 3.2 shows the locations of the proposed facilities described below.

5.1 DETENTION BASIN

Two detention basins, the SW and SE Detention Basins, are proposed on the site's southwest and southeast corners, respectively, to collect onsite runoff volume. The storage volume of the SW Detention Basin is approximately 688 cubic feet, while the SE Detention Basin is approximately 2,716 cubic feet. They are designed to capture the increase in runoff volume and release post-development condition runoff at the pre-development condition rate.

5.2 CULVERT

A 12-inch corrugated metal pipe (CMP) culvert is proposed under the site access road from the west. The headwater depth at the culvert is estimated to be 0.625 feet using the culvert performance chart included in Appendix B.2.

6 CONCLUSION

The Blackberry Grove BESS project is being constructed on approximately 9.5 acres of privately-owned land in Washington County, Oregon. The purpose of this Preliminary Drainage Study is to establish the 25-year and 100-year storm hydrology, flow rate, and runoff volume that will ultimately be used for the design of the substation and battery pad elevations and drainage facilities.

The storm scenario was evaluated using the rational method. Under pre-development condition, flows run through the site from north to south as sheet flow and shallow concentrated flow. Offsite flow from the north is diverted around and away from the site via a 5- to 7-foot-tall earthen berm surrounding the site.

Recommended stormwater management facilities include a culvert under the site access road from the west and two detention basins located at the site's southwest and southeast corners. These basins are designed to store the 25-year storm runoff volume and ensure that post-development peak runoff rates do not exceed pre-development rates. It is recommended that all substation equipment be elevated a minimum of 12-inches above the 100-year flow depth established in this study.

7 REFERENCES

1. Bentley Systems Inc., 2020. FlowMaster CONNECT Edition Update 3, March 2020.
2. Federal Emergency Management Agency (FEMA), Flood Insurance Rate Map (FIRM), 2018, Map Number 41067C0334F.
3. Multi-Resolution Land Characteristic Consortium (MRLC), National Land Cover Database (NLCD), 2023. Retrieved from https://www.mrlc.gov/data?f%5B0%5D=project_tax_term_term_parents_tax_term_name%3AAnnual%20NLCD
4. Oregon Department of Transportation (ODOT), Highway Division, Hydraulics Design Manual, April 2014. Retrieved from https://www.oregon.gov/odot/hydraulics/Docs_Hydraulics_Manual/HDM_Complete.pdf
5. Washington County, Oregon, On-site Stormwater Disposal System (OSDS), Design and Construction Minimum Guidelines and Requirements, September 2007. Retrieved from https://www.washingtoncountyor.gov/lut/building-services/documents/onsite-stormwater-disposal-system-guidelines/download?inline

APPENDIX A – HYDROLOGIC PARAMETERS AND ANALYSIS

- A.1 LAND USE MAP
- A.2 RATIONAL METHOD

A.1 Land Use Map

- A.1.1 NLCD Land Use Map
- A 1.2 Pre-Development Condition Land Use Map
- A.1.3 Post-Development Condition Land Use Map

A.1.1 NLCD Land Use Map

NLCD Land Use Map
Figure A.1.1

A.1.2 Pre-Development Condition Land Use Map

Pre-Development Condition
Land Use Map
Figure A.1.2

A.1.3 Post-Development Condition Land Use Map

Jupiter POWER

Blackberry Grove BESS Project

Legend

- Site Boundary
- Proposed Improvements
- Industrial
- Meadows and Pasture Land

N

0 50 100

Feet

wsp

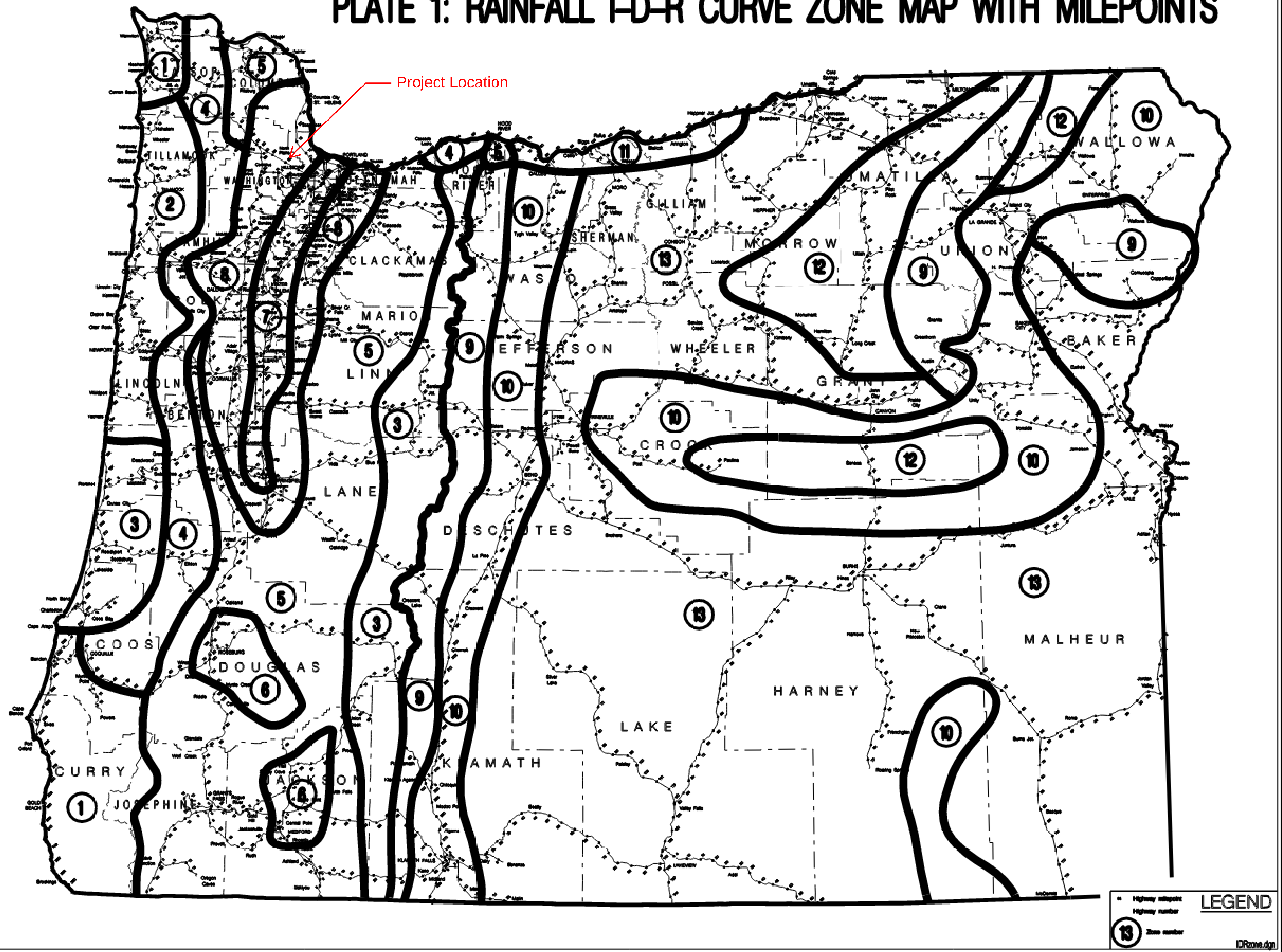
Post-Development Condition Land Use Map
Figure A.1.3

