CIRCULAR DEQ-8

MONTANA STANDARDS FOR

SUBDIVISION STORM WATER DRAINAGE

2017 Edition

FOREWORD

These standards, based on demonstrated technology, set forth the requirements for the design and preparation of plans and specifications of storm water drainage systems in subdivisions in the State of Montana.

Users of these standards need to be aware that some storm water drainage systems are considered by the Environmental Protection Agency (EPA) to be Class V injection wells and may require associated permits.

These standards replace Department of Environmental Quality (DEQ) Circular DEQ-8, 2002 Edition.

TABLE OF CONTENTS

LIST OF TABLES AND FIGURES

1. INTRODUCTION

1.1 APPLICABILITY

Storm water is an all-inclusive term that refers to any of the water running off of the land's surface after a rainfall event. Prior to development, storm water is a small component of the annual water balance. However, as development increases, the elimination of pervious surfaces (that is, surfaces able to soak water into the ground) with construction of new roads, driveways, and rooftops means less water soaks into the ground and more water runs off. In an undeveloped area, the majority of rainfall infiltrates the soil and subsequently percolates deeper into groundwater or is evapotranspired back to the atmosphere. As development occurs and the percentage of impervious area increases, an increasing amount of rainfall runs off.

This document contains standards and technical procedures applicable to storm water drainage plans and related designs to ensure proper drainage ways in subdivisions in Montana. The standards were developed by the Montana Department of Environmental Quality (Department) in compliance with Section 76-4-104, MCA, of the Sanitation in Subdivisions Act, and ARM 17.36.310.

These minimum standards apply to all storm water drainage plans for subdivisions in Montana. In some cases, a reviewing authority (other than the Department) may have requirements that are more stringent than those set forth in this Circular. In these standards, however, the term "reviewing authority" means the Department or a local department or board of health certified to conduct a review, as defined in Section 76-4-102(12), MCA.

In addition to review under this document, the Department issues permits for storm water discharges associated with industrial activities, small municipal separate storm sewer systems (MS4s), and storm water discharges associated with construction activities. For those activities subject to departmental permitting, a separate application must be made to the Department for review.

1.2 DEFINITIONS

1.2.1. **Class V Injection Well** means a well that is used to inject non-hazardous fluids into or above underground sources of drinking water.

1.2.2. **Conveyance** means the transport of storm water from one point to another.

1.2.3. **Conveyance Structures** means facilities used to convey storm water that include, but are not limited to, ditches, pipes, and channels.

1.2.4. **Culvert** means a closed conduit to convey surface water under a roadway, railroad, or other impediment.

1.2.5. **Detention Facility** means an area or structure where excess storm water is stored or held temporarily and then drains through a designed outlet. Compare to "retention facility."

1.2.6. **Discharge** means the amount of flow, in volume per unit time, from any structure that is used for collecting and conveying storm water, often expressed in units of cubic feet per second or acre-inches per hour.

1.2.7. **Duration** means the length of time over which a storm event occurs (e.g., one hour, 24 hours, etc.).

1.2.8. **Frequency** means the rate of recurrence of a storm event, usually expressed in years.

1.2.9. **Flow rate** means a volume, or quantity, of water conveyed over a specified unit of time, often expressed in units of cubic feet per second or acre-inches per hour.

1.2.10. **Hydrograph** means a graphical representation of the time distribution of runoff from a watershed.

1.2.11. **Intensity-Duration-Frequency (IDF) Curve** means a graphical representation of the relationship between rainfall or rainfall intensity and duration for different frequencies.

1.2.12. **Impervious area** means a hard surface area that prevents or retards the entry of water into the soil. Impervious areas include, but are not limited to, rooftops, traditional asphalt, concrete and gravel parking lots, driveways, roads, and sidewalks.

1.2.13. **Infiltration Facility** means a structure or feature that captures and temporarily stores storm water runoff so that it may permeate over time into underlying or surrounding soils.

1.2.14. **Initial Storm Water Facility** means an area or structure sized to capture and infiltrate or evapotranspire the volume of storm water runoff generated from the first 0.5 inches of rainfall on impervious areas.

1.2.15. **Landscaping** means grass, foliage, shrubbery, and/or trees.

1.2.16. **MS4** means municipal separate storm sewer systems.

1.2.17. **Offsite Basin** means any storm water basin located outside the subdivision boundaries.

1.2.18. **Onsite Basin** means any storm water basin located within the subdivision boundaries.

1.2.19. **Overtopping Roadways or Driveways** means covering a road or driveway with storm water.

1.2.20. **Peak Flow** means the maximum rate of storm water flow passing a given point during or after a storm event.

1.2.21. **Pre-treatment Facility** means a structure that improves storm water quality by reducing sediment, trash, debris, or organic materials. The term does not apply to the pre-treatment standards promulgated by the EPA and set forth in 40 C.F.R. Part 403 and 40 C.F.R. chapter 1, subchapter N.

1.2.22. **Post-development** refers to the conditions of the site after construction of the proposed development.

1.2.23. **Rainfall Intensity** means the rainfall rate for a duration.

1.2.24. **Retention Facility** means an area or structure where excess storm water is stored or held and is not discharged. Compare to “detention facility.”

1.2.25. **Runoff** means that portion of the rainfall on a drainage area that discharges from the land’s surface after a storm event.

1.2.26. **Runoff Coefficient** means a representation of the effect that different surface areas have on storm water runoff, expressed as a unitless number between zero and one.

1.2.27. **Shallow Flow** means a continuous film of overland flow that is concentrated into surface features such as rills, rivulets, or channels, usually developed after runoff flows for approximately 300 feet.

1.2.28. **Sheet Flow** means a thin continuous film of overland flow that is not concentrated into surface features such as rills, rivulets, or channels.

1.2.29. **State Waters** is defined in 75-5-103, MCA.

1.2.30. **Storm Sewer** means a network of pipes that conveys surface drainage to an outfall from an inlet or through a manhole.

1.2.31. **Storm Water** means water that originates during a storm event. Storm water can infiltrate, evaporate, or runoff. Used interchangeably with the term “storm water drainage.”

1.2.32. **Time of Concentration** means the amount of time it takes storm water runoff to travel from the most distant point on a site or drainage basin to a specific point of interest (e.g., a conveyance structure, a retention or detention pond, etc.).

1.2.33. **Undeveloped Area or Condition** means land without improvements and without other changes that would increase storm water flow.

1.2.34. **Volume** means the amount of storm water runoff, often expressed in units of cubic feet.

1.2.35. **Watershed** means an area of land upon which runoff flows to the outlet or point of interest during a storm event.

1.2.36. **Wetlands** are areas inundated or saturated by ground or surface water sufficient to support, and that under normal circumstances do support, a prevalence of vegetation typically adapted for life in saturated soil conditions. Wetlands generally include swamps, marshes, bogs, and similar areas. *See* 33 C.F.R. § 328.3(4).

2. SUBMISSION OF PLANS

2.1 GENERAL

Applications for review must be submitted to the Montana Department of Environmental Quality (Department) or a delegated division of local government. No approval may be issued until all required information has been submitted to the reviewing authority and found to be satisfactory.

Applications must show that development does not adversely impact the surrounding area, the environment, or the health and safety of the lot owner.

Applications must include either a Standard or a Simplified Plan as described in Chapter 3.

Applicants shall submit one copy of the engineering report and four copies of drawings, specifications, and the operation and maintenance plan.

2.2 REPORT

A copy of a storm water drainage report must be included with each application and must include the following:

A. The name of the subdivision;

B. For simplified plans only, qualifying criteria for a simplified plan in accordance subchapter 3.2;

C. Narrative describing site information:

   1. Slope and direction across project;

   2. Vegetation patterns including wetlands, forestland, grasses, pasture, range, sage areas, etc.;

   3. Hydrologic patterns such as sheet flow, shallow flow, channel flow, etc. and features such as natural drainages and depressions; and

   4. Surrounding land use such as residential, agricultural, industrial, etc.;

D. A description of the Initial Storm Water Facility, as defined in subchapter 3.4;

E. A description of all storm water drainage facilities, including those for retention, detention, conveyance, infiltration, and pre-treatment;

F. For standard plans only, a description of methods used to convey all offsite runoff and onsite runoff flowing through the development, to ensure no roads, driveways, or other access points are overtopped during the 10-year storm event; and that no drainfields or buildings will be inundated during the 100-year storm event; and

G. Calculations supporting facility size and design.

2.3 DRAWINGS

Four sets of storm water drainage drawings must be submitted and must include the following:

A. The name of the subdivision;

B. A north arrow and scale;

C. The name and affiliation of the person who prepared the plan;

D. An identifier or number for each lot;

E. The area of each lot;

F. Locations of existing and proposed easements;

G. Locations of existing and proposed roads, driveways, buildings, wells, drainfields, and utilities;

H. Locations, sizes, and design details of existing and proposed storm water structures;

I. Locations of drainage ways;

J. Floodplains as delineated by FEMA or local floodplain authorities;

K. Direction of drainage flow across the site, along each road, and at each intersection;

L. Location and construction details of any proposed detention facilities, retention facilities, infiltration facilities, and erosion control and conveyance structures; and

M. Profile sheets of proposed conveyance structures may be required for complex designs.

2.4 CONSTRUCTION DOCUMENTS

Four sets of complete, detailed, technical specifications may be required for the construction of complex storm water drainage facilities.

2.5 OPERATION AND MAINTENANCE PLANS

Four sets of an operation and maintenance plan must be submitted and must include:

A. Procedures for the long-term operation and maintenance of facilities;
B. Designation of the party responsible for the overall management and implementation of the operation and maintenance; and
C. Easement information as necessary to ensure continued access to storm water drainage facilities.

2.6 DEVIATIONS

The Department may grant a deviation from a requirement of this Circular. The terms **shall**, **must**, **may not**, and **require** indicate mandatory items, and applicants must obtain approval to deviate from these mandatory requirements. Other terms, such as **should**, **may**, **recommended**, and **preferred**, indicate desirable procedures or methods. These non-mandatory items serve as guidelines for designers and do not require approval for deviations.

A request for a deviation must be made in writing to the Department and must include the appropriate review fee. The request must identify the specific chapter or subchapter of the Circular to be considered. Adequate justification for the deviation must be provided. “Engineering judgment” or “professional opinion” without supporting data is not adequate justification. The justification must address the following issues:

A. The deviation would be unlikely to cause pollution of state waters in violation of 75-5-605, MCA;
B. The deviation would protect the quality and potability of water for public water supplies and domestic uses and would protect the quality of water for other beneficial uses, including those specified in 76-4-101, MCA; and
C. The deviation would not adversely affect public health, safety, or welfare.

The Department will review the request and make a final determination on whether a deviation may be granted.

2.7 ILLUSTRATIONS, SPREADSHEETS AND EXAMPLES

The images, pictures, examples, and spreadsheets found in this Circular are presented for illustration purposes only and may not include all design requirements. Please refer to the specific rules in this Circular pertaining to each element for details.

3. DESIGN CRITERIA

3.1 GENERAL

All storm water drainage designs must address pre-development and post-development site conditions. A Simplified Plan or a Standard Plan as described in this chapter must be submitted and must include an Initial Storm Water Facility sized to infiltrate, evapotranspire, and/or capture for reuse the post-development runoff generated from the first 0.5 inches of rainfall on impervious areas. The required volume of an Initial Storm Water Facility may be included in the design of any proposed retention, detention, or infiltration facility.

3.2 SIMPLIFIED PLAN

Simplified Plans maintain the same level of protection as Standard Plans but are appropriate for smaller, less-complicated developments. An example spreadsheet and design for Simplified Plans are provided in Appendices F and I.

A Simplified Plan may be used only if all of the following criteria are satisfied:

A. The impervious area within each proposed lot has a slope of three percent or less;
B. Impervious areas comprise less than or equal to 25 percent of the total acreage of each lot; and
C. The proposed subdivision will not alter historic runoff patterns outside the boundaries of the lot.

Simplified Plans may not increase the volume of runoff between lots or adjoining property as a result of development during the 100-year storm event.

3.3 STANDARD PLAN

Standard Plans are required if the development does not satisfy the criteria for Simplified Plans in Subchapter 3.2. An example spreadsheet and designs for Standard Plans are provided in Appendices G, J, K, and L.

Standard Plans must address storm water drainage peak flow and volume in accordance with Appendix B. Standard Plans must demonstrate that the proposed subdivision will not allow storm water to do any of the following:

A. Exceed the pre-development runoff to an adjoining property during the 2-year storm event;
B. Overtop roadways or driveways during a 10-year storm event; or
C. Inundate any buildings or drain fields during a 100-year storm event. This may be demonstrated through either narrative descriptions or calculations.

3.4 INITIAL STORM WATER FACILITY

Storm drainage designs must include an Initial Storm Water Facility sized to infiltrate, evapotranspire, and/or capture for reuse the post-development runoff generated from the first 0.5 inches of rainfall on impervious areas. The required volume of the Initial Storm Water Facility may be included in the design of any proposed retention, detention, or infiltration facility. The equation to calculate the minimum facility size for the 0.5-inch storm event is:

$$V = \frac{(0.5 * A_{imp})}{12\,\frac{inches}{ft}}$$

Where: V = minimum volume (ft^3)
A_{imp} = total impervious area (ft^2)

An example of calculations for an Initial Storm Water Facility is provided in Appendix H.