A.2 Rational Method

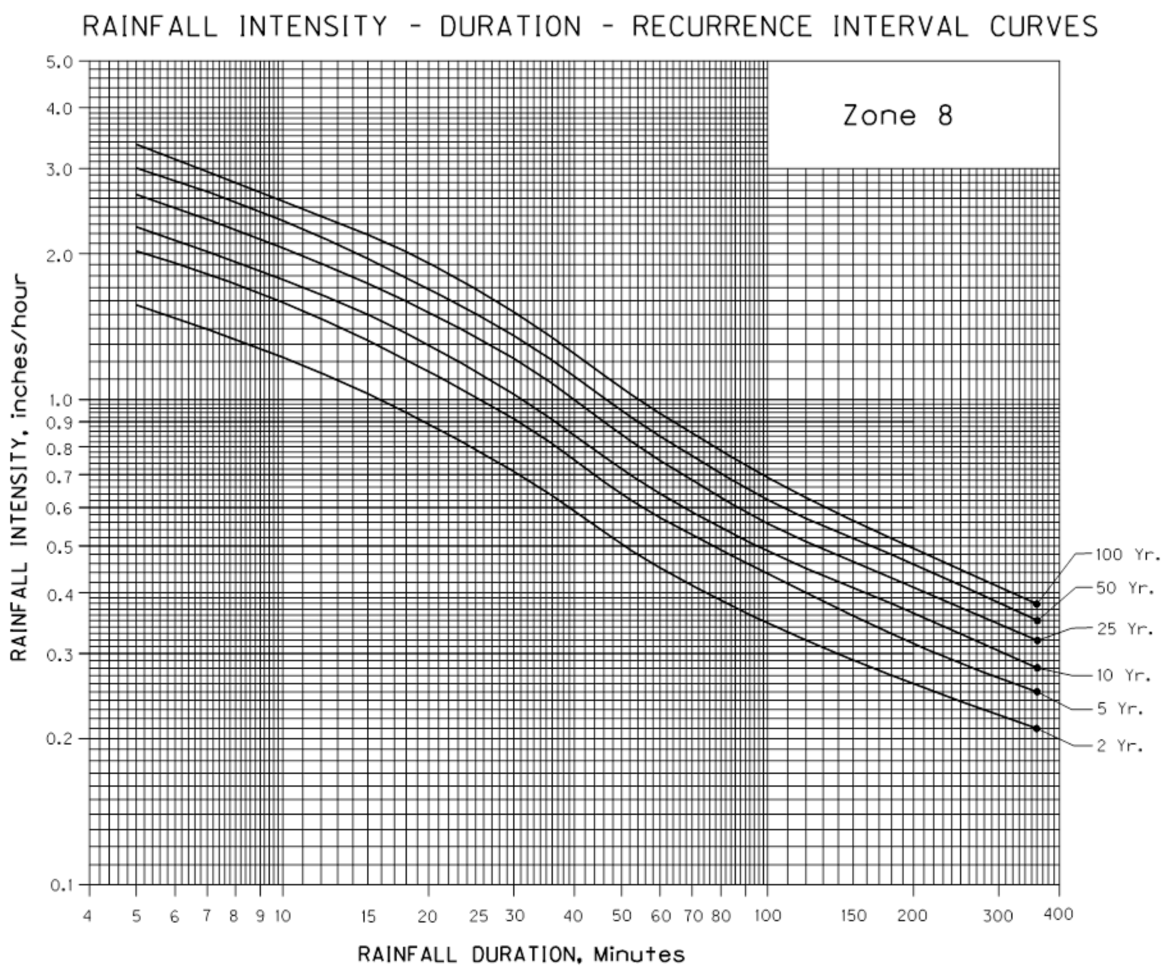
- A.2.1 ODOT HDM Appendix A Plate 1A – Rainfall IDR Curve Zone With Mile Points
- A.2.2 ODOT HDM Appendix A Rainfall Intensity-Duration-Recurrence Interval Curves Zone 8
- A.2.3 ODOT HDM Appendix F Table 1 Runoff Coefficients for the Rational Method
- A.2.4 ODOT HDM Appendix F Table 2 Runoff Coefficients Adjustment Factors
- A.2.5 ODOT HDM Appendix F Table 3 MANNING'S Roughness Coefficients for Overland Sheet Flow
- A.2.6 ODOT HDM Appendix F Figure 1 Shallow Concentrated Flow Velocities
- A.2.7 Pre-Development Condition Calculations
- A.2.8 Post-Development Condition Calculations

A.2.1 ODOT HDM Appendix A Plate 1A – Rainfall IDR Curve Zone With Mile Points

PLATE 1: RAINFALL I-D-R CURVE ZONE MAP WITH MILEPOINTS



A.2.2 ODOT HDM Appendix A Rainfall Intensity-Duration-Recurrence Interval Curves Zone 8



RAINFALL INTENSITY - DURATION - RECURRENCE INTERVAL CURVES
Zone 8
RAINFALL INTENSITY, inches/hour
5.0
4.0
3.0
2.0
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
4
5
6
7
8
9
10
15
20
25
30
40
50
60
70
80
100
150
200
300
400
RAINFALL DURATION, Minutes
100 Yr.
50 Yr.
25 Yr.
10 Yr.
5 Yr.
2 Yr.

A.2.3 ODOT HDM Appendix F Table 1 Runoff Coefficients for the Rational Method

Table 1 Runoff Coefficients for the Rational Method

	FLAT	ROLLING	HILLY
Pavement & Roofs	**0.90**	**0.90**	**0.90**
Earth Shoulders	0.50	0.50	0.50
Drives & Walks	0.75	0.80	**0.85**
Gravel Pavement	**0.85**	**0.85**	**0.85**
City Business Areas	0.80	**0.85**	**0.85**
Apartment Dwelling Areas	0.50	0.60	0.70
Light Residential: 1 to 3 units/acre	$\boxed{0.35}$	0.40	0.45
Normal Residential: 3 to 6 units/acre	0.50	0.55	0.60
Dense Residential: 6 to 15 units/acre	0.70	0.75	0.80
Lawns	0.17	0.22	0.35
Grass Shoulders	0.25	0.25	0.25
Side Slopes, Earth	0.60	0.60	0.60
Side Slopes, Turf	0.30	0.30	0.30
Median Areas, Turf	0.25	0.30	0.30
Cultivated Land, Clay & Loam	0.50	0.55	0.60
Cultivated Land, Sand & Gravel	0.25	0.30	0.35
Industrial Areas, Light	0.50	0.70	0.80
Industrial Areas, Heavy	$\boxed{0.60}$	0.80	**0.90**
Parks & Cemeteries	0.10	0.15	0.25
Playgrounds	0.20	0.25	0.30
Woodland & Forests	0.10	0.15	0.20
Meadows & Pasture Land	$\boxed{0.25}$	0.30	0.35
Unimproved Areas	0.10	0.20	0.30

Note:

- ***Impervious surfaces in bold***
- *Rolling = ground slope between 2 percent to 10 percent*
- *Hilly = ground slope greater than 10 percent*

A.2.4 ODOT HDM Appendix F Table 2 Runoff Coefficients Adjustment Factors

Table 2 Runoff Coefficient Adjustment Factors

RECURRENCE INTERVAL	RUNOFF COEFFICIENT ADJUSTMENT FACTOR
10 years or less	1.0
25 years	$\boxed{1.1}$
50 years	1.2
100 years	$\boxed{1.25}$

- **Time of Concentration "T_c" -** The time of concentration (T_c), is defined as the time it takes for runoff to travel from the hydraulically most distant point in the watershed to the point of reference downstream. Most drainage paths consist of overland flow segments as well as channel flow segments. The overland flow component can be further divided into a sheet flow segment and a shallow concentrated flow segment. Urban drainage basins often will have one or more pipe flow segments. The travel time is computed for each flow segment and the time of concentration is equal to the sum of the individual travel times, as follows:

  $$T_c = T_{osf} + T_{scf} + T_{ocf} + T_{pf} \qquad \text{(Equation 3)}$$

  Where:

  T_c = Time of concentration in minutes (min.)
  T_{osf} = Travel time for the overland sheet flow segment in minutes (min.)
  T_{scf} = Travel time for the shallow concentrated flow segment in minutes (min.)
  T_{ocf} = Travel time for the open-channel flow segment(s) in minutes (min.)
  T_{pf} = Travel time for the pipe flow segment(s) in minutes (min.)

  The drainage path used to determine the time of concentration need not include all of the listed segments. As an example, a roadway pavement bounded by curbs and drained by an inlet connected to a storm drain will have segments of overland sheet flow (pavement), open-channel flow (gutter), and pipe flow (storm drain). There is no shallow concentrated flow segment.

The travel times for the flow segments are determined as follows.

- **Overland Sheet Flow -** Overland sheet flow is shallow flow over a plane surface. It occurs in the furthest upstream segment of the drainage path, which is located immediately downstream from the drainage divide. The length of the overland sheet flow segment is the shorter of: the distance between the drainage divide and the upper end of a defined channel,

A.2.5 ODOT HDM Appendix F Table 3 MANNING'S Roughness Coefficients for Overland Sheet Floweet

or a distance of 300 feet. The overland sheet flow velocity is usually slower than the velocities further downstream.

The kinematic wave equation can be used to estimate the time of concentration associated with overland sheet flow. The equation is shown below, and it is only applicable for travel

$$T_{osf} = \frac{0.93(L^{0.6}n^{0.6})}{(i^{0.4}S^{0.3})} \qquad \text{(Equation 4)}$$

distances equal to or less than 300 feet.

Where:

T_{osf} = Travel time for the overland sheet flow segment in minutes (min.)
L = Length of the overland sheet flow segment in feet (ft)
n = Manning's roughness coefficient (See Table 3)
i = Rainfall intensity in inches per hour (in/hr) See Appendix A.
S = The average slope of the overland area in feet per feet (ft/ft)

Note: Calculating the time of concentration for overland sheet flow is an iterative or trial and error solution because both the flow time and the rainfall intensity are unknown. The procedure is illustrated in the Example.

Table 3 Manning's Roughness Coefficients for Overland Sheet Flow

(Maximum Flow Depth = 1 inch)

Pavement & Roofs	0.014
City Business Areas	0.014
Graveled Surfaces	0.020
Apartment Dwelling Areas	0.050
Industrial Areas	0.050
Urban Residential Areas (more than 6 units acre)	0.080
Meadows, Pastures & Range Land	0.150
Rural Residential Areas (more than 6 units acre)	0.240
Playgrounds, Light Turf	0.240
Parks & Cemeteries, Heavy Turf	0.400
Woodland & Forests	0.400

- **Shallow Concentrated Flow -** Overland sheet flow often becomes either shallow concentrated flow or open-channel flow as it progresses down the drainage. It becomes

A.2.6 ODOT HDM Appendix F Figure 1 Shallow Concentrated Flow Velocities

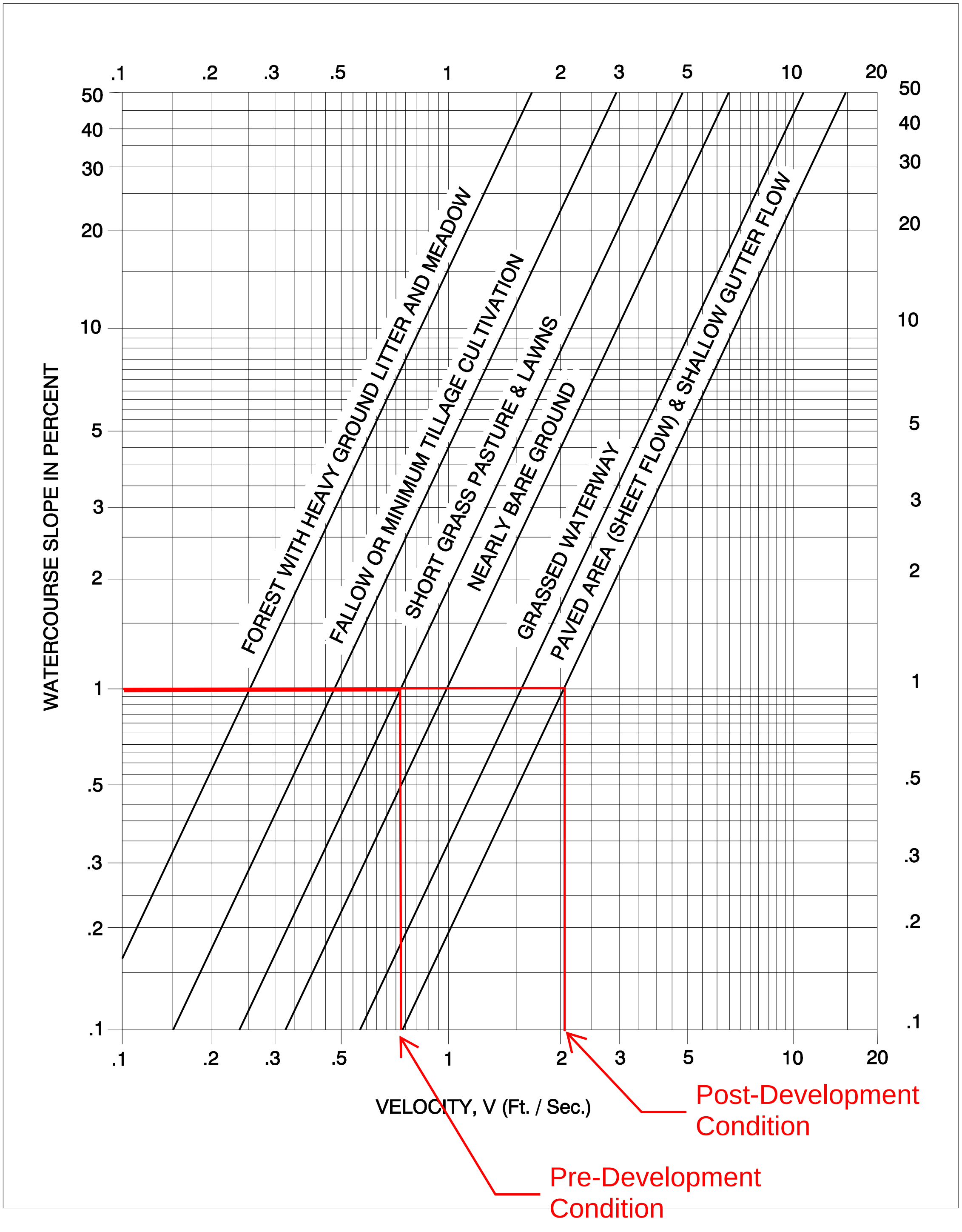


Figure 1 Shallow Concentrated Flow Velocities

A.2.7 Pre-Development Condition Calculations

Pre-Development Condition

Runoff Coefficients

Landuse	Runoff Coefficient[1]
	C
Meadows & Pasture Land	0.25
Light Residential: 1 to 3 units/acre	0.35

[1]ODOT Hydraulics Manual, Hydrology Appendix F, Table 1

Composite Runoff Coefficients

Subbasin	Total Area (acres)	Total Area (sf)	Land Use Area per Subbasin				Total Weighted Runoff Coefficients
			Meadows & Pasture Land		Light Residential		
			Area (sf)	%	Area (sf)	%	
ON1	6.54	284,746	257,005	90	27,741	10	0.26
ON2	2.98	129,980	97,988	75	31,992	25	0.27

Pre-Development Condition

Time of Concentration (Trial calculations shown only for Pre-Development Condition subbasin ON1)

Subbasin Data				Overland Sheet Flow, T_{osf}[3] $T_{osf} = 0.93(L^{0.6}n^{0.6})/(i^{0.4}S^{0.3})$					Shallow Concentrated Flow, T_{scf}[4] $T_{scf}=L/(60V)$			Final		Note
Subbasin	Rainfall intensity, i[1] 25-yr (in/hr)	Rainfall intensity, i[1] 100-yr (in/hr)	Manning's Roughness Coefficient, n[2]	Length (ft)	Δ Elev. (ft)	Slope (ft/ft)	T_{osf} (min)	T_{osf} (min)	Length (ft)	Velocity[5] (fps)	T_{scf} (min)	T_c 25-yr (min)	T_c 100-yr (min)	
ON1	1.38	-	0.15	100	5.5	0.055	10	-	643	0.755	14	24	-	Trial 1, Assuming T_{osf} = 24 min for 25-yr
ON1	2.05	-	0.15	100	5.5	0.055	8	-	643	0.755	14	22	-	Trial 2, Assuming T_{osf} = 10 min for 25-yr
ON1	2.24	2.95	0.15	100	5.5	0.055	8	7	643	0.755	14	22	21	Trial 3, Assuming T_{osf} = 8 min for 25-yr
ON2	1.90	2.50	0.15	100	2	0.020	12	11	350	0.755	8	20	19	