3.5 PRE- AND POST-DEVELOPMENT CONDITIONS

For areas without an existing approval under the Sanitation in Subdivisions Act, the pre-development runoff must be calculated based on undeveloped conditions. For rewrite applications under ARM 17.36.112, the pre-development runoff may be calculated based on the approved developed conditions.

Post-development runoff must be computed based on proposed developed conditions, including additional impervious area and any revisions to the conditions of the previous approval.

If the extent of impervious area is known, the proposed development conditions must be used in the storm water drainage design. Where the extent of impervious area is unknown, an assumed estimated area must be provided for review and approval.

3.6 RAINFALL INTENSITY

Rainfall intensity must be derived from the 24-hour storm duration. Rainfall information for a site can be determined from the following sources:

A. Hydrometeorological Design Studies Center's Precipitation Frequency Data Server (NOAA Atlas 2), available online at http://hdsc.nws.noaa.gov/hdsc /pfds/index.html;

B. Data for select sites in accordance with Appendix A. For sites not represented in Appendix A, use the value from the closest reported station;

C. An IDF curve at the time of concentration; or

D. Other sources approved by the reviewing authority.

3.7 ACCEPTABLE METHODS

Storm water volume and flow rates must be computed in accordance with Appendix B. Other methods may be used upon approval by the reviewing authority.

3.8 STORM WATER VOLUME

Storm water volumes that exceed the capacity of the Initial Storm Water Facility as described in this Chapter must be retained, detained, or infiltrated. Facilities must be sized in accordance with Appendix B.

3.8.1 SIMPLIFIED PLAN

Simplified Plans must include storm water volume calculations for the pre-development and post-development site conditions for each lot based on the 100-year storm event.

3.8.2 STANDARD PLAN

Standard Plans must include storm water volume calculations for the pre-development and post-development site conditions during a 2-year storm event.

3.9 PEAK FLOW

3.9.1 SIMPLIFIED PLAN

Simplified Plans may not alter historic runoff patterns outside the boundaries of the lot.

3.9.2 STANDARD PLAN

Standard Plans must include calculations of runoff peak flow for onsite and offsite drainage basins for the pre-development and post-development conditions of the site. The peak flow must be computed in accordance with Appendix B. All conveyance structures must be designed and sized in accordance with Chapter 4.

3.9.2.1 ONSITE DRAINAGE BASINS

Calculations of the peak flow must be submitted for the following storm events during pre- and post-development conditions.

A. Pre-development peak flow for the 2-year storm event.

B. Post-development peak flow for the (1) 2-year storm event; (2) 10-year storm event; and (3) 100-year storm event.

3.9.2.2 OFFSITE DRAINAGE BASINS

Development onsite may not change the offsite pre- and post-flow conditions. Calculations showing peak flow from all offsite basins impacting the site must be submitted for the following storm events: (1) 2-year storm event; (2) 10-year storm event; and (3) 100-year storm event.

4. CONVEYANCE STRUCTURES

4.1 GENERAL

A conveyance structure is a permanent waterway for the delivery of storm water. To allow continued and dependable access to subdivisions, and to protect buildings and drainfields, conveyance structures must be designed and constructed in accordance with this chapter, local regulations, and the manufacturer's recommendations. Impacts from sediment deposition and erosion must be addressed.

Conveyance structures must be designed to convey post-development peak flow without overtopping roadways or driveways during a 10-year storm event and without inundating any buildings or drain fields during a 100-year storm event. Flow rates must be calculated in accordance with Chapter 3.

Conveyance structures include open channels, storm sewer pipes, and culverts. Suggested formulas to determine conveyance structure capacity are provided in Appendix E, although other appropriate formulas may be used if approved by the reviewing authority. An example of a conveyance structure design is provided in Appendix M.

4.2 OPEN CHANNELS

Open channel conveyance structures have a free surface subject to atmospheric pressure and include, but are not limited to, ditches, swales, street gutters, and natural channels. They may be constructed of metal, concrete, or native materials.

Designs must include:

A. Channel capacity and velocity calculations;

B. A typical section view and plan view of each reach; and

C. Adequate protection from erosion.

4.3 STORM SEWERS

A storm sewer is a network of pipes that conveys surface drainage to an outfall from an inlet or through a manhole. Designs must include:

A. Pipe capacity and volume calculations. The design velocity for storm sewer pipes must be between 3 and 10 feet per second (fps). The minimum slope required to achieve these velocities is provided in Table 3 of Appendix E.

B. Profiles of the proposed storm sewer indicating size, type of pipe, percent grade, existing ground and proposed ground over the proposed system, and invert elevations at both ends of each pipe run.

C. Hydraulic grade line. Energy losses at inlets, transitions, and manholes along with flow considerations to avoid surcharging must be addressed.

D. No closed loops. For purposes of this circular, a closed loop is a network of pipes in which there is an inlet but not outlet for storm water.

4.4 CULVERTS

Culverts are conduits used to convey runoff under roadways, driveways, railroads, etc. Designs must include:

A. Culvert capacity and velocity calculations. If sized using inlet, barrel, and/or outlet controls, these assumptions must be documented.

B. Culvert inverts, roadway elevations, and runoff water elevations for both the 10-year and 100-year storm events.

C. Adequate protection from erosion at the outlet structure.

5. RETENTION AND DETENTION FACILITIES

5.1 GENERAL

Retention and detention storm water facilities:

A. Must be sized for the minimum volumes and peak flow rates in accordance with Chapter 3;

B. Must be at locations where the increased runoff will naturally accumulate, or where runoff can be directed;

C. Must be shown on the plans with cross-sections and design details provided; and

D. Should include safety precautions such as warning signs or fencing.

Designs must show that a retention facility, or discharge from a detention facility, will not overtop roads during a 10-year storm event and will not inundate buildings or drainfields during a 100-year storm event.

5.2 RETENTION FACILITIES

Retention facilities may be used in Standard and Simplified Plans. An example of a retention facility design is provided in Appendix J.

Retention facilities must be sized for the difference between the pre- and post-development runoff volumes, with no consideration for infiltration or designed outlet. The capacity of a retention facility may be used to satisfy the minimum volume requirement for an Initial Storm Water Facility, as described in Chapter 3.

Features such as topography, wetlands, floodplains, structures, utilities, property lines, and easements must be considered for the location and construction of the facility.

The side slopes of a retention facility must be no steeper than 3 to 1 and must be stabilized.

Retention facilities should be designed to infiltrate, evapotranspire, and/or capture for reuse storm water and to hold runoff no more than 72 hours.

5.3 DETENTION FACILITIES

Detention facilities may not be used in Simplified Plans. An example calculation of a detention facility is provided in Appendix L.

Detention facilities must capture runoff and release it at a flow rate equal to or less than the pre-development peak flow rate for the 2-year storm event.

The capacity of a detention facility may be used to satisfy the minimum volume requirement for an Initial Storm Water Facility, as described in Chapter 3. When included as part of the capacity of a detention facility, the volume of the Initial Storm Water Facility must be provided as either retention or infiltration below the elevation of the detention facility outlet.

Detention facilities should be designed to infiltrate, evapotranspire, and/or capture for reuse storm water and to hold runoff no more than 72 hours.

The outlet from a detention facility must be designed to provide a stabilized transition from the facility to the receiving area at non-erosive velocities.

Outlet structures may include weirs, discharge pipes, or other methods approved by the reviewing authority. Example drawings of outlet structures are provided in Figure 19 and Figure 20 in Appendix O.

Appendices D and N provide suggested formulas and information for sizing typical outlet structures. Other appropriate formulas may be used if approved by the reviewing authority.

6. INFILTRATION FACILITIES

6.1 GENERAL

Infiltration facilities must be sized in accordance with Chapter 3. Infiltration facilities include subsurface features such as drainage sumps, french drains, boulder pits, catch basins, dry wells, and surface features such as lawns and infiltration trenches. Infiltration facilities collect and discharge storm water runoff through infiltration into surrounding subsurface soils. Since infiltration facilities impound runoff only temporarily, they are normally dry during non-rainfall periods. Future land use and water-right permitting considerations should be addressed for infiltration facilities that rely on vegetation and irrigation. A typical subsurface infiltration facility detail is provided in Figure 21 in Appendix O.

Some infiltration facilities may be classified as Class V EPA injection wells. This Circular does not supersede or replace the standards promulgated by the EPA. Local and federal agencies should be contacted regarding other applicable rules, and authorization should be obtained prior to construction.

6.2 DESIGNS

The infiltration facility capacity may also include the required volume of the Initial Storm Water Facility.

Lawns and landscaping areas proposed as infiltration facilities must be sized using the appropriate runoff coefficient, curve number, or other factor consistent with the proposed land use and as designated by the selected design method in accordance with Appendix B.

Infiltration facilities, except lawns and landscaping, must:

A. Be sized based on infiltration rates in accordance with Appendix C;

B. Be constructed above the seasonal high groundwater level;

C. Be lined with a minimum 30 mil filter fabric or other material approved by the reviewing authority when needed to prevent clogging;

D. Be sized based on test data for the specified fill material or by assuming a fill material void space of 30%;

E. Be sized to drain within 48 hours; and

F. Include a pre-treatment facility, designed in accordance with Chapter 7, where sediment, trash, debris, or organic materials are likely to impact the operation or maintenance of the infiltration facility.

7. PRE-TREATMENT FACILITIES

7.1 GENERAL

Subchapter 6.2 requires the use of pre-treatment facilities where sediment, trash, debris, or organic materials are likely to impact the operation or maintenance of the infiltration facility. Pre-treatment facilities are recommended to preserve the longevity of all infiltration facilities, as well as detention, retention, and conveyance facilities. Pre-treatment facilities also may be used to increase the infiltration rates of soils, as provided in Table 2 of Appendix C.

Only those facilities described in this Chapter may be used as pre-treatment facilities. Pre-treatment facilities must be selected and designed to effectively treat storm water runoff for the purpose for which the facility is proposed.

7.2 DESIGNS

7.2.1 VEGETATIVE FILTER STRIPS

Vegetative filter strips reduce the velocity of storm water runoff, allowing settling of sediment. They work best when receiving runoff as sheet flow, making them suitable alongside roads, parking lots, and other paved surfaces.

7.2.2 VEGETATED SWALES

Vegetated swales are open channel conveyances that, when properly vegetated and designed with a shallow slope, provide for sedimentation and trash deposition.

7.2.3 SCREENS

Screens are used to prevent leaf litter and other debris from entering the system.

7.2.4 OIL/WATER SEPARATORS

Oil/water separators are designed specifically to remove petroleum hydrocarbons, grease, sand, and grit. The separators can be split into two categories: American Petroleum Institute (API) separators and coalescing plate separators (Water Environment Federation, WEF, 2012). API separators are vaults with baffles that enhance hydraulic efficiency. Coalescing plate separators use sloped plates or extruded tubes to achieve sediment and oil removal.

7.2.5 PROPRIETARY SPINNERS/SWIRL CHAMBERS/CENTRIFUGES

Proprietary spinners/swirl chambers/centrifuges cause storm water to move in a circular motion that enhances the settling of sediments, removes particulates, oils/greases, floatable sands, and debris. These must be installed in accordance with manufacturer specifications.

7.2.6 DRAIN INLET INSERTS

Drain inlet inserts are devices placed into storm water drains or catch basins to remove pollutants from storm water prior to entry into the storm sewer system. These inserts use an inert filter material, such as polypropylene, to enhance pollutant removal (WEF, 2012). Drain inlet inserts have the ability to remove debris, trash, and sediments and, if a filter material is present, can also remove oils/greases and other pollutants.