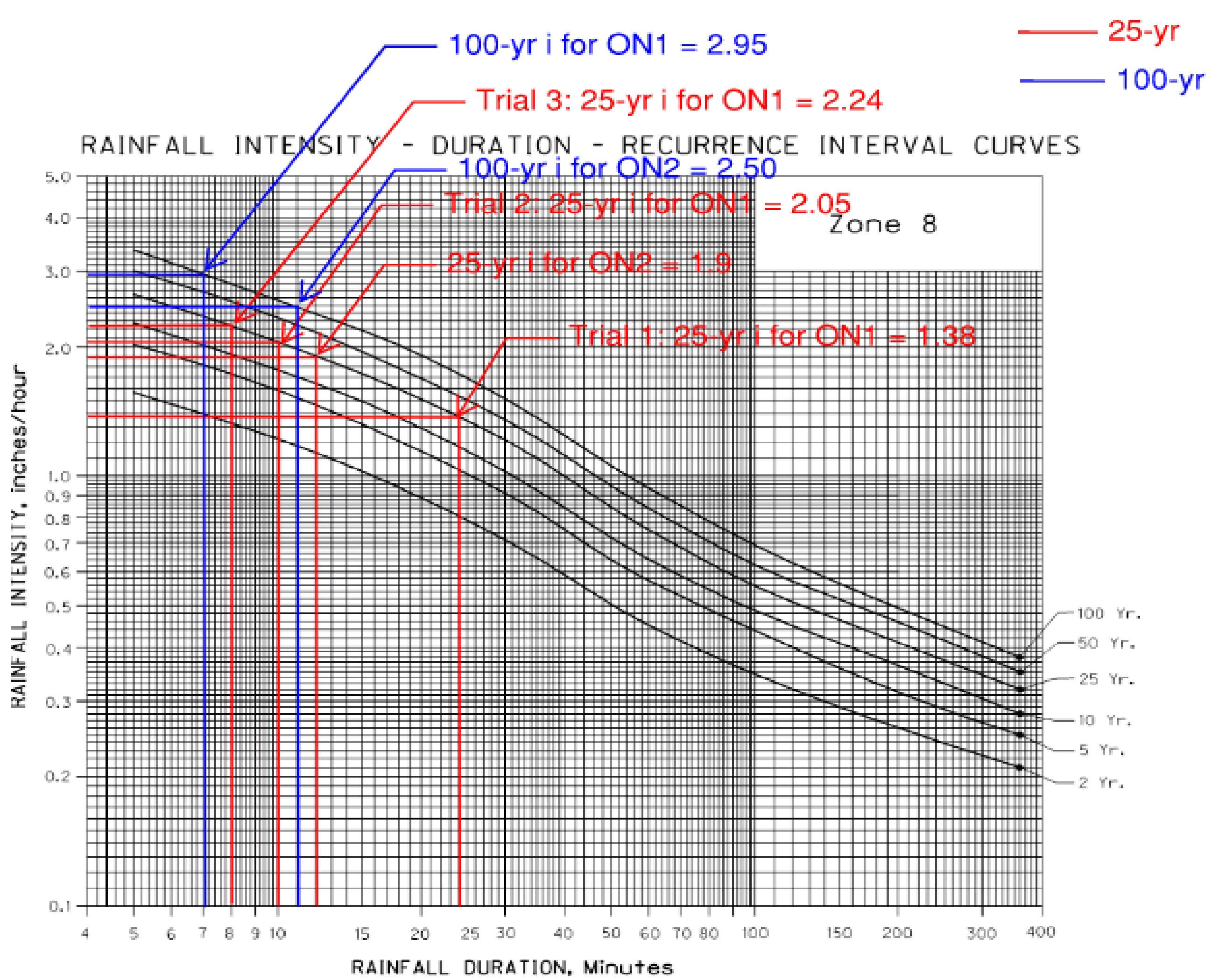
[1] ODOT Hydraulics Manual, Hydrology Appendix A, I-D-R Curve Zone 8

[2] ODOT Hydraulics Manual, Hydrology Appendix F, Table 3

[3] ODOT Hydraulics Manual, Hydrology Appendix F, Equation 4

[4] ODOT Hydraulics Manual, Hydrology Appendix F, Equation 5

[5] ODOT Hydraulics Manual, Hydrology Appendix F, Figure 1



Pre-Development Condition

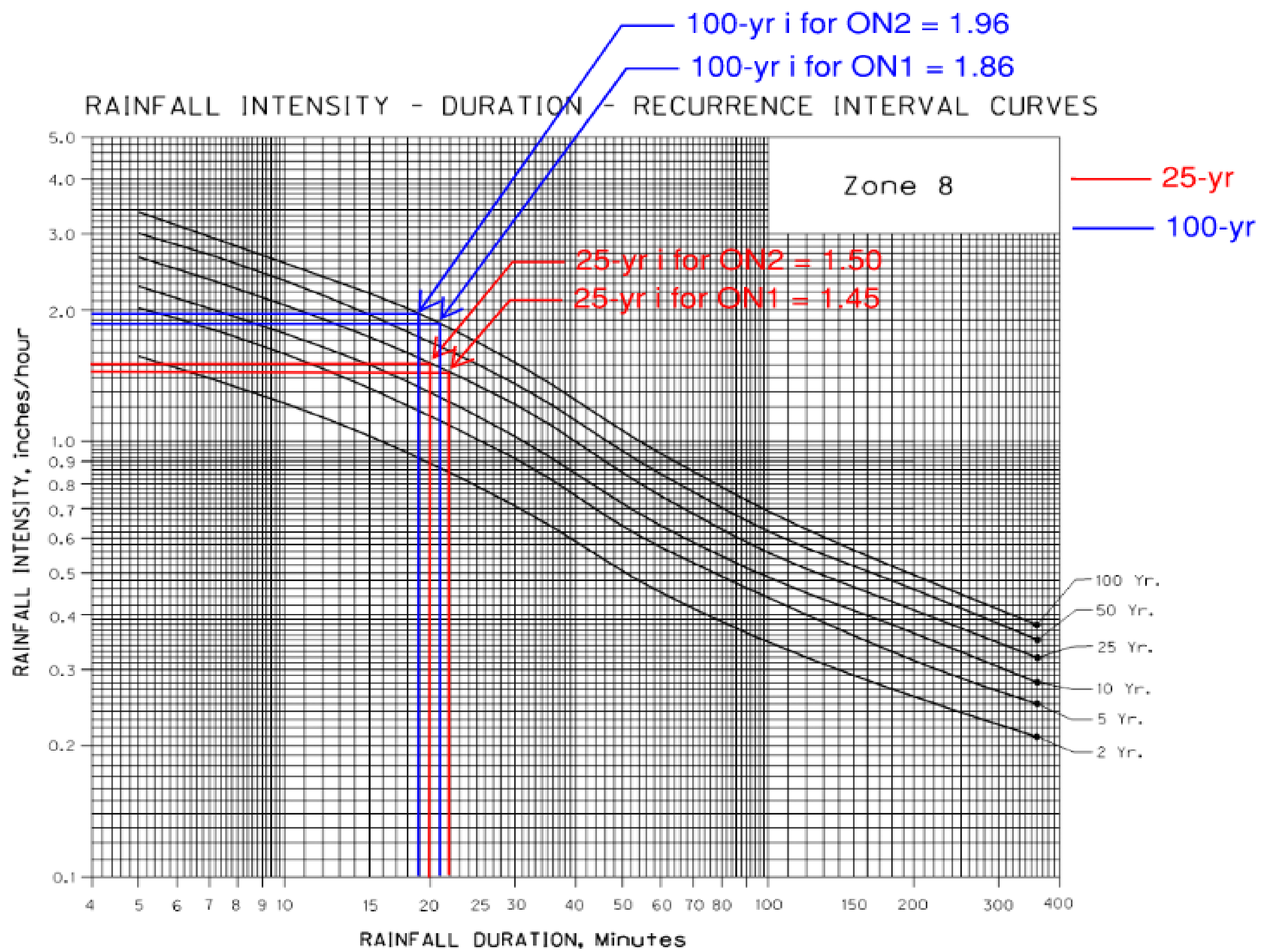
Hydrologic Analysis

Subbasin	Area, A (acres)	Time of Concentration T_c 25-yr (min)	Time of Concentration T_c 100-yr (min)	Runoff Coefficients C_f[1] 25-yr	C_f[1] 100-yr	C	Rainfall Intensity[2] i 25-yr (in/hr)	i 100-yr (in/hr)	Peak Discharge Q[3] 25-yr (cfs)	Q[3] 100-yr (cfs)
ON1	6.54	22	21	1.10	1.25	0.26	1.45	1.86	2.7	3.9
ON2	2.98	20	19	1.10	1.25	0.27	1.50	1.96	1.4	2.0

[1] ODOT Hydraulics Manual, Hydrology Appendix F, Table 2

[2] ODOT Hydraulics Manual, Hydrology Appendix A, I-D-R Curve Zone 8

[3] ODOT Hydraulics Manual, Hydrology Appendix F, Equation 1



A.2.8 Post-Development Condition Calculations

Post-Development Condition

Runoff Coefficients

Landuse	Runoff Coefficient[1]
	C
Meadows & Pasture Land	0.25
Industrial Areas, Heavy	0.60

[1]ODOT Hydraulics Manual, Hydrology Appendix F, Table 1

Composite Runoff Coefficients

Subbasin	Total Area (acres)	Total Area (sf)	Land Use Area per Subbasin				Total Weighted Runoff Coefficients
			Meadows & Pasture Land		Industrial Areas, Heavy		
			Area (sf)	%	Area (sf)	%	
ON1A	1.99	86,617	86,617	100	0	0	0.25
ON1B	2.75	119,593	36,948	31	82,646	69	0.49
ON2	4.79	208,515	92,398	44	116,117	56	0.44

Post-Development Condition

Time of Concentration

Subbasin Data				Overland Sheet Flow, T_{osf}[3] $T_{osf} = 0.93(L^{0.6}n^{0.6})/(i^{0.4}S^{0.3})$					Shallow Concentrated Flow, T_{scf}[4] $T_{scf}=L/(60V)$			Final	
Subbasin	Rainfall intensity, i[1] 25-yr (in/hr)	Rainfall intensity, i[1] 100-yr (in/hr)	Manning's Roughness Coefficient, n[2]	Length (ft)	Δ Elev. (ft)	Slope (ft/ft)	T_{osf} (min)	T_{osf} (min)	Length (ft)	Velocity[5] (fps)	T_{scf} (min)	T_c 25-yr (min)	T_c 100-yr (min)
ON1A	2.24	2.95	0.15	100	5.5	0.055	8	7	606	0.755	13	21	20
ON1B	2.35	3.12	0.05	100	1.1	0.011	7	6	254	2.100	2	9	8
ON2	2.35	3.12	0.05	100	1.0	0.010	7	6	437	2.100	3	10	9

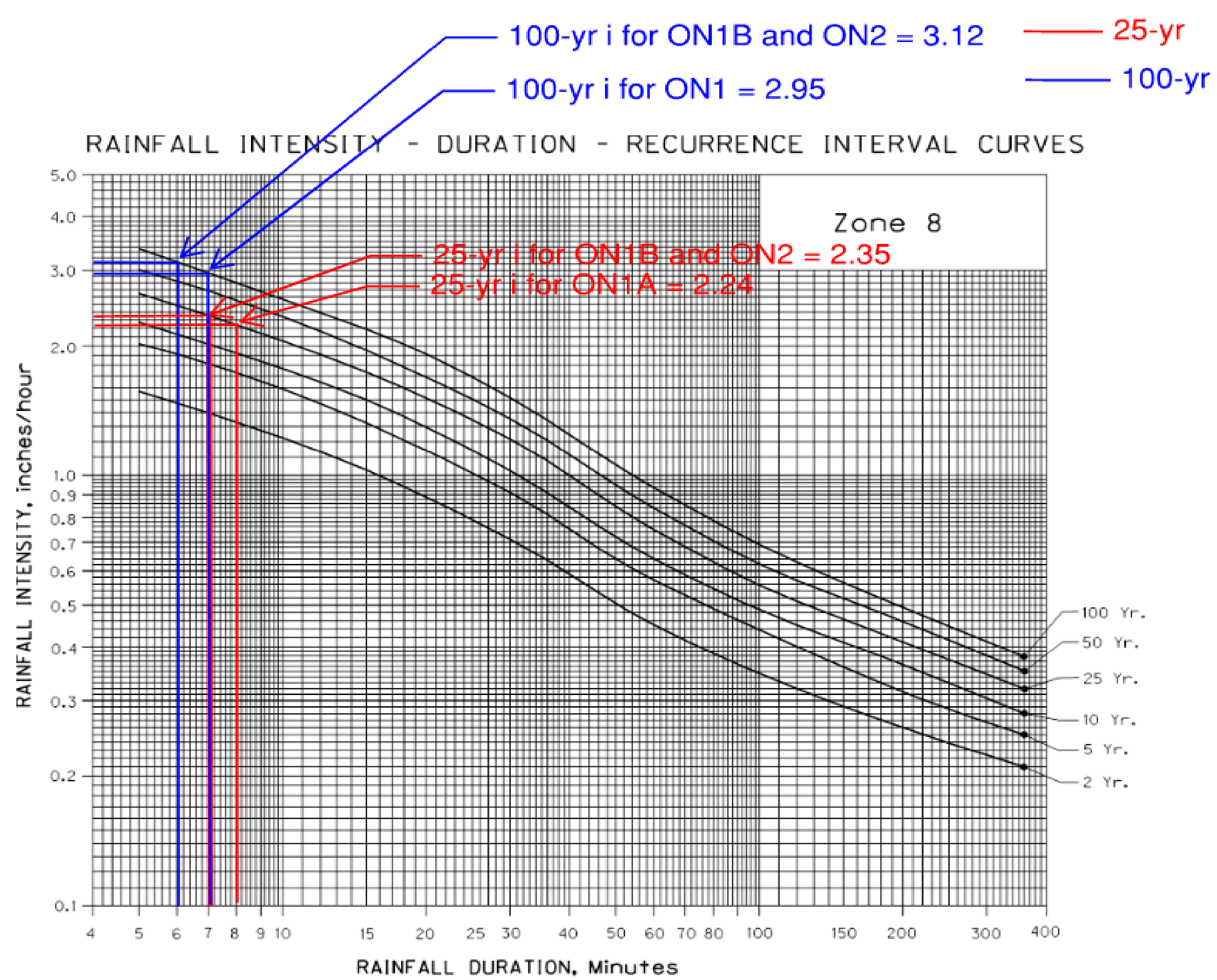
[1] ODOT Hydraulics Manual, Hydrology Appendix A, I-D-R Curve Zone 8

[2] ODOT Hydraulics Manual, Hydrology Appendix F, Table 3

[3] ODOT Hydraulics Manual, Hydrology Appendix F, Equation 4

[4] ODOT Hydraulics Manual, Hydrology Appendix F, Equation 5

[5] ODOT Hydraulics Manual, Hydrology Appendix F, Figure 1



Post-Development Condition

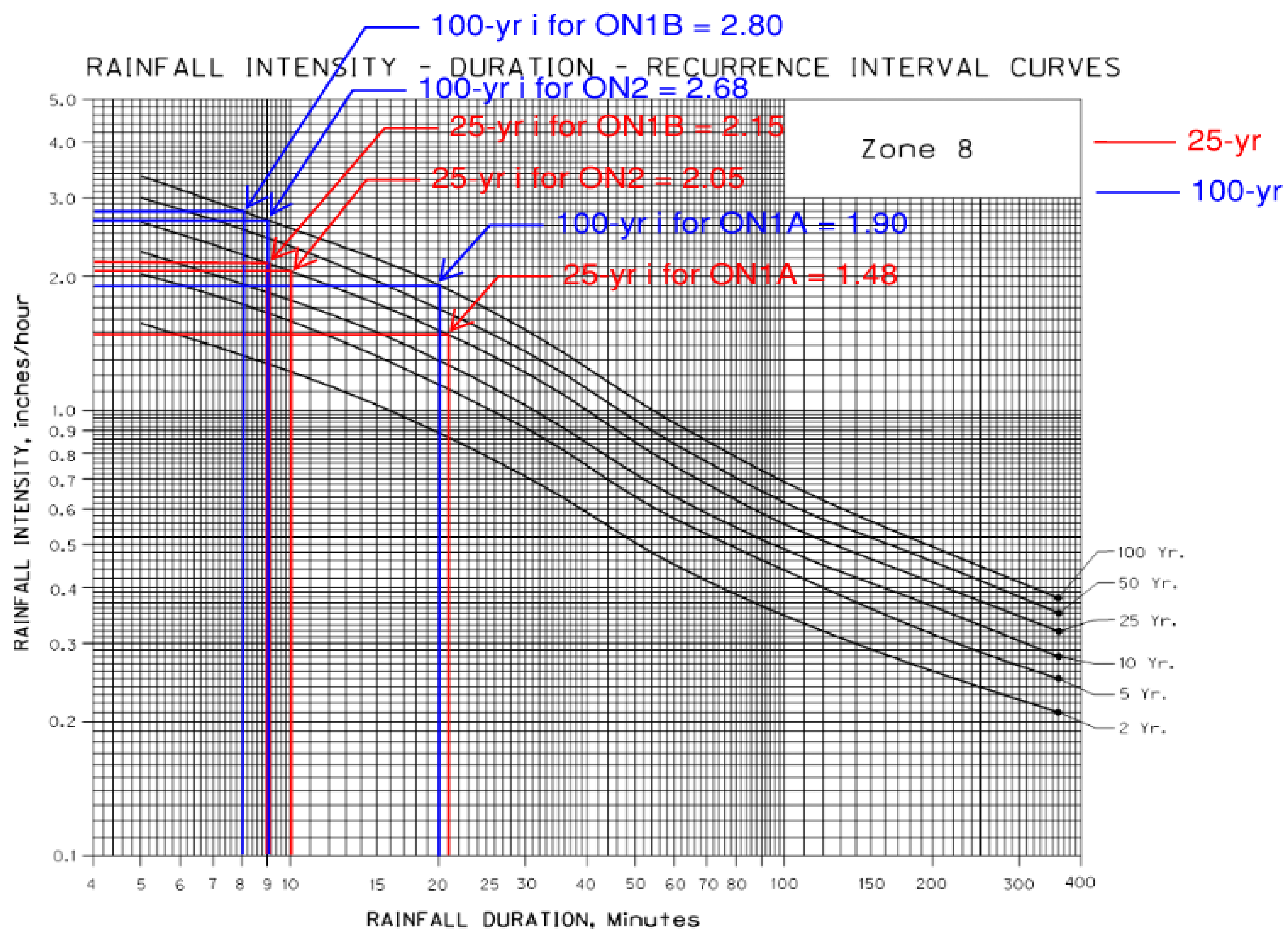
Hydrologic Analysis

				Runoff Coefficients			Rainfall Intensity[2]		Peak Discharge	
Subbasin	Area, A (acres)	Time of Concentration T_c 25-yr (min)	Time of Concentration T_c 100-yr (min)	C_f[1] 25-yr	C_f[1] 100-yr	C	i 25-yr (in/hr)	i 100-yr (in/hr)	Q[3] 25-yr (cfs)	Q[3] 100-yr (cfs)
ON1A	1.99	21	20	1.10	1.25	0.25	1.48	1.90	0.8	1.2
ON1B	2.75	9	8	1.10	1.25	0.49	2.15	2.80	3.2	4.7
ON2	4.79	10	9	1.10	1.25	0.44	2.05	2.68	4.8	7.1