APPENDIX A - RAINFALL DATA

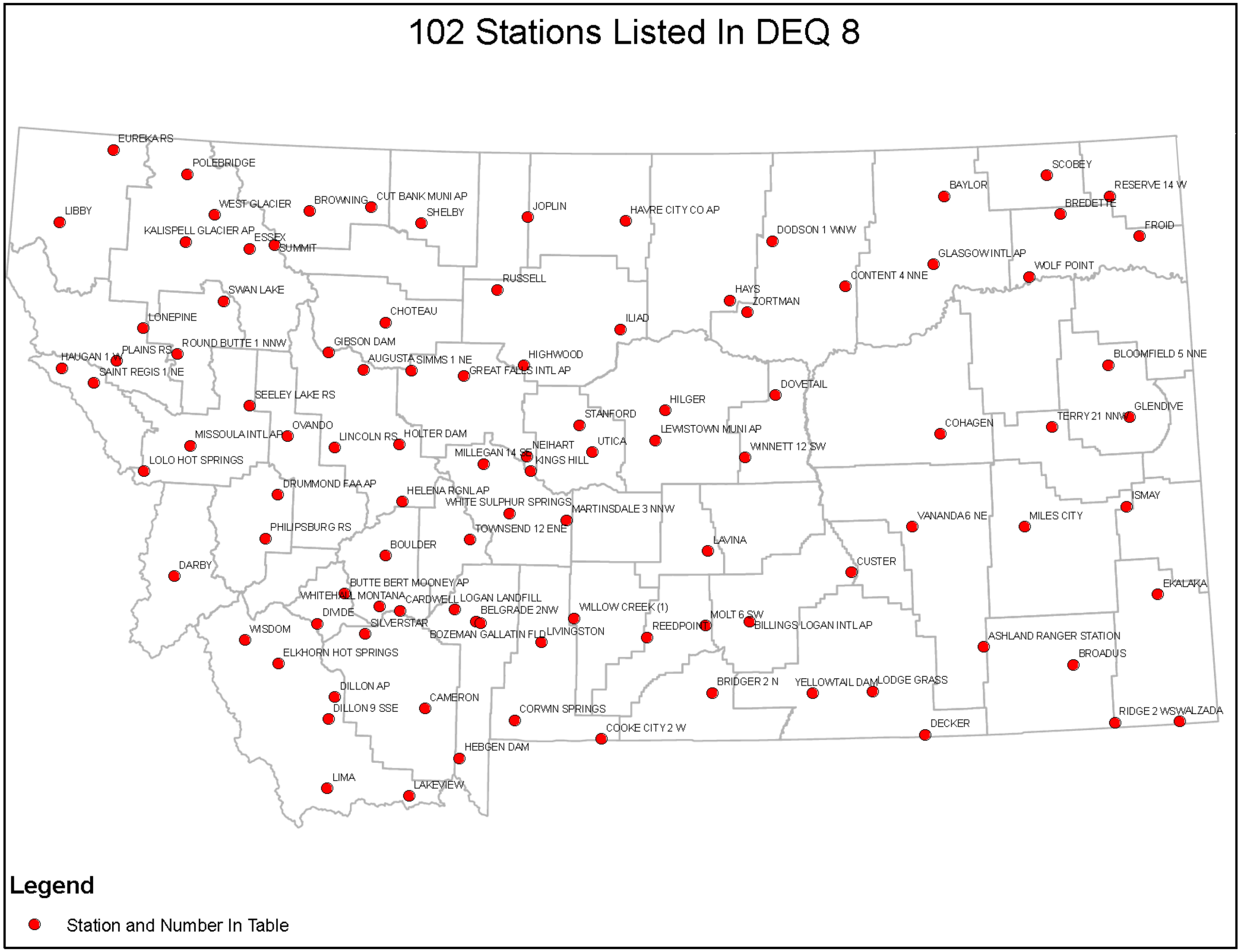

Table 1. Stations with ID Number Shown on the Station Map

Name	2yr-24hr	10yr-24hr	100yr-24hr
ALZADA	1.60	2.50	3.79
ASHLAND RANGER STATION	1.19	1.98	3.11
AUGUSTA	1.64	2.58	3.94
BAYLOR	1.59	2.49	3.79
BELGRADE 2NW	1.13	1.80	2.75
BILLINGS LOGAN INTL AP	1.42	2.21	3.48
BLOOMFIELD 5 NNE	1.59	2.49	3.77
BOULDER	1.04	1.60	2.40
BOZEMAN GALLATIN FLD	1.13	1.80	3.71
BREDETTE	1.55	2.44	2.98
BRIDGER 2 N	1.20	1.93	3.82
BROADUS	1.51	2.46	3.87
BROWNING	1.64	2.56	2.31
BUTTE BERT MOONEY AP	1.15	1.79	2.79
CAMERON	1.06	1.57	3.80
CARDWELL	1.19	1.85	3.26
CHOTEAU	1.50	2.45	4.05
COHAGEN	1.30	2.11	3.12
CONTENT 4 NNE	1.66	2.64	2.79
COOKE CITY 2 W	1.20	1.99	2.54
CORWIN SPRINGS	1.00	1.74	3.38
CUSTER	1.35	2.12	2.68
CUT BANK MUNI AP	1.41	2.32	2.76
DARBY	1.19	1.75	3.64
DECKER	1.38	2.21	3.13
DILLON 9 SSE	1.11	1.75	2.63
DILLON AP	1.05	1.65	4.61
DIVIDE	1.16	1.82	2.77
DODSON 1 WNW	1.57	2.42	3.68
DOVETAIL	1.25	2.02	2.90
DRUMMOND FAA AP	1.01	1.68	3.96
EKALAKA	1.81	2.96	4.17
ELKHORN HOT SPRINGS	1.16	1.82	4.14
ESSEX	1.60	2.45	4.00
EUREKA RS	1.36	1.99	3.63
FROID	1.59	2.57	2.68
GIBSON DAM	2.00	2.89	4.53

Table 1. Stations with ID Number Shown on the Station Map (*continued*)

Name	2yr-24hr	10yr-24hr	100yr-24hr
GLASGOW INTL AP	1.82	2.90	3.65
GLENDIVE	1.69	2.69	3.17
GREAT FALLS INTL AP	1.68	2.58	3.26
HAUGAN 1 W	1.92	2.77	4.11
HAVRE CITY CO AP	1.60	2.49	3.68
HAYS	1.48	2.36	2.94
HEBGEN DAM	1.30	1.87	4.58
HELENA RGNL AP	1.25	1.90	3.04
HIGHWOOD	2.16	3.14	3.32
HILGER	1.50	2.39	2.80
HOLTER DAM	1.37	2.11	2.60
ILIAD	1.33	2.13	2.99
ISMAY	1.68	2.68	2.95
JOPLIN	1.45	2.37	4.05
KALISPELL GLACIER AP	1.32	1.99	2.79
KINGS HILL	2.10	3.12	4.10
LAKEVIEW	1.46	2.11	2.50
LAVINA	1.18	2.06	3.49
LEWISTOWN MUNI AP	1.63	2.53	3.51
LIBBY	1.31	1.92	3.00
LIMA	1.17	1.76	3.35
LINCOLN RS	1.31	2.00	4.72
LIVINGSTON	1.10	1.86	2.90
LODGE GRASS	1.50	2.55	3.22
LOGAN LANDFILL	1.16	1.83	2.60
LOLO HOT SPRINGS	1.93	2.82	2.93
LONEPINE	1.20	1.73	3.39
MARTINSDALE 3 NNW	1.45	2.29	4.11
MILES CITY	1.43	2.28	4.28
MILLEGAN 14 SE	1.29	1.99	2.62
MISSOULA INTL AP	1.26	1.90	3.49
MOLT 6 SW	1.32	2.16	3.40
NEIHART	2.15	3.20	3.92
OVANDO	1.31	1.96	3.05
PHILIPSBURG RS	1.49	2.20	3.40
PLAINS RS	1.20	1.78	2.52
POLEBRIDGE	1.39	2.02	3.82
REEDPOINT	1.38	2.21	3.75
RESERVE 14 W	1.65	2.66	3.80
RIDGE 2 WSW	1.81	2.83	3.38

Table 1. Stations with ID Number Shown on the Station Map (*continued*)

Name	2yr-24hr	10yr-24hr	100yr-24hr
ROUND BUTTE 1 NNW	1.21	1.79	3.46
RUSSELL	1.39	2.25	2.85
SAINT REGIS 1 NE	1.60	2.34	3.73
SCOBEY	1.60	2.55	3.20
SEELEY LAKE RS	1.37	2.06	-0.10
SHELBY	1.33	2.18	3.08
SILVERSTAR	1.13	1.70	3.55
SIMMS 1 NE	1.45	2.42	3.64
STANFORD	1.53	2.44	2.59
SUMMIT	1.70	2.56	3.91
SWAN LAKE	1.60	2.33	3.29
TERRY 21 NNW	1.40	2.25	3.88
TOWNSEND 12 ENE	1.24	1.90	2.66
UTICA	1.56	2.45	3.35
VANANDA 6 NE	1.28	2.07	3.83
WEST GLACIER	1.63	2.39	3.23
WHITE SULPHUR SPRINGS	1.33	2.05	2.71
WHITEHALL MONTANA	1.09	1.74	3.62
WILLOW CREEK (1)	1.53	2.36	2.50
WINNETT 12 SW	1.53	2.40	3.88
WISDOM	0.97	1.64	2.82
WOLF POINT	1.70	2.61	2.81
YELLOWTAIL DAM	1.35	2.15	4.43
ZORTMAN	1.65	2.57	3.77

APPENDIX B - ACCEPTABLE HYDROLOGIC MODELS AND TIME OF CONCENTRATION

B.1 METHODS

B.1.1 RATIONAL METHOD AND MODIFIED RATIONAL METHOD

The Rational Method is appropriate for calculating peak flow of storm water runoff for areas less than 200 acres.

The Rational Method is represented by:

$$Q = C * i * A$$

Where: Q = flow (ft^3/sec or, in-ac/hour)
C = runoff coefficient (unitless)
i = intensity (in/hour)
A = Area (acres)

When using the Rational Method:

A. The runoff coefficient (C) must be a weighted average (C_w) of the site conditions below:
   1. paved or other hard surface areas — 0.90;
   2. gravel areas — 0.80;
   3. undeveloped areas — 0.20; or
   4. lawns or other landscaped areas — 0.10.

B. The intensity (i) must be determined using:
   1. tabulated rainfall data in Appendix A. This data is a conservative estimate of intensity and the value must be assumed to be in/hour or,
   2. Intensity-Duration-Frequency (IDF) curve developed for the location of the development for a time period equal to the time of concentration of the drainage basin. The minimum time of concentration is 5 minutes. For multiple sub-drainage areas, the longest time of concentration must be selected. IDF curves for selected areas are available from the Department.

The Modified Rational Method is used to calculate volume of storm water runoff using the flow (Q) calculated using the Rational Method equation.

The total volume of runoff can be represented through Figure 1 in the Storage-Indication Routing method below and calculated by:

$$V = T_D * Q$$

Where: V= Volume (cubic feet)
T_d = Storm Duration (minimum of 3600 seconds)
Q = peak flow rate (cfs)

If the Rational Method is used to size a detention facility, the synthetic hydrograph in Figure 1 must be used to determine runoff volume using a time of concentration (T_c). The minimum duration (T_d) is 1 hour or 3600 seconds.

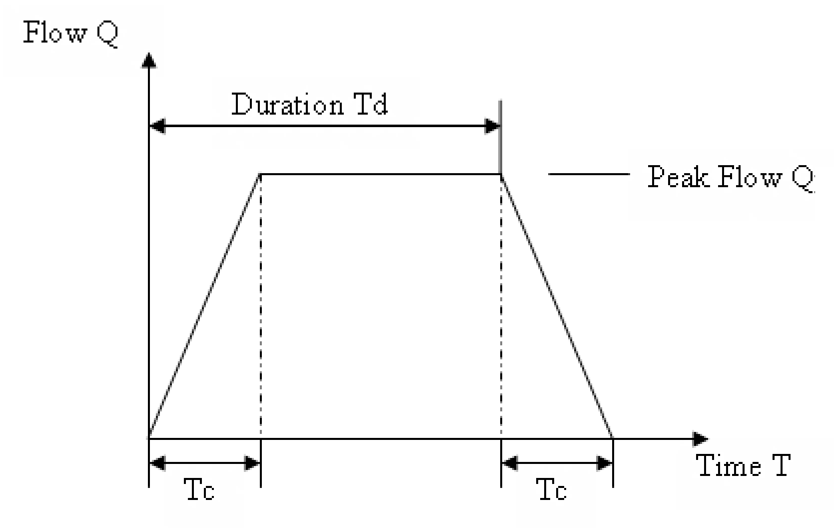

Figure 1. Modified Rational Method Synthetic Hydrograph

The example spreadsheets in Appendices F and G use the Rational and Modified Rational Method to calculate volume and flow.

B.1.2 URBAN HYDROLOGY FOR SMALL WATERSHEDS TECHNICAL RELEASE 55 (TR-55) OR SCS CURVE NUMBER METHOD

Urban Hydrology for Small Watersheds TR-55 is based on the SCS Curve Number Method. TR-55 can also be used for storage and routing effects for many structures, and for multistage outflow devices. The applicability of TR-55 is limited to drainage areas of 3 square miles or smaller.

The TR-55 method is represented by:

$$Q = \frac{(P - 0.2S)^2}{P + 0.8S}, \qquad I_a = 0.2S, \qquad S = \left(\frac{1000}{CN}\right) - 10$$

Where:
Q = runoff (units of inches)
P = precipitation (rainfall in units of inches)
S = potential maximum retention after runoff begins (unitless)
I_a = initial abstractions (unitless)
CN = curve number (unitless)

When using TR-55:

A. The hydrologic group for each soil type, vegetation/land use, and slope of the site must be known.

B. The soil type curve number must be computed as a weighted average of the site conditions.

C. The minimum time of concentration is 5 minutes.

D. The rainfall intensity must be determined using an IDF curve for a time period equal to the time of concentration of the drainage basin. IDF curves for selected areas are available from the Department.