[1] ODOT Hydraulics Manual, Hydrology Appendix F, Table 2

[2] ODOT Hydraulics Manual, Hydrology Appendix A, I-D-R Curve Zone 8

[3] ODOT Hydraulics Manual, Hydrology Appendix F, Equation 1



APPENDIX B

APPENDIX B – HYDRAULIC DESIGN AND ANALYSIS

- B.1 DETENTION BASIN CALCULATIONS
- B.2 ODOT HDM CULVERT PERFORMANCE CHART
- B.3 FLOWMASTER CALCULATIONS

wsp

B.1 DETENTION BASIN CALCULATIONS

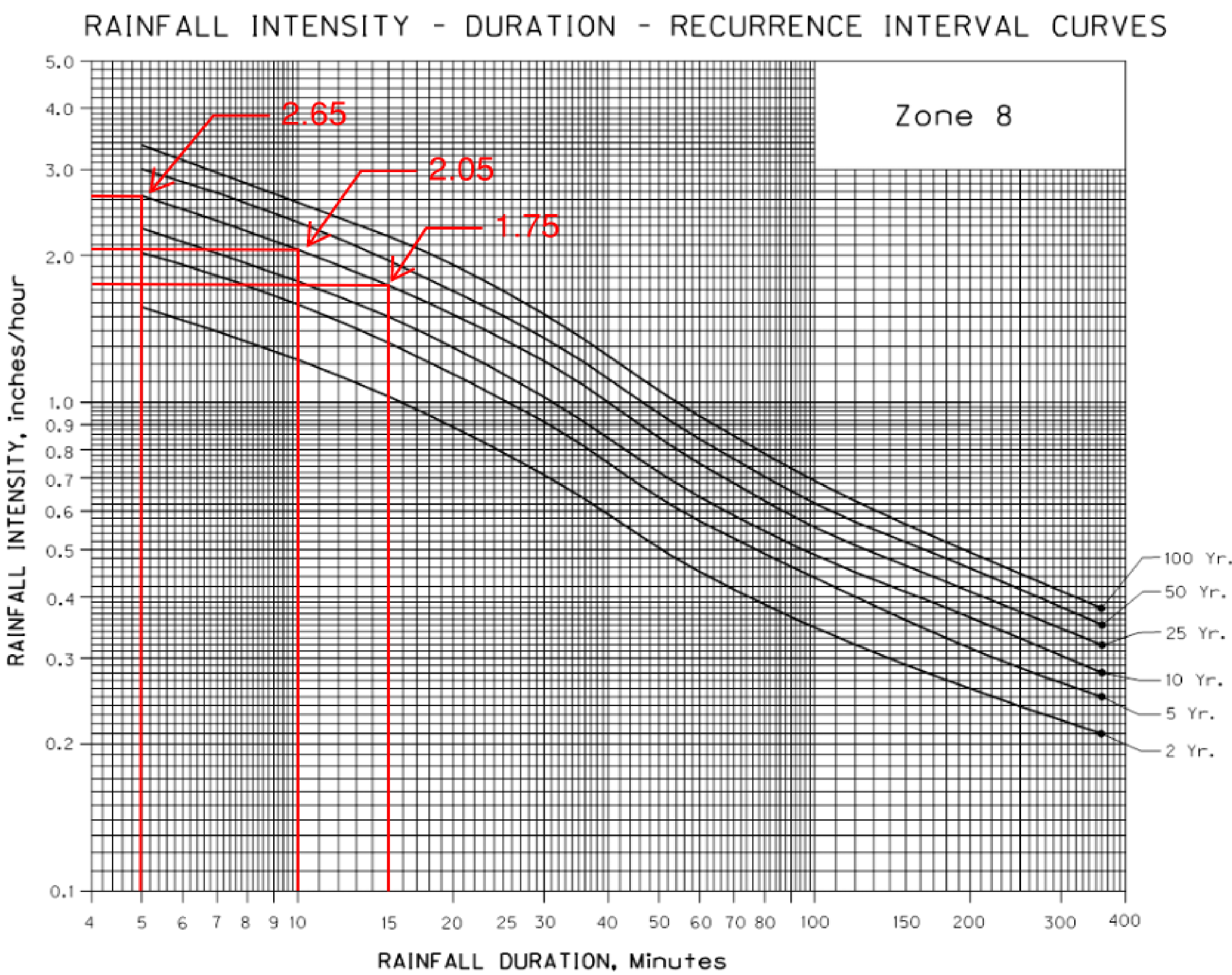
- B.1.1 STORAGE VOLUME CALCULATIONS
- B.1.2 OUTLET STRUCTURE CALCULATIONS

B.1.1 Storage Volume Calculations

Storage Volume Tabulation for Simplified Rational Method for SW Detention Basin receiving runoff from ON1B

1	2	3	4	5	6	7	8	
Time (T)	Impervious Area (CA)	Rainfall Intensity (i)	Inflow Rate (Q_i)	Inflow Volume (V_i)	Outflow Rate (Q_o)*	Outflow Volume (V_o)	Required Storage (V_S)	Note
Minute	Acre	in/hr	cfs	Cubic feet	cfs	Cubic feet	Cubic feet	
5	1.35	2.65	3.9	1181	1.9	570	611	
10	1.35	2.05	3.0	1827	1.9	1139	**688**	Max required storage
15	1.35	1.75	2.6	2340	1.9	1709	631	

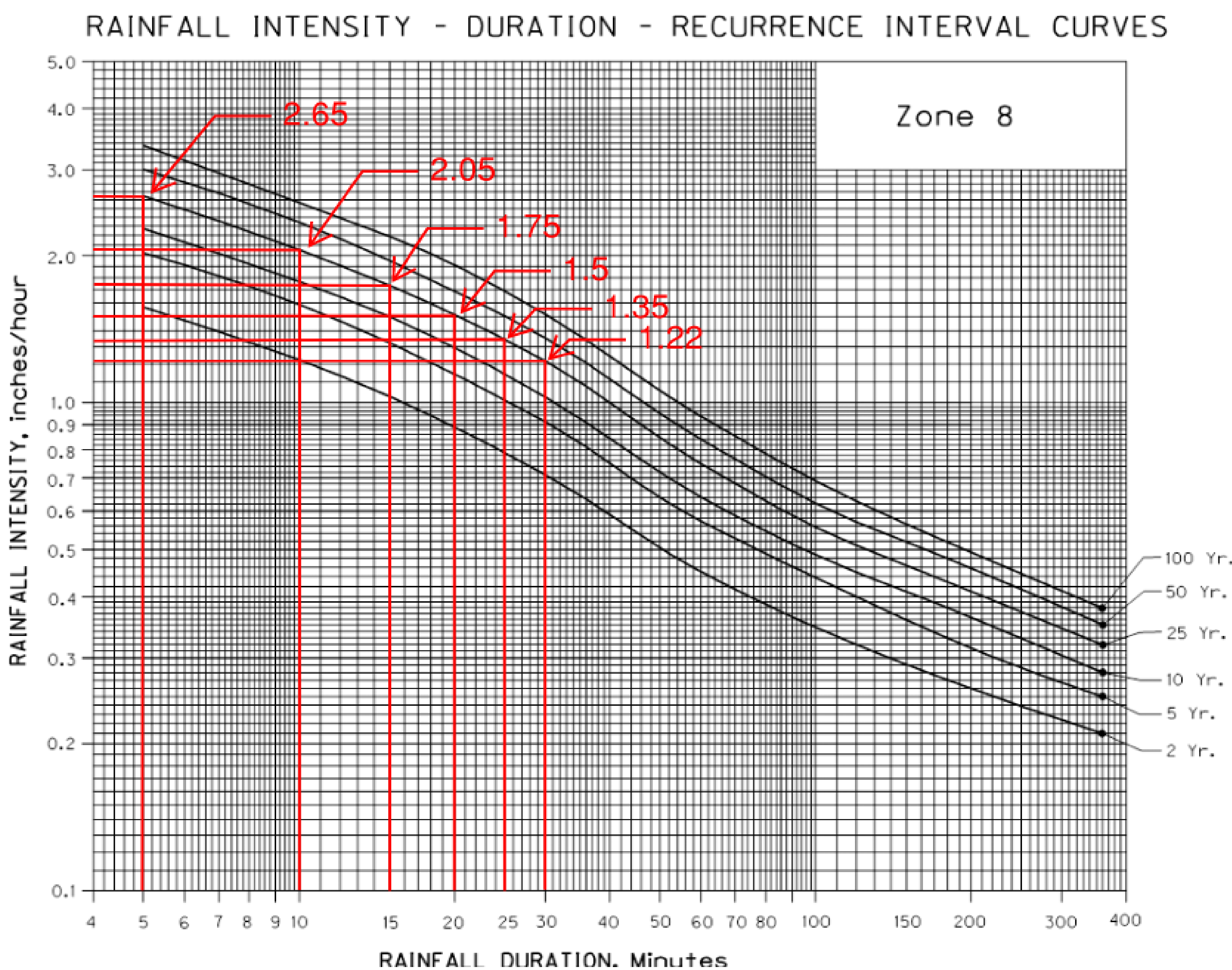
*Pre-Development ON1 25-Year Q minus Post-Development ON1A 25-Year Q = 2.7 cfs - 0.8 cfs = 1.9 cfs



Storage Volume Tabulation using Simplified Rational Method for SE Detention Basin receiving runoff from ON2

1	2	3	4	5	6	7	8	
Time (T)	Impervious Area (CA)	Rainfall Intensity (i)	Inflow Rate (Q_i)	Inflow Volume (V_i)	Outflow Rate (Q_o)*	Outflow Volume (V_o)	Required Storage (V_S)	Note
Minute	Acre	in/hr	cfs	Cubic feet	cfs	Cubic feet	Cubic feet	
5	2.13	2.65	6.2	1862	1.4	406	1457	
10	2.13	2.05	4.8	2881	1.4	811	2070	
15	2.13	1.75	4.1	3690	1.4	1217	2473	
20	2.13	1.5	3.5	4217	1.4	1622	2594	
25	2.13	1.35	3.2	4744	1.4	2028	**2716**	Max required storage
30	2.13	1.22	2.9	5145	1.4	2434	2711	

*Pre-Development 25-Year Q = 1.4 cfs



B.1.2 Outlet Structure Calculations

Primary outlet control structure for SW Detention Basin

Parameter	Value	Unit
Discharge (Q)	1.9	cfs
Effective head (H_o)	**2**	**ft**
Diameter (D)	0.60	ft
Diameter (D)	**7.15**	**in**

Auxillary Riser for SW Detention Basin

Parameter	Value	Unit
Discharge (Q)	4.7	cfs
Riser Diameter (D)[1]	**18**	**in**
Sharp Crested Weir Length (L)[2]	4.71	ft
Head (H)	**0.45**	**ft**

[1] Based on Figure 12-35, for flow of 5 cfs and head of 0.5 ft, risers with diameter from 18 inch or larger will provide weir flow

[2] From Table 12-3

Primary outlet control structure for SE Detention Basin

Parameter	Value	Unit
Discharge (Q)	1.4	cfs
Effective head (H_o)	**0.5**	**ft**
Diameter (D)	0.71	ft
Diameter (D)	**8.53**	**in**

Auxillary Riser for SW Detention Basin

Parameter	Value	Unit
Discharge (Q)	7.1	cfs
Riser Diameter (D)[1]	**24**	**in**
Sharp Crested Weir Length (L)[2]	6.28	ft
Head (H)	**0.49**	**ft**

[1] Based on Figure 12-35, for flow of 7 cfs and head of 0.5 ft, risers with diameter from 24 inch or larger will provide weir flow