E. For multiple sub-drainage areas, the longest time of concentration must be selected.

F. Initial abstractions may not exceed more than 50% of the total precipitation ($I_a/P < 0.50$)

G. Refer to the TR-55 manual for more detailed discussions and limitations.

B.1.3 STORAGE-INDICATION ROUTING

Storage-Indication Routing may be used to analyze storage detention practices. This approach requires that the inflow hydrograph be developed through one of the methods listed (TR-55, WinTR-55, SWMM, Rational Method, etc.), as well as the required maximum outflows.

The Storage-Indication Routing method is represented by:

$$\frac{I_1 + I_2}{2} - \frac{O_1 + O_2}{2} = \frac{S_2 - S_1}{t_2 - t_1}$$

Where:
I_1 = inflow rate at t_1 (units of ft^3/sec)
I_2 = inflow rate at t_2 (units of ft^3/sec)
O_1 = outflow rate at t_1 (units of ft^3/sec)
O_2 = outflow rate at t_2 (units of ft^3/sec)
t_1 = time at the beginning of the interval (units of seconds)
t_2 = time at the end of the interval (units of seconds)
S_1 = storage volume at t_1 (units of ft^3)
S_2 = storage volume at t_2 (units of ft^3)

B.2 TIME OF CONCENTRATION

Time of concentration must be calculated using an applicable combination of sheet flow, shallow overland flow, concentrated/channel flow, and culvert/pipe flow. Sheet flow length must be limited to a maximum of 300 feet, as most sheet flow becomes shallow concentrated flow at greater flow lengths.

Pre-development time of concentration must be based on the sum of computed or estimated flow times across and through the natural features.

Post-development time of concentration must be based on the sum of computed or estimated flow times across the developed site and through proposed conveyance and storm water drainage facilities.

For multiple drainage areas, the longest time of concentration must be selected.

Time of concentration for sheet flow can be calculated using the soil conservation service (SCS) equation:

$$T_{t-sheet\ flow} = \frac{0.007(nL)^{0.8}}{(P_2)^{0.5} s^{0.4}}$$

Where:
T_t = travel time (hr),
n = Manning's roughness coefficient
L = flow length (ft, max of 300 ft)
P_2 = 2-year, 24-hour rainfall (in)
s = slope of hydraulic grade line (land slope, ft/ft)

Time of concentration for shallow flow can be calculated using the SCS equation:

$$T_{t-shallow\ flow} = \frac{L}{3600V}$$

Where: T_t = travel time (hr),
L = flow length (ft, max of 300 ft)
V = velocity, ft/s (see Figure 18)

B.3 COMPUTER MODELS

Computer models such as Hydraflow extensions for AutoCad, HEC-1, WINTR-55, WINTR-20, and SWMM may also be used to calculate peak flow and volume of storm water runoff, storage routing effects, and detention outflow sizing.

When using computer models:

A. The minimum time of concentration is 5 minutes.

B. The rainfall intensity must be determined using an IDF curve for a time period equal to the time of concentration of the drainage basin.

C. For multiple sub-drainage areas, the longest time of concentration, either individually or collectively, must be selected.

D. Computations and assumptions for the model must be provided.

E. Inflow-outflow hydrographs must be presented graphically.

F. Schematic (node) diagrams must be provided for all routings.

APPENDIX C - INFILTRATION TESTING PROCEDURES

One of the following methods must be used to determine the design infiltration rate:

A. Design Infiltration Rate in C.1; or

B. Encased Falling Head Test in C.2.

C.1 DESIGN INFILTRATION RATE

For infiltration systems with less than 5,000 square feet, a design infiltration rate may be selected from Table 2 using the texture of the least-permeable soil layer encountered in a soil test pit. The soil test pit must be located within 25 feet of the infiltration facility and at the infiltrative depth. The infiltration rates in Table 2 may be increased by 50 percent with the use of sediment reducing pre-treatment facilities in accordance with Chapter 7.

Table 2. Infiltration Rates

Texture	Infiltration rate (inches per hour)
Gravel, gravelly sand, or very coarse sand (c)	2.6
Loamy sand, coarse sand (d)	1.05
Medium sand, sandy loam	0.9
Fine sandy loam, loam	0.7
Very fine sand, sandy clay loam, silt loam	0.7
Clay loam, silty clay loam	0.07
Sandy clay	0.07
Clays, silts, silty clays (e)	0.0

C.2 ENCASED FALLING HEAD TEST

The encased falling head test is performed with a 6-inch casing that is embedded approximately 24 inches into the native soil. The goal of this field test is to evaluate the vertical infiltration rate through a 24-inch plug of soil, without allowing any lateral infiltration. The test is not appropriate in gravelly soils or in other soils where a good seal with the casing cannot be established.

A minimum of three encased falling head tests must be conducted within the footprint of each infiltration system. For proposed infiltration systems with more than 10,000 square feet of infiltration area, one additional encased falling head test is required for each additional 10,000 square feet. Different soil types may be encountered during the soil infiltration testing; a minimum of two encased falling head tests per soil type are required. The encased falling head test locations must be spaced throughout the proposed infiltration system. The results of the infiltration tests must be averaged to determine the measured infiltration rate for the infiltration system. The measured infiltration must be divided by a safety factor of 2.0 to arrive at the design infiltration rate.

C.3 ENCASED FALLING HEAD TEST PROCEDURE:

A. Embed a solid 6-inch diameter casing into the native soil at the elevation of the proposed facility bottom. Ensure that the embedment provides a good seal around the pipe casing so that percolation will be limited to the 24-inch plug of the material within the casing. The minimum casing length must be 48 inches; longer casings may be used.

B. Fill the 6-inch diameter casing with clean water a minimum of 24 inches above the soil to be tested, and maintain this depth for at least 4 hours (or overnight if clay soils are present) to presoak the native material. In sandy soils with little or no clay or silt, soaking is not necessary. If the water infiltrates completely in less than 10 minutes after filling the hole twice with 24 inches of water, the test can proceed immediately.

C. To conduct the first trial of the test, fill the 6-inch diameter casing to approximately 24 inches above the soil and measure the water level to the nearest 0.01 foot (⅛ inch). The head used in the test may be greater than 24 inches, provided the head is not greater than 50 percent of the maximum head in the proposed infiltration system. The pre-saturation head must be the same as the infiltration testing. The level must be measured with a tape or other device with reference to a fixed point. The top of the pipe is often a convenient reference point. Record the exact time.

D. Measure the water level to the nearest 0.01 foot (⅛ inch) at 10-minute intervals for a total period of 1 hour (or 20-minute intervals for 2 hours in slower soils) or until all the water has infiltrated. In faster draining soils (sands and gravels), it may be necessary to shorten the measurement interval in order to obtain a well-defined infiltration rate curve. Constant head tests may be substituted for falling head tests with prior approval of the reviewing authority. Successive trials must be run until the percent change in measured infiltration rate between two successive trials is minimal. The trial must be discounted if the infiltration rate between successive trials increases. At least three trials must be conducted. After each trial, the water level must be readjusted to the 24-inch level. Enter results into the data table.

E. Measure the depth and approximate volume of any water that accumulates in the borehole or trench around the test casing, which indicates a bad seal around the pipe or short circuiting through the soil being tested.

F. The average infiltration rate over the last trial must be used to calculate the measured infiltration rate

G. The location of the test must correspond to the infiltration system location.

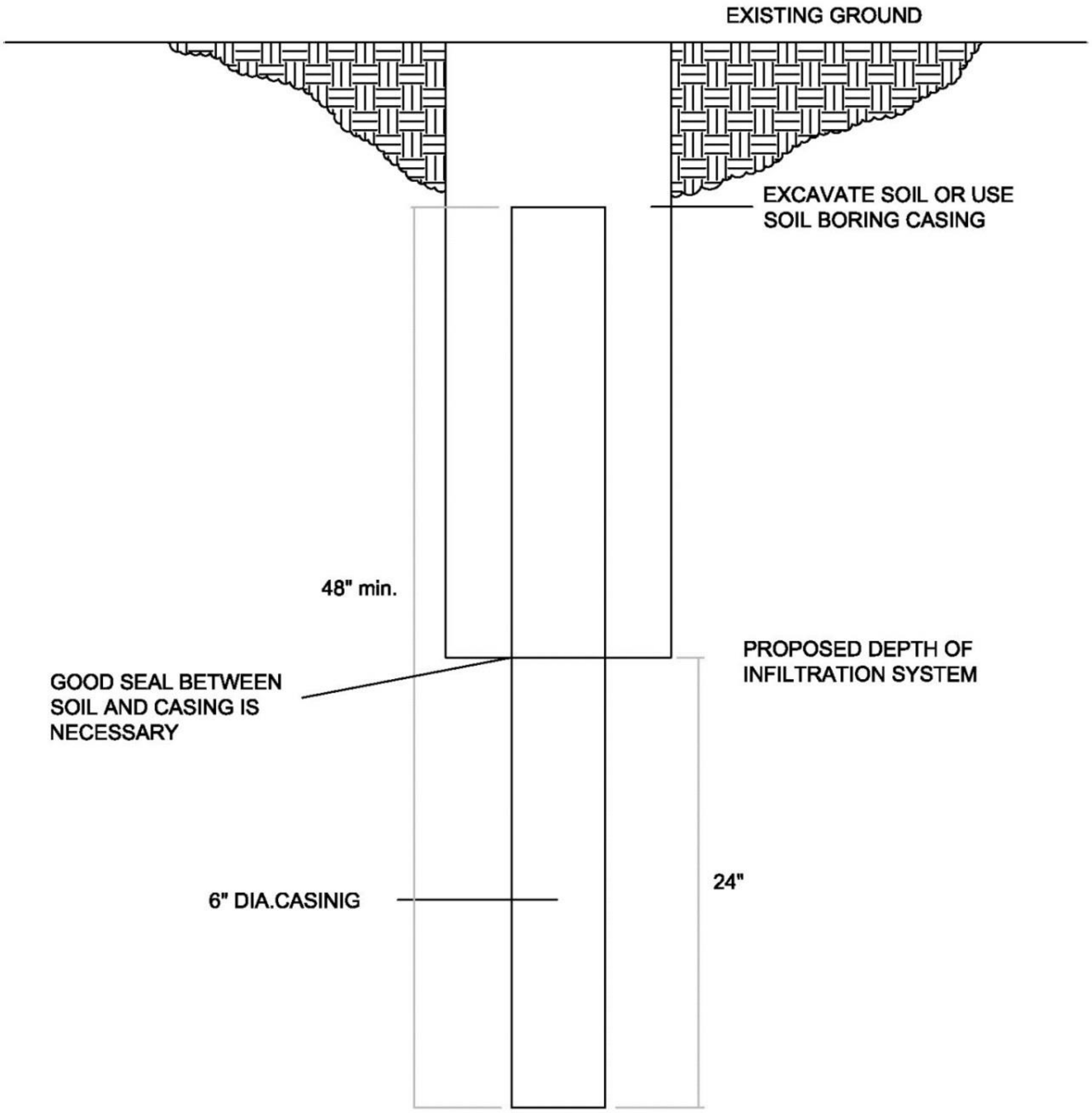

Figure 2. Encased Falling Head Schematic

APPENDIX D - DETENTION OUTLET STRUCTURE EQUATIONS

D.1 CIRCULAR ORIFICES

Design capacity of all circular orifices may use the following equation, although other appropriate equations may be used if approved by the reviewing authority:

$$Q = CA(2gh)^{0.5}$$

Where:
Q = orifice discharge (cfs)
C = discharge coefficient = 0.6
A = orifice cross-sectional area = $3.1416(D^2/4)$ (ft^2)
g = 32.2 ft/sec^2 (gravitational acceleration)
h = hydraulic head above the center of the orifice (ft)

D.2 WEIRS

Design capacity of all weirs may use the following equation, although other appropriate equations may be used if approved by the reviewing authority:

Rectangular
$$Q = 3.33h^{1.5} * (L - 0.2h)$$

60˚ V-notch
$$Q = 1.43h^{2.5}$$

90˚ V-notch
$$Q = 2.49h^{2.48}$$

Cipoletti
$$Q = 3.367bh^{1.5}$$

Where:
Q = flow through the weir (cfs)
h = hydraulic head above the bottom of the weir (ft)
L = length of the weir crest (ft)
b = base width of Cipoletti weir (ft)

APPENDIX E - CONVEYANCE STRUCTURE EQUATIONS

E.1 CHEZY-MANNING FORMULA

The Chezy-Manning formula may be used to compute the conveyance capacities. Other appropriate formulas may be used if approved by the reviewing authority. The Chezy-Manning formula is as follows:

$$Q = \frac{1.486}{n} * A * R^{2/3} * S^{1/2}$$

Where:
Q = channel flow (cfs)
n = Manning's roughness coefficient
A = cross-sectional area of flow (ft^2)
R = hydraulic radius (ft)
S = channel slope (ft/ft)
WP = wetted perimeter
R = A/WP

References for Manning's n determinations must be provided. Values for Manning's n for different conveyance features and materials may be found in various hydrology textbooks and publications such as Natural Resources and Conservation Urban Hydrology for Small Watersheds (TR-55) and the Federal Highway Administration Hydraulic Engineering Circular No. 22, Third Edition (HEC-22).