[2] From Table 12-3

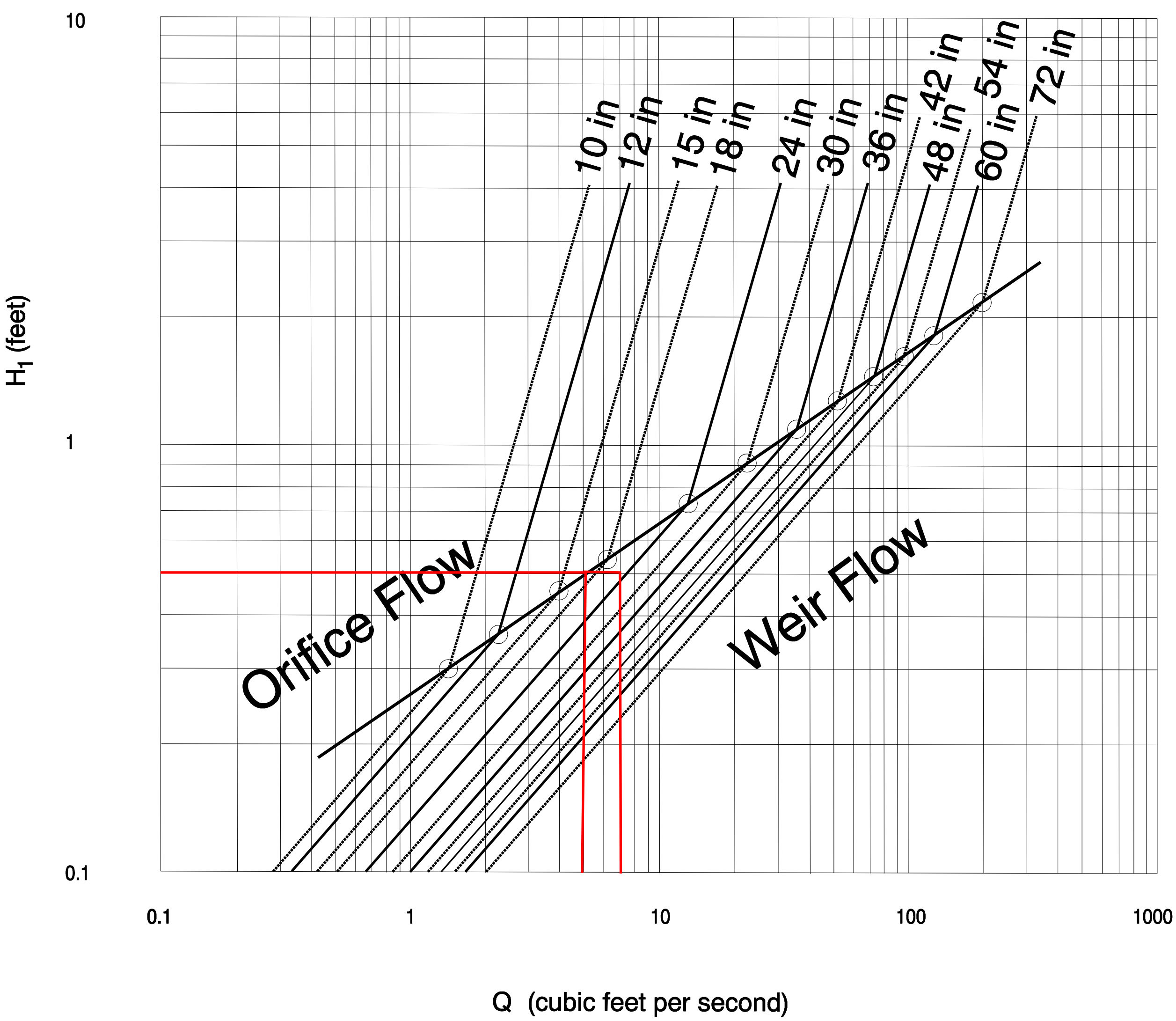


Figure 12-35 Riser Overflow Hydraulics

Table 12-3 Weir Lengths for Pipe Risers

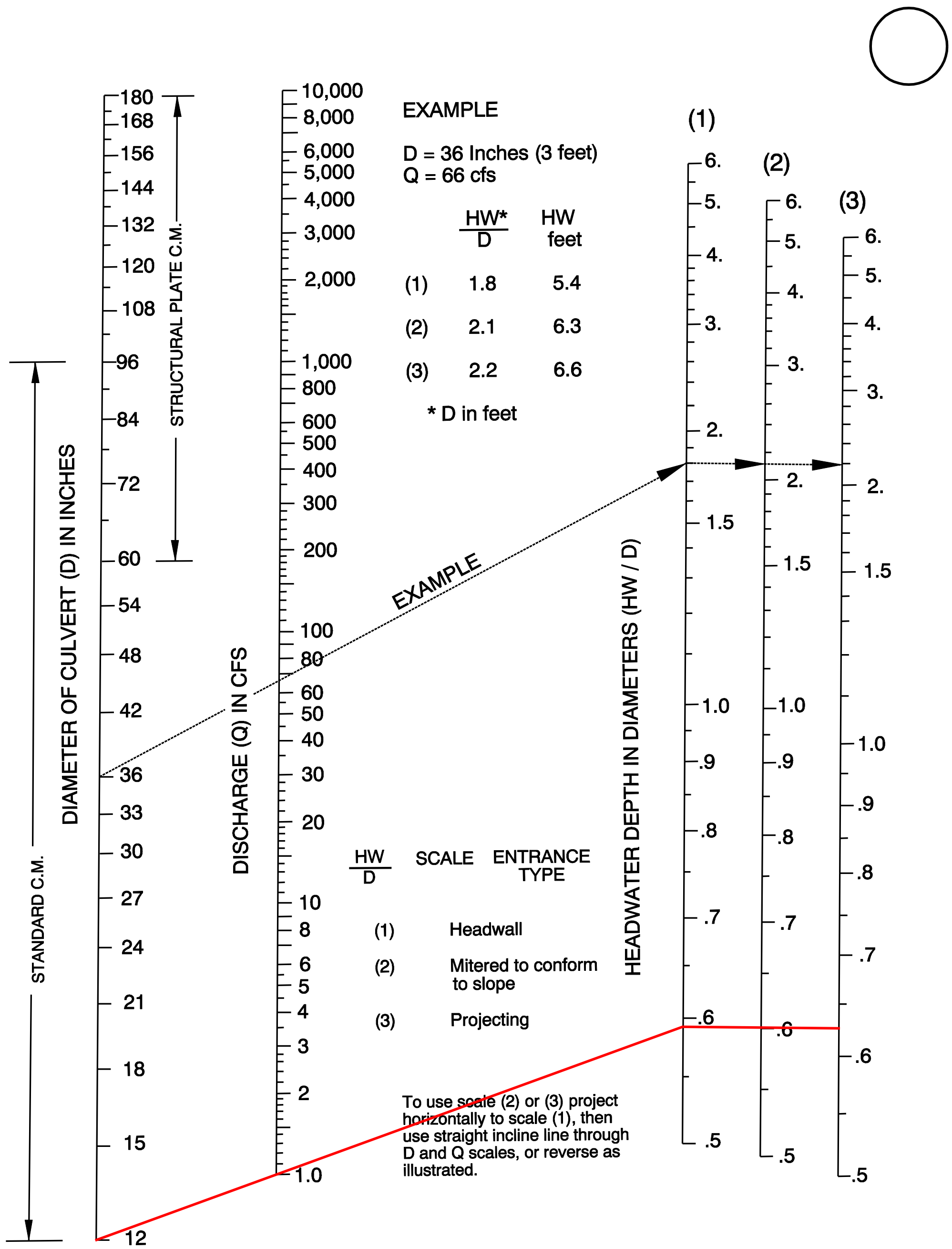
Pipe Diameter, D (inches)	Weir Length, L = πD (feet)
10	2.62
12	3.14
18	4.71
24	6.28
30	7.85
36	9.42
42	11.00
48	12.57
54	14.14
60	15.71
72	18.85

Risers operating in orifice flow may be subject to a phenomenon called vortex flow. This is a circular spiraling of flow immediately above the submerged riser and it can reduce the flow through an orifice by as much as 75 percent. Vortex flow can be prevented by the addition of an anti-vortex plate, as shown in Figure 12-36. Another method of preventing vortex flow is to increase the size of the riser to ensure that weir flow is predominant during the check storm and orifice flow does not occur.

B.2 ODOT HDM Culvert Performance Chart

CHART 3

HEADWATER DEPTH FOR C.M. CULVERTS WITH INLET CONTROL



B.3 FlowMaster Calculations

Normal Depth - North

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.030
Channel Slope	0.005 ft/ft
Left Side Slope	3.000 H:V
Right Side Slope	3.000 H:V
Bottom Width	40.00 ft
Discharge	1.20 cfs

Results

Normal Depth	0.1 ft
Flow Area	2.3 ft²
Wetted Perimeter	40.4 ft
Hydraulic Radius	0.1 ft
Top Width	40.34 ft
Critical Depth	0.0 ft
Critical Slope	0.042 ft/ft
Velocity	0.53 ft/s
Velocity Head	0.00 ft
Specific Energy	0.06 ft
Froude Number	0.396
Flow Type	Subcritical

GVF Input Data

Upstream Depth	0.0 ft
Length	0.0 ft
Number Of Steps	0

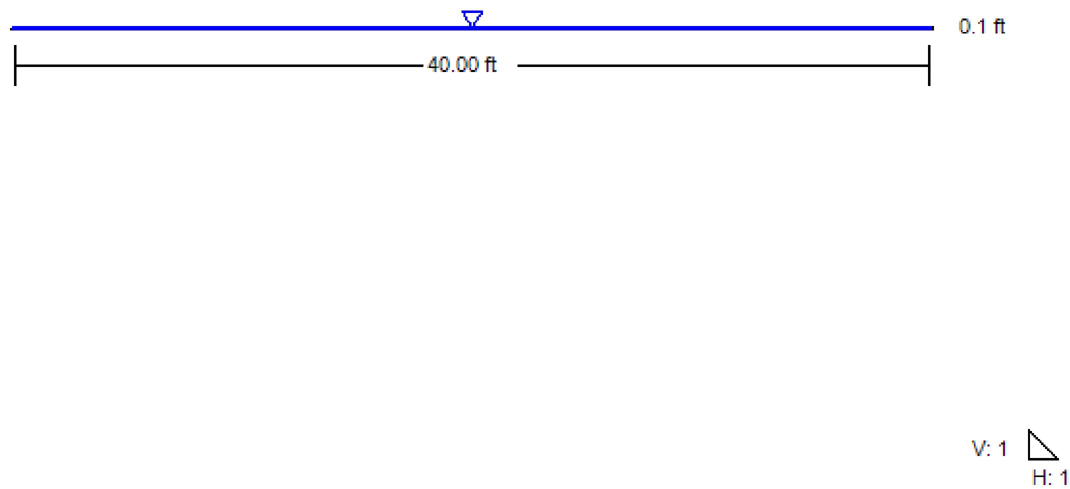
GVF Output Data

Downstream Depth	0.0 ft
Profile Description	N/A
Profile Headloss	0.00 ft
Downstream Velocity	Infinity ft/s
Upstream Velocity	Infinity ft/s
Normal Depth	0.1 ft
Critical Depth	0.0 ft
Channel Slope	0.005 ft/ft
Critical Slope	0.042 ft/ft

Cross Section for Normal Depth - North

Project Description	
Friction Method	Manning Formula
Solve For	Normal Depth

Input Data	
Roughness Coefficient	0.030
Channel Slope	0.005 ft/ft
Normal Depth	0.1 ft
Left Side Slope	3.000 H:V
Right Side Slope	3.000 H:V
Bottom Width	40.00 ft
Discharge	1.20 cfs



Normal Depth - East

Project Description

Friction Method	Manning Formula
Solve For	Normal Depth

Input Data

Roughness Coefficient	0.030
Channel Slope	0.008 ft/ft
Left Side Slope	3.000 H:V
Right Side Slope	3.000 H:V
Bottom Width	40.00 ft
Discharge	7.10 cfs

Results

Normal Depth	0.1 ft
Flow Area	5.9 ft²
Wetted Perimeter	40.9 ft
Hydraulic Radius	0.1 ft
Top Width	40.88 ft
Critical Depth	0.1 ft
Critical Slope	0.028 ft/ft
Velocity	1.20 ft/s
Velocity Head	0.02 ft
Specific Energy	0.17 ft
Froude Number	0.556
Flow Type	Subcritical

GVF Input Data

Downstream Depth	0.0 ft
Length	0.0 ft
Number Of Steps	0

GVF Output Data

Upstream Depth	0.0 ft
Profile Description	N/A
Profile Headloss	0.00 ft
Downstream Velocity	Infinity ft/s
Upstream Velocity	Infinity ft/s
Normal Depth	0.1 ft
Critical Depth	0.1 ft
Channel Slope	0.008 ft/ft
Critical Slope	0.028 ft/ft

Cross Section for Normal Depth - East

Project Description	
Friction Method	Manning Formula
Solve For	Normal Depth

Input Data	
Roughness Coefficient	0.030
Channel Slope	0.008 ft/ft
Normal Depth	0.1 ft
Left Side Slope	3.000 H:V
Right Side Slope	3.000 H:V
Bottom Width	40.00 ft
Discharge	7.10 cfs