Computations for the flow velocity of the channels may use the continuity equation, although other appropriate equations may be used if approved by the reviewing authority:

$$Q = A * V$$

Where:
V = velocity (ft/sec)
A = cross-sectional area of the flow (ft^2)

Hydraulic grade-line calculations showing supercritical or subcritical flow regimes may be required by the reviewing authority.

E.2 CURB AND GUTTER

Design capacity of curb and gutter sections may use the following equation, although other formulas may be used if deemed appropriate by the reviewing authority:

$$Q = \left(\frac{0.56}{n}\right) * (S_x)^{1.67} * S^{0.5} * T^{2.67}$$

Where:
Q = flow rate (cfs)
n = Manning's roughness coefficient
S_x = cross slope (ft/ft)
S = longitudinal slope (ft/ft)
T = width of flow or spread (ft)

E.3 STORM SEWER VELOCITIES

The design velocity for storm sewer pipes must be between 3 to 10 feet per second (fps). Velocity is calculated under full flow conditions even if the pipe is only flowing partially full with the design storm. The minimum slope required to achieve these velocities is provided in Table 3.

Table 3. Minimum Grades to Ensure 3 fps for Full Flow (ft/ft)

Pipe Size (in)	Q Full Flow (cfs)	Grade (ft/ft)
12	2.36	.0037
15	3.68	.0028
18	5.30	.0022
21	7.22	.0018
24	9.43	.0015
27	11.93	.0013
30	14.73	.0011
33	17.82	.00097
36	21.21	.00086

APPENDIX F - SPREADSHEET – SIMPLIFIED PLAN

Appendix F: Simplified Storm Drainage Plan

DEQ Montana Department of Environmental Quality

Sudivision Name			
EQ#			
County			
Location			
Lot/Area No.			
Max. Slope on Lot		%	OK
Impervious Surfaces		%	OK
Will Alter Off-site Pass-Through?			STOP, Sumbit a DEQ-8 Plan

Rational Method Co-Efficients (C)	
0.9	Paved/hard surfaces
0.8	Gravel surfaces
0.1	Lawn/landscaping
0.2	Unimproved areas

Q=C*i*A

100-year, 24-hour, i ____ inches

Total Area/Lot Size ____ acres = 0 ft^2

Pre-Development Characteristics			100-year, 24-hour i (volume)	
Paved/House Area	0 acres	0 ft^2	V=	0 ft^3
Gravel Area	0 acres	0 ft^2	V=	0 ft^3
Lawn/Landscaping	0 acres	0 ft^2	V=	0 ft^3
Unimproved Area	0 acres	0 ft^2	V=	0 ft^3
Total	0 acres	0 ft^2	V_{Total}=	**0.00** ft^3

Post-Development Characteristics			100-year, 24-hour i (volume)	
Paved/House Area	0 acres	0 ft^2	V=	0 ft^3
Gravel Area	0 acres	0 ft^2	V=	0 ft^3
Lawn/Landscaping	0 acres	0 ft^2	V=	0 ft^3
Unimproved Area	0 acres	0 ft^2	V=	0 ft^3
Total	0 acres	0 ft^2	V_{Total}=	**0.00** ft^3

Increase in Runoff Volume (Minimum Retention Pond Size)	ΔV=	**0.00** ft^3

= input field

APPENDIX G - SPREADSHEET – STANDARD PLAN

Appendix G: Standard Storm Drainage Plan

DEQ Montana Department of Environmental Quality

Sudivision Name	
EQ#	
County	
Location	
Lot/Area No.	

Rational Method Co-Efficients	
0.9	Paved/hard surfaces
0.8	Gravel surfaces
0.1	Lawn/landscaping
0.2	Unimproved areas

Q=C*i*A

Intensity Values

2-year, T_c		inches/hour
2-year, 24-hour		inches
10-year, T_c		inches/hour
100-year, T_c		inches/hour
100-year, 24-hour		inches

Total Area/Lot Size [ ] acres = 0 ft^2

Initial Stormwater Facility Volume (0.5" x Impervious Area) : 0 ft^3

Pre-Development Characteristics			2-year, T_c (flow rate)	2-year, 24-hour (volume)	10-year, T_c (flow rate)	100-year, T_c (flow rate)	100-year, 24-hour (volume)
Paved/House Area	0 acres	ft^2	Q= 0.000 ft^3/sec	V= 0.000 ft^3	Q= 0.000 ft^3/sec	Q= 0.000 ft^3/sec	V= 0.000 ft^3
Gravel Area	0 acres	ft^2	Q= 0.000 ft^3/sec	V= 0.000 ft^3	Q= 0.000 ft^3/sec	Q= 0.000 ft^3/sec	V= 0.000 ft^3
Lawn/Landscaping	0 acres	ft^2	Q= 0.000 ft^3/sec	V= 0.000 ft^3	Q= 0.000 ft^3/sec	Q= 0.000 ft^3/sec	V= 0.000 ft^3
Unimproved Area	0 acres	0 ft^2	Q= 0.000 ft^3/sec	V= 0.000 ft^3	Q= 0.000 ft^3/sec	Q= 0.000 ft^3/sec	V= 0.000 ft^3
Total	0 acres	0 ft^2	Q_{Total}= **0.000** ft^3/sec	V_{Total}= **0.000** ft^3	Q_{Total}= **0.000** ft^3/sec	Q_{Total}= **0.000** ft^3/sec	V_{Total}= **0.000** ft^3

Post-Development Characteristics			2-year, T_c (flow rate)	2-year, 24-hour volume)	10-year, T_c (flow rate)	100-year, T_c (flow rate)	100-year, 24-hour (volume)
Paved/House Area	0 acres	ft^2	Q= 0.000 ft^3/sec	V= 0.000 ft^3	Q= 0.000 ft^3/sec	Q= 0.000 ft^3/sec	V= 0.000 ft^3
Gravel Area	0 acres	ft^2	Q= 0.000 ft^3/sec	V= 0.000 ft^3	Q= 0.000 ft^3/sec	Q= 0.000 ft^3/sec	V= 0.000 ft^3
Lawn/Landscaping	0 acres	ft^2	Q= 0.000 ft^3/sec	V= 0.000 ft^3	Q= 0.000 ft^3/sec	Q= 0.000 ft^3/sec	V= 0.000 ft^3
Unimproved Area	0 acres	0 ft^2	Q= 0.000 ft^3/sec	V= 0.000 ft^3	Q= 0.000 ft^3/sec	Q= 0.000 ft^3/sec	V= 0.000 ft^3
Total	0 acres	0 ft^2	Q_{Total}= **0.000** ft^3/sec	V_{Total}= **0.000** ft^3	Q_{Total}= **0.000** ft^3/sec	Q_{Total}= **0.000** ft^3/sec	V_{Total}= **0.000** ft^3

Runoff Flow/Volume Change					
	ΔQ= **0.000** ft^3/sec	ΔV= **0.000** ft^3	ΔQ= **0.000** ft^3/sec	ΔQ= **0.000** ft^3/sec	ΔV= **0.000** ft^3

Required Minimum Facility Volume: **0** ft^3

= input field

APPENDIX H - INITIAL STORM WATER FACILITY EXAMPLE

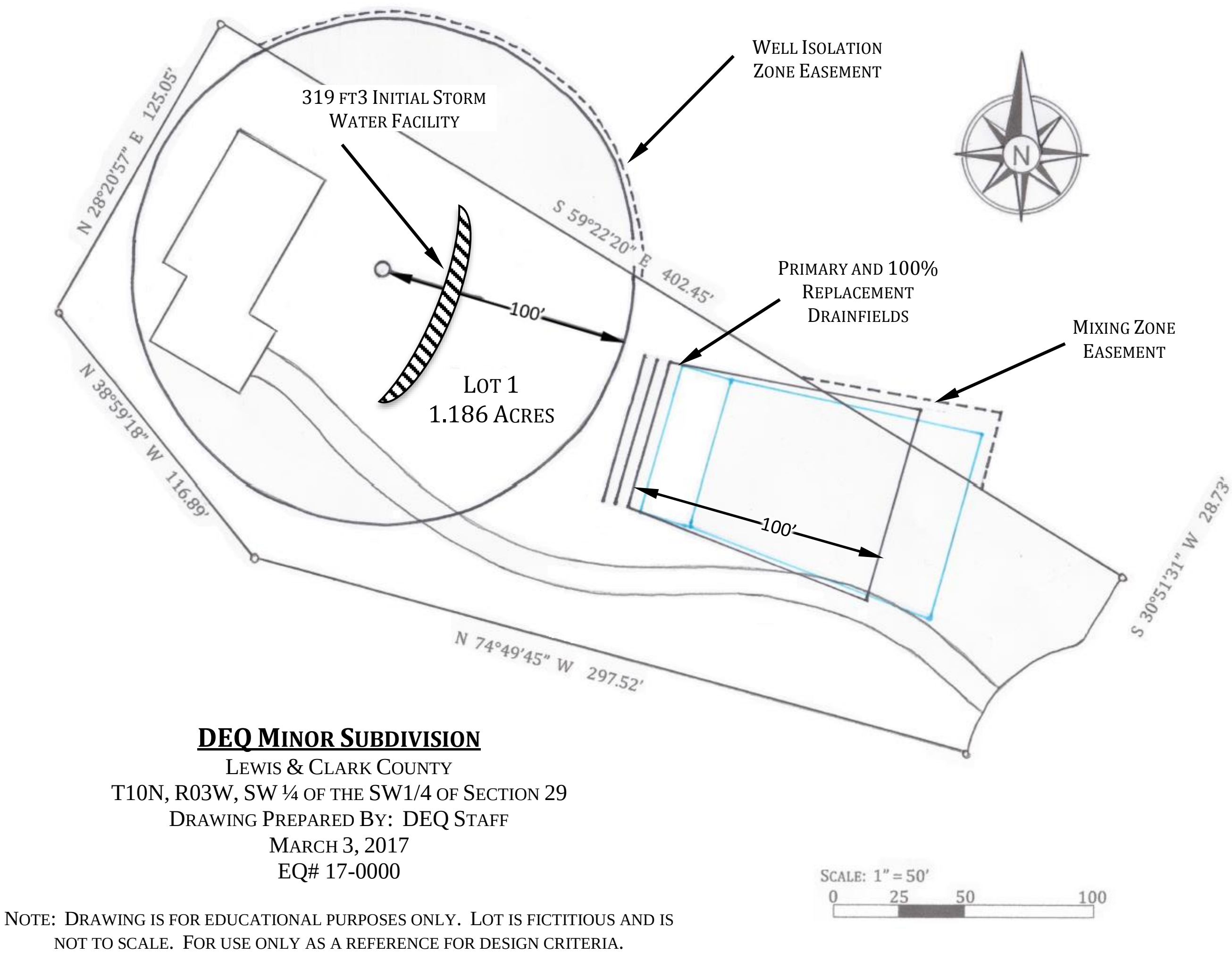

Figure 3. Initial Storm Water Facility Lot Layout

Given the following hypothetical conditions, determine the minimum facility volume required to retain and/or infiltrate the first 0.5 inches of rainfall from any storm event:

Location: Helena, Montana
Lot size: 1.186 acres
No previous approval
Current use is short grass prairie
No setbacks, easements (other than those shown), rights-of-way, surface water, floodplains
Post –Development includes:
- 3,600 ft^2 of house/roof
- 300 ft^2 patio
- 10,000 ft^2 of lawn and landscaped area
- 3,750 ft^2 gravel driveway

Solution: First, determine the total impervious area. Landscaping and undeveloped areas are not included in this facility sizing, so the total impervious area is the house/roof, patio, and gravel driveway.

$$A_{imp} = 3{,}600ft^2_{\frac{house}{roof}} + 300ft^2_{patio} + 3{,}750ft^2_{gravel\ driveway}$$
$$A_{imp} = 7{,}650ft^2_{total\ impervious}$$

Using the equation given in Subchapter 3.4, with A_{imp} = 7,650 ft^2 and P equal to 0.5 inches:

$$V = \frac{(0.5\ inches * 7{,}650ft^2_{total\ impervious})}{12\frac{inches}{ft}} = 318.75ft^3$$

The **Initial Storm Water Facility** must provide a minimum volume of **319 ft^3** of storage to retain or infiltrate the first 0.5 inches from a storm event.

APPENDIX I - SIMPLIFIED PLAN EXAMPLE

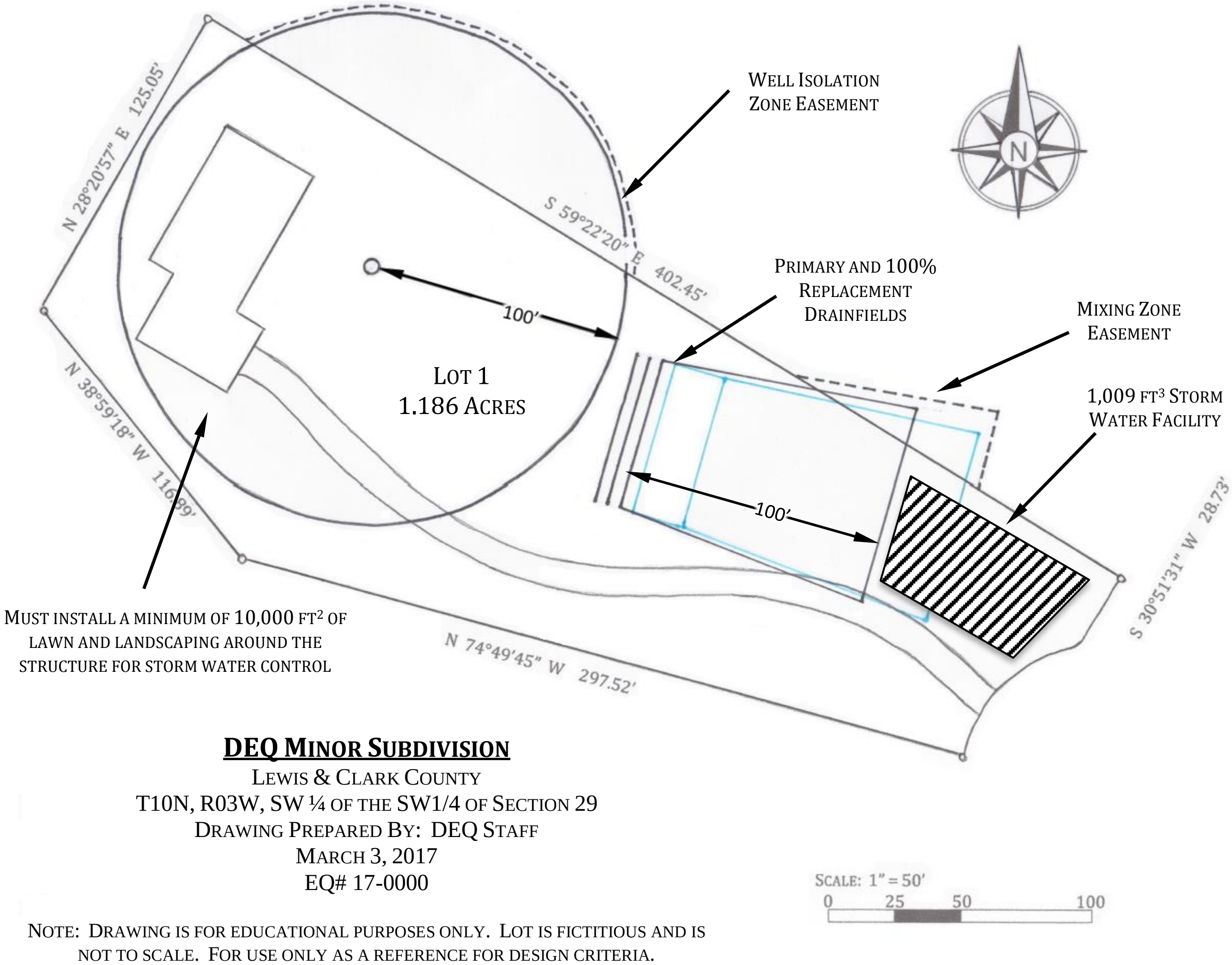

Figure 4. Simplified Plan Lot Layout

Given the following hypothetical conditions, create a Simplified Storm Drainage design:

- Location: Helena, Montana
- Lot size: 1.186 acres
- No previous approval
- Current use is short grass prairie
- No setbacks, easements (other than those shown), rights-of-way, surface water, floodplains
- Post-Development includes:
  - 3,600 ft^2 of house/roof
  - 300 ft^2 patio
  - 10,000 ft^2 of lawn and landscaped area
  - 3,750 ft^2 gravel driveway

Solution: First, determine if the proposed development meets the criteria of the Simplified Plan outlined in Subchapter 3.2.

- The slope of the disturbed area is less than 3%. The parcel has minimal slope, with a maximum slope of 2.5% in the area of the proposed driveway, so slopes will not exceed 3%.
- The total impervious area is the sum of the house/roof, patio, and gravel driveway and must be less than 25% of the total lot size.

$$25\%\ of\ lot = 1.186\ acres * \frac{43{,}560 ft^2}{acre} * \frac{25}{100} = 12{,}916 ft^2$$

$$A_{imp} = 3{,}600 ft^2_{\frac{house}{roof}} + 300 ft^2_{patio} + 3{,}750 ft^2_{gravel\ driveway}$$

$$A_{imp} = 7{,}650 ft^2_{total\ impervious} < 12{,}916 ft^2 \ \therefore\ OK$$

- There is not any historical storm water through the lots, so new development will not alter any pre-development flow patterns.

The proposed development meets all the criteria of a Simplified Plan. The minimum retention facility size can be determined using the Modified Rational Method (example spreadsheet provided in Appendix F) and the 100-year rainfall intensity for Helena of 3.04 inches (provided in Appendix A).

The formula for the Rational Method used to determine the volume of a retention facility is:

$$Q = C_w * i * A$$

Using the rational coefficients in Appendix B.1.1, the pre-development coefficient of runoff is $C_{w\text{-}pre} = 0.2$. The pre-development runoff volume is:

$$Q_{pre-development} = 0.2 * 3.04 inches * 1.186\ acres * \frac{43{,}560\ ft^2}{1\ acre} * \frac{1\ ft}{12\ in} = 2{,}617.55\ ft^3$$

The weighted post-development co-efficient is:

$$C_{w-post} = \frac{C_1A_1 + C_2A_2 + \ldots + C_nA_n + C_{n+1}A_{n+1}}{A_{total}}$$

$$C_{w-post} = \frac{[0.9(3{,}600 + 300) + 0.8(3{,}750) + 0.1(10{,}000) + 0.2(34012)]ft^2}{1.186\ acres * \frac{43{,}560 ft^2}{acre}}$$

$$C_{w-post} = \frac{(3{,}510 + 3{,}000 + 1{,}000 + 6{,}802.4)ft^2}{51{,}662.16 ft^2} = 0.277$$

The post-development runoff volume is:

$$\begin{aligned} Q_{post-development} &\\ &= 0.277 * 3.04 inches * 1.186\ acres * \frac{43{,}560\ ft^2}{1\ acre} * \frac{1\ ft}{12\ in} \\ &= 3{,}625.82\ ft^3 \end{aligned}$$

The minimum retention facility size is that which will retain the difference in runoff (increase) between the pre- and post-development conditions.

$$V_{minimum} = Q_{post-development} - Q_{pre-development}$$

$$V_{minimum} = 3{,}625.82\ ft^3 - 2{,}617.55\ ft^3 = 1{,}008.27 ft^3$$

The **Simplified Plan Facility** must provide a minimum volume of **1,009 ft^3** of storage to retain the increase in runoff from the 100-year storm event. This is larger than the 319 ft^3 of storage required for the example Initial Storm Water Facility in Appendix H, so it meets the design criteria of both the Initial Storm Water Facility and the Simplified Plan.

APPENDIX J - STANDARD PLAN – RETENTION FACILITY EXAMPLE

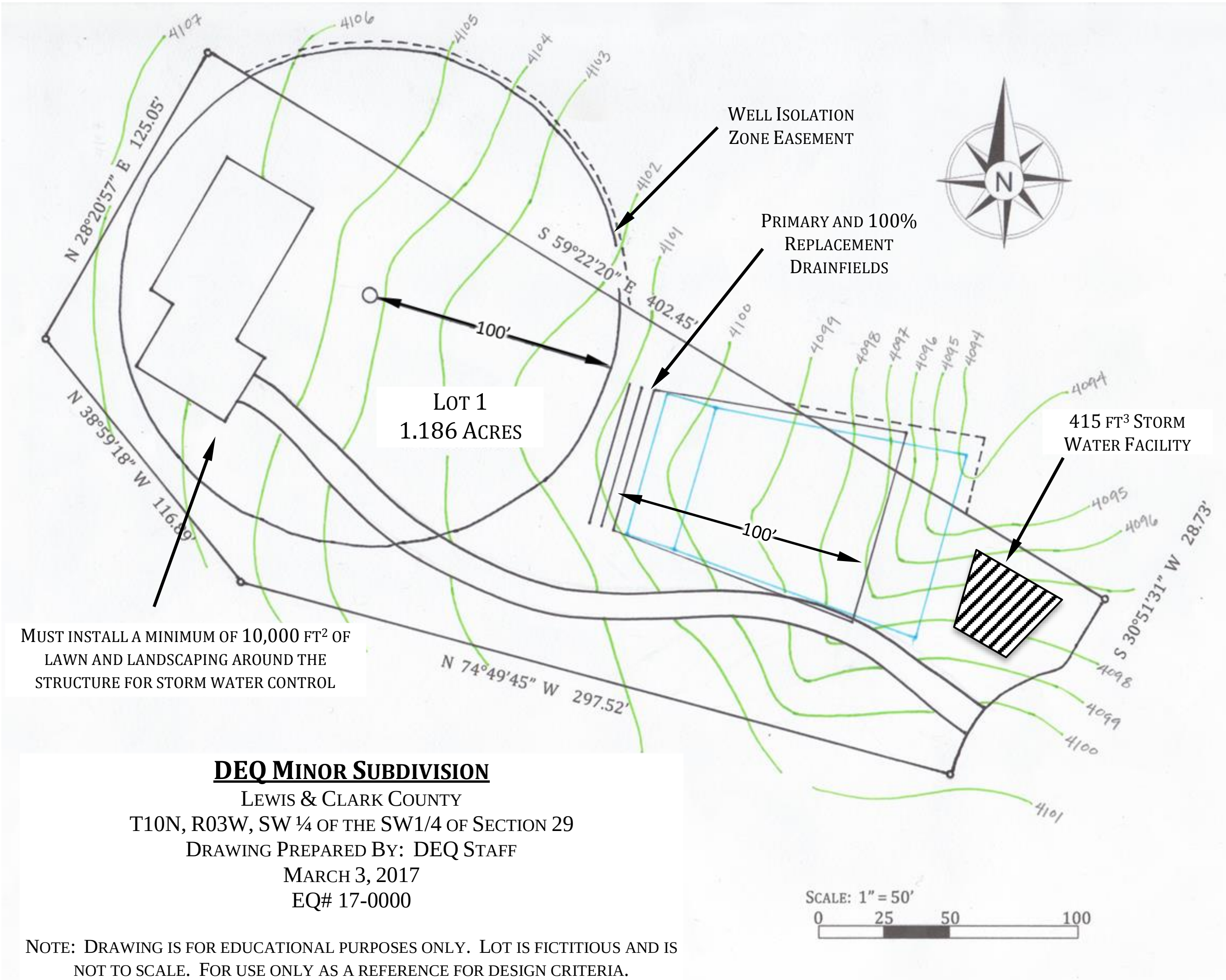

Figure 5. Standard Plan Retention Facility Lot Layout

Given the following hypothetical conditions, create a standard retention storm drainage design:

Location: Helena, Montana
Lot size: 1.186 acres
No previous approval
Current use is short grass prairie
No setbacks, easements (other than those shown), rights-of-way, surface water, floodplains
Post-Development includes:
- 3,600 ft^2 of house/roof
- 300 ft^2 patio
- 10,000 ft^2 of lawn and landscaped area
- 3,750 ft^2 gravel driveway
- Some impervious areas have slopes in excess of 3%.

Solution: The minimum retention facility size can be determined using the Modified Rational method (example spreadsheet provided in Appendix F) and the rainfall intensity for Helena (provided in Appendix A).

The minimum size of the retention facility for the Standard Plan must be determined using the 2-year storm event. Using Appendix A, find that the 2-year rainfall intensity for Helena is 1.25 inches.

The formula for the Rational Method used to determine the volume of a retention facility is:

$$Q = C_w * i * A$$

Where: Q = flow (ft^3/sec or, in-ac/hour)
C = runoff coefficient (unitless)
i = intensity (in/hour)
A = Area (acres)

Using the rational coefficients in Appendix B.1.1, the pre-development coefficient of runoff is $C_{w\text{-}pre} = 0.2$. The pre-development runoff volume for the 2-year storm event is:

$$Q_{pre-development} = 0.2 * 1.25 inches * 1.186\ acres * \frac{43{,}560\ ft^2}{1\ acre} * \frac{1\ ft}{12\ in} = 1{,}076.30\ ft^3$$

The weighted post-development co-efficient is:

$$C_{w-post} = \frac{C_1A_1 + C_2A_2 + \ldots + C_nA_n + C_{n+1}A_{n+1}}{A_{total}}$$

$$C_{w-post} = \frac{[0.9(3{,}600 + 300) + 0.8(3{,}750) + 0.1(10{,}000) + 0.2(34012)]ft^2}{1.186\ acres * \frac{43{,}560 ft^2}{acre}}$$

$$C_{w-post} = \frac{(3{,}510 + 3{,}000 + 1{,}000 + 6{,}802.4)ft^2}{51{,}662.16 ft^2} = 0.277$$

The post-development runoff volume is:

$$Q_{post-development}$$
$$= 0.277 * 1.25 inches * 1.186\ acres * \frac{43{,}560\ ft^2}{1\ acre} * \frac{1\ ft}{12\ in}$$
$$= 1{,}490.88\ ft^3$$

The minimum retention facility size is that which will retain the difference in runoff (increase) between the pre- and post-development conditions.

$$V_{minimum} = Q_{post-development} - Q_{pre-development}$$

$$V_{minimum} = 1{,}076.30\ ft^3 - 1{,}490.88\ ft^3 = 414.58 ft^3$$

The **Standard Plan Retention Facility** must provide a minimum volume of **415 ft^3** of storage to retain the increase in runoff from the 2-year storm event. This is larger than the 319 ft^3 of storage required for the example Initial Storm Water Facility in Appendix H, so it meets the design criteria of both the Initial Storm Water Facility and the Standard Plan.

APPENDIX K - STANDARD PLAN – INFILTRATION FACILITY EXAMPLE

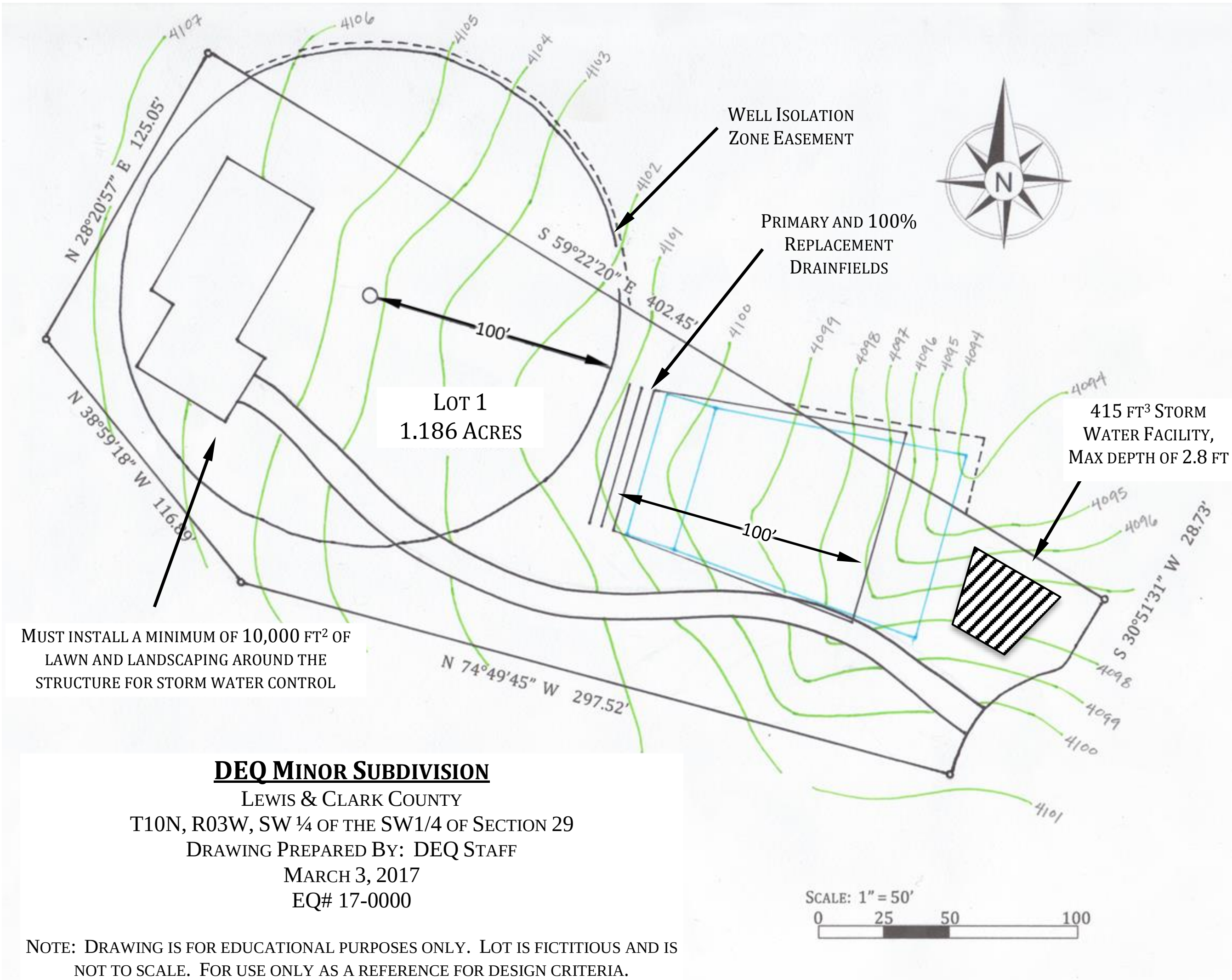

Figure 6. Standard Plan Infiltration Facility Lot Layout

Given the following hypothetical conditions, create a standard infiltration storm drainage design:

Location: Helena, Montana
Lot size: 1.186 acres
No previous approval
Current use is short grass prairie
No setbacks, easements (other than those shown), rights-of-way, surface water, floodplains
Post-Development includes:
- 3,600 ft^2 of house/roof
- 300 ft^2 patio
- 10,000 ft^2 of lawn and landscaped area

3,750 ft^2 gravel driveway
Some impervious areas have slopes in excess of 3%.
Test pit shows the most restrictive soil texture is loam.

Solution: The minimum infiltration facility size can be determined using the Modified Rational method (example spreadsheet provided in Appendix F), the rainfall intensity for Helena provided in Appendix A, and the infiltration rates in Appendix C.

The minimum size of the infiltration facility for the Standard Plan must be determined using the 2-year storm event as specified in 3.8.2 . Using Appendix A, find that the 2-year rainfall intensity for Helena is 1.25 inches.

The formula for the Rational Method used to determine the volume of an infiltration facility is:

$$Q = C_w * i * A$$

Where: Q = flow (ft^3/sec or, in-ac/hour)
C = runoff coefficient (unitless)
i = intensity (in/hour)
A = Area (acres)

Using the rational coefficients in Appendix B.1.1, the pre-development coefficient of runoff is $C_{w\text{-}pre} = 0.2$. The pre-development runoff volume for the 2-year storm event is:

$$Q_{pre-development} = 0.2 * 1.25 inches * 1.186\ acres * \frac{43{,}560\ ft^2}{1\ acre} * \frac{1\ ft}{12\ in}$$
$$= 1{,}076.30\ ft^3$$

The weighted post-development co-efficient is:

$$C_{w-post} = \frac{C_1A_1 + C_2A_2 + \ldots + C_nA_n + C_{n+1}A_{n+1}}{A_{total}}$$

$$C_{w-post} = \frac{[0.9(3{,}600 + 300) + 0.8(3{,}750) + 0.1(10{,}000) + 0.2(34012)]ft^2}{1.186\ acres * \frac{43{,}560 ft^2}{acre}}$$

$$C_{w-post} = \frac{(3{,}510 + 3{,}000 + 1{,}000 + 6{,}802.4)ft^2}{51{,}662.16 ft^2} = 0.277$$

The post-development runoff volume is:

$$Q_{post-development} = 0.277 * 1.25 inches * 1.186\ acres * \frac{43{,}560\ ft^2}{1\ acre} * \frac{1\ ft}{12\ in} = 1{,}490.88\ ft^3$$

The minimum infiltration facility size is that which will retain and infiltrate the difference in runoff (increase) between the pre- and post-development conditions.

$$V_{minimum} = Q_{post-development} - Q_{pre-development}$$

$$V_{minimum} = 1{,}076.30\ ft^3 - 1{,}490.88\ ft^3 = 414.58 ft^3$$

The **Standard Plan Infiltration Facility** must provide a minimum volume of **415 ft^3** of storage to retain the increase in runoff from the 2-year storm event. This is larger than the 319 ft^3 of storage required for the example Initial Storm Water Facility in Appendix H, so it meets the design criteria of both the Initial Storm Water Facility and the Standard Plan.

The size of the Infiltration Facility can be determined using the infiltration rates in Appendix C. Table 2 specifies that a loam soil texture, as found in the test pit, corresponds to an infiltration rate of 0.7 inches per hour. Subchapter 6.2 specifies that infiltration facilities must fully drain within 48 hours of the storm event.

$$Depth\ of\ Facility = Infiltration\ Rate \frac{inches}{hour} \times time\ to\ drain$$

$$D = \frac{0.7\ inches}{hour} \times \frac{1\ ft}{12\ inches} \times 48\ hours = 2.8\ feet$$

Determine the required square footage of the facility by dividing the total volume by the depth calculated above.

$$Area = \frac{Runoff\ Volume}{Depth\ of\ Facility}$$

$$Area = \frac{415\ ft^3}{2.8\ ft} = 148.2\ ft^2$$

The **Infiltration Facility for the Standard Plan** must have a maximum **depth of 2.8 ft** and a **surface area of 148.2 ft^2**.

APPENDIX L - STANDARD PLAN – DETENTION FACILITY EXAMPLE

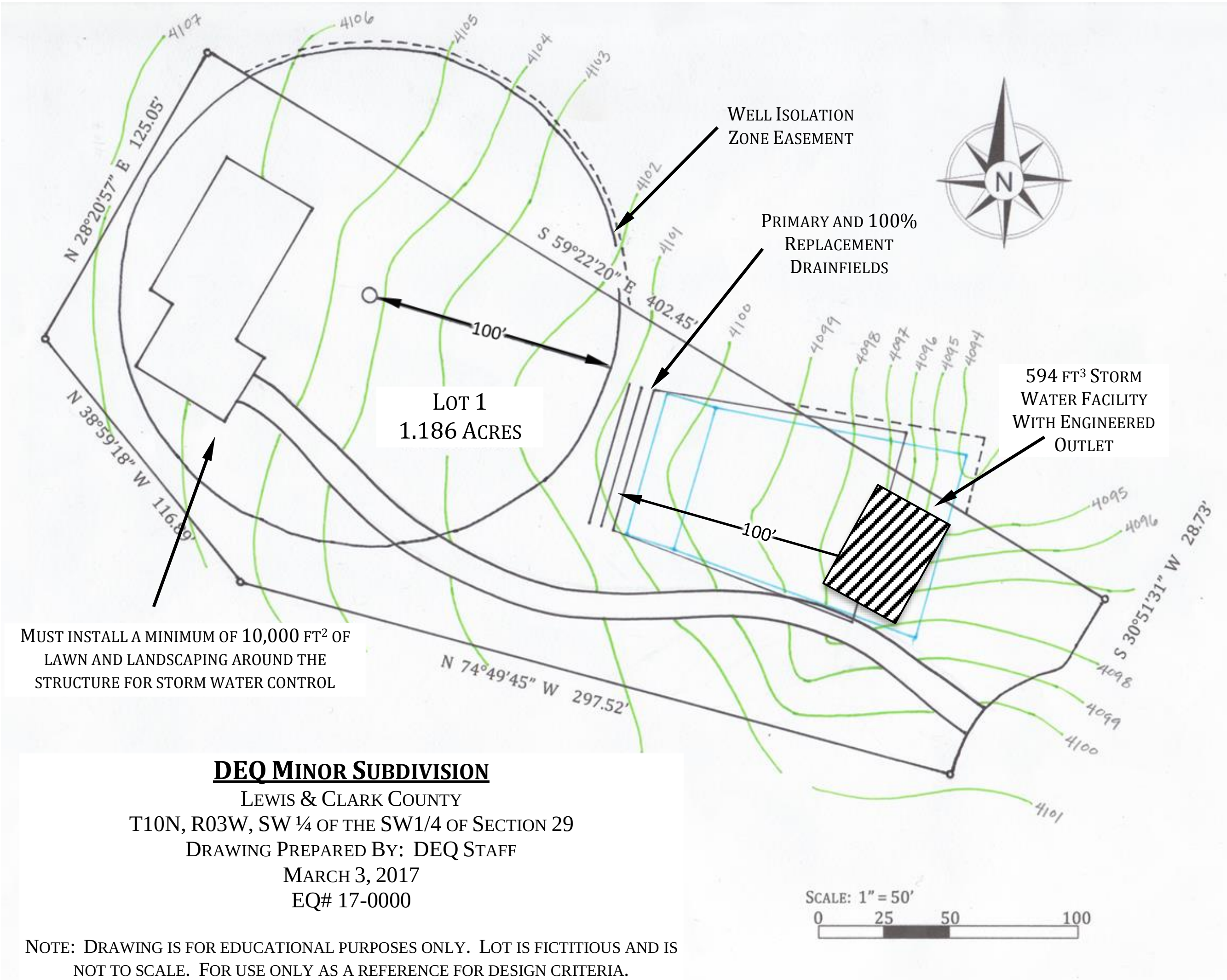

Figure 7. Standard Plan Detention Facility Lot Layout

Given the following hypothetical conditions, create a standard detention storm drainage design:

Location: Helena, Montana
Lot size: 1.186 acres
No previous approval
Current use is short grass prairie
No setbacks, easements (other than those shown), rights-of-way, surface water, floodplains
Post-Development includes:
- 3,600 ft^2 of house/roof
- 300 ft^2 patio

10,000 ft^2 of lawn and landscaped area
3,750 ft^2 gravel driveway
Some impervious areas have slopes in excess of 3%.
Additionally, the site has a basin to the south of roughly 10 acres with agricultural use that contributes historic storm water runoff to the natural drainage on the eastern portion of Lot 1.

Solution: First, calculate the pre-development time of concentration and use the time of concentration to determine the intensity and pre-development peak flow rate.

L.1 PRE-DEVELOPMENT TIME OF CONCENTRATION

Given the following determine the pre-development time of concentration:
Length of sheet flow line: 300 ft
Elevation at top of flow path: 4,105 ft
Elevation at bottom of flow path: 4,095 ft

Solution: First, find the slope along the sheet flow line.

$$s = \frac{\Delta h}{L} \therefore s = \frac{4{,}106 - 4{,}095}{300} = 0.04\frac{ft}{ft}$$

Then use the Manning Kinematic Equation (Overton and Meadows 1976 formulation) for sheet flow:

$$T_{t-sheet\ flow} = \frac{0.007(nL)^{0.8}}{(P_2)^{0.5}s^{0.4}}$$

Where: T_t = travel time (hr),
n = Manning's roughness coefficient
L = flow length (ft, max of 300)
P_2 = 2-year, 24-hour rainfall (in)
s = slope of hydraulic grade line (land slope, ft/ft)

Assume a Manning's roughness coefficient of 0.15 for sheet flow on short grass prairie from the Federal Highway Administration Hydraulic Engineering Circular No. 22, Third Edition. Find that the 2-year rainfall intensity for Helena in Appendix A is 1.25 inches.

$$T_{t-pre} = \frac{0.007(0.15 * 300)^{0.8}}{(1.25)^{0.5}0.04^{0.4}} = 0.477\ hours\ or\ 29\ minutes$$

L.2 PRE-DEVELOPMENT RUNOFF PEAK FLOW

Given the following determine the pre-development runoff peak flow for the 2-year storm event:
No previous approval
Pre-Development Time of Concentration: 29 minutes

Solution: The Rational Method described in Appendix B.1.1 can be used to determine the pre-development peak flow rate. Since there is no previous approval under the Sanitation in Subdivisions Act, consider the entire 1.186-acre parcel as undeveloped and a corresponding rational method coefficient (C) of 0.2 per Appendix B.

Set the duration equal to the time of concentration; therefore D = 29 minutes.

Using an IDF curve generated for Helena shown in Figure 8, locate an intensity for the 29-minute duration and the 2-year storm event of i = 0.68 in/hr.

Using the rational method:

$$Q = C * i * A$$

Where: Q = flow
C = 0.2
i = 0.68 in/hr
A = 1.186 acres

$$Q = 0.2 * \frac{0.68\ in}{hr} * \frac{1\ hr}{3600\ sec} * 1.186\ acres * \frac{43{,}560\ ft^2}{1\ acre} * \frac{1\ ft}{12\ in} = 0.16\ \frac{ft^3}{sec}$$

The pre-development runoff peak flow rate is **0.16 cfs**.

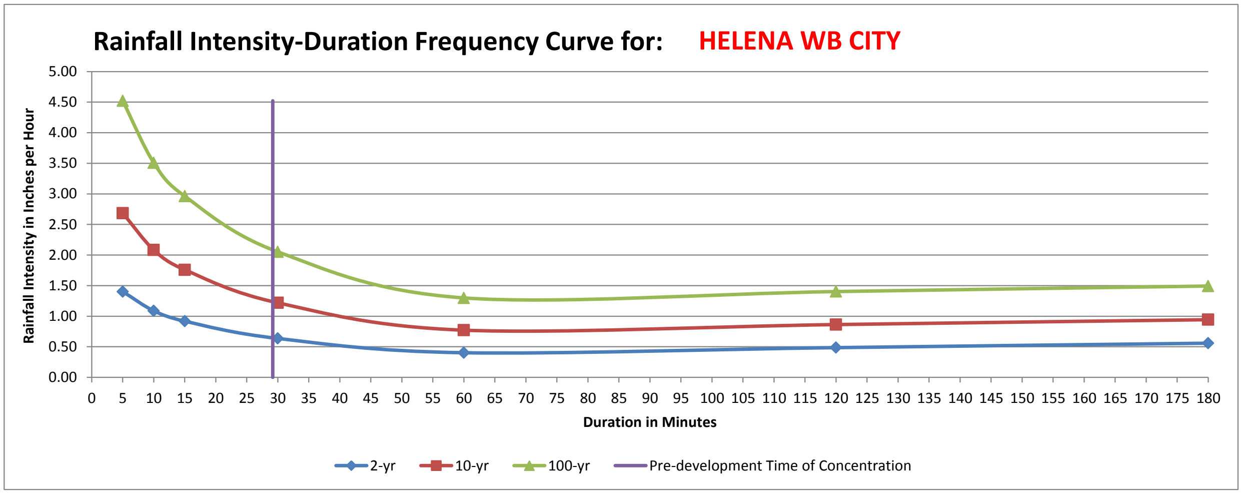

Figure 8. Pre-Development IDF Curve

L.3 POST-DEVELOPMENT TIME OF CONCENTRATION

Given the following hypothetical conditions, determine the post-development time of concentration:

Location: Helena, Montana
Lot size: 1.186 acres
No previous approval
Post-Development includes:
- 3,600 ft^2 of house/roof
- 300 ft^2 patio
- 10,000 ft^2 of lawn and landscaped area
- 3,750 ft^2 gravel driveway

Assume Length of sheet flow line: 155 ft
Assume Length of channel flow along the driveway: 120 ft
Elevation at top of sheet flow path: 4,106 ft
Elevation at bottom of sheet flow path: 4,101 ft
Elevation at top of shallow concentrated flow path: 4,100 ft
Elevation at bottom of shallow concentrated flow path: 4,094 ft

Solution: first, find the slope along the sheet flow line.

$$s = \frac{\Delta h}{L} \therefore s = \frac{4{,}106 - 4{,}101}{155} = 0.0323 \frac{ft}{ft}$$

Use the equation from Appendix B to calculate the time of concentration for sheet flow:

$$T_t = \frac{0.007(nL)^{0.8}}{(P_2)^{0.5} s^{0.4}}$$

Where:
T_t = time of concentration (hr),
n = Manning's roughness coefficient
L = flow length (ft, max of 300 ft)
P_2 = 2-year, 24-hour rainfall (in)
s = slope of hydraulic grade line (land slope, ft/ft)

Assume a Manning's roughness coefficient of 0.15 for sheet flow on short grass prairie from the Federal Highway Administration Hydraulic Engineering Circular No. 22, Third Edition. Find the 2-year rainfall intensity for Helena in Appendix A to be 1.25 inches.

$$T_{t-sheet\ flow} = \frac{0.007(0.15 * 155)^{0.8}}{(1.25)^{0.5} 0.0323^{0.4}} = 0.302\ hours\ or\ 18\ minutes$$

Next, find the slope along the shallow concentrated flow line along the driveway and to the storm water facility.

$$s = \frac{\Delta h}{L} \therefore s = \frac{4{,}100 - 4{,}094}{120} = 0.05 \frac{ft}{ft}$$

Use the equation in Appendix B to calculate the time of concentration for shallow flow:

$$T_t = \frac{L}{3600V}$$

Where: T_t = time of concentration (hr),
L = flow length (ft, max of 300 ft)
V = velocity, ft/s

The velocity term, V, can be determined using Figure 18 in Appendix N, which is based on the slope and type of ground cover in the area of shallow concentrated flow.

Assume that the area of shallow concentrated flow will be a grassed waterway post development. Based on the slope of 0.05 ft/ft, the velocity is about 3.6 ft/sec (see Figure 9 for how this determination was made).

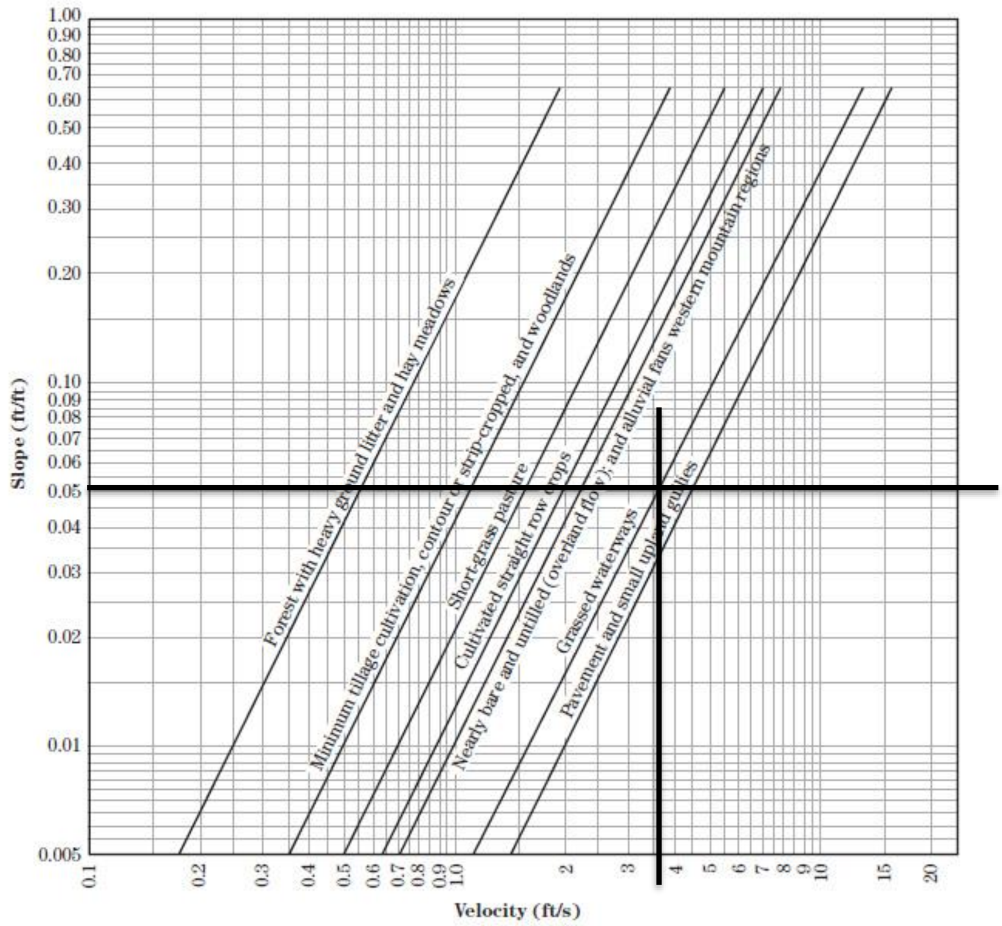

Figure 9. Detention Example Slope and Velocity Graph

The time of travel for shallow concentrated flow can now be calculated using the length of the flow path and the velocity.

$$T_{t-shallow\ concentrated\ flow} = \frac{L}{3600V} = \frac{120}{3600 * 3.6}$$
$$= 0.009\ hours\ or\ 1\ minutes$$

The total time of concentration is the travel time for sheet flow plus the travel time for shallow concentrated flow.

$$T_{total} = T_{t-sheet\ flow} + T_{t-shallow\ concentrated\ flow}$$

$$T_{total} = 18\ mins + 1\ mins = 19\ minutes$$

L.4 POST-DEVELOPMENT RUNOFF PEAK FLOW

Given the following hypothetical conditions, determine the post-development runoff peak flow for the 2-year storm event:

Location: Helena, Montana
Lot size: 1.186 acres
No previous approval
Post-Development includes:
- 3,600 ft^2 of house/roof
- 300 ft^2 patio
- 10,000 ft^2 of lawn and landscaped area
- 3,750 ft^2 gravel driveway

Post-Development Time of Concentration: 19 minutes

Solution: As with the pre-development peak flow calculation, the Rational Method described in Appendix B.1.1 can also be used to determine the post-development peak flow rate. First determine a weighted rational coefficient, C_w, for the post-development conditions.

The weighted post-development co-efficient, C_{w-post}, is:

$$C_{w-post} = \frac{C_1A_1 + C_2A_2 + \ldots + C_nA_n + C_{n+1}A_{n+1}}{A_{total}}$$

$$C_{w-post} = \frac{[0.9(3{,}600 + 300) + 0.8(3{,}750) + 0.1(10{,}000) + 0.2(34012)]ft^2}{1.186\ acres * \frac{43{,}560 ft^2}{acre}}$$

$$C_{w-post} = \frac{(3{,}510 + 3{,}000 + 1{,}000 + 6{,}802.4)ft^2}{51{,}662.16ft^2} = 0.277$$

Set the duration equal to the time of concentration; therefore D = 19 minutes.

Using the IDF curve generated for Helena shown in Figure 10, locate an intensity for the 19-minute duration and the 2-year storm event of i = 0.85 in/hr.

$$Q = 0.277 * \frac{0.85\ in}{hr} * \frac{1\ hr}{3600\ sec} * 1.186\ acres * \frac{43{,}560\ ft^2}{1\ acre} * \frac{1\ ft}{12\ in} = 0.28\ \frac{ft^3}{sec}$$

The post-development runoff peak flow rate is **0.28 cfs**. However, the pre-development runoff peak flow rate was **0.16 cfs**. Any proposed detention facility will require an engineered outlet that restricts the runoff to the pre-development flow rate of **0.16 cfs**.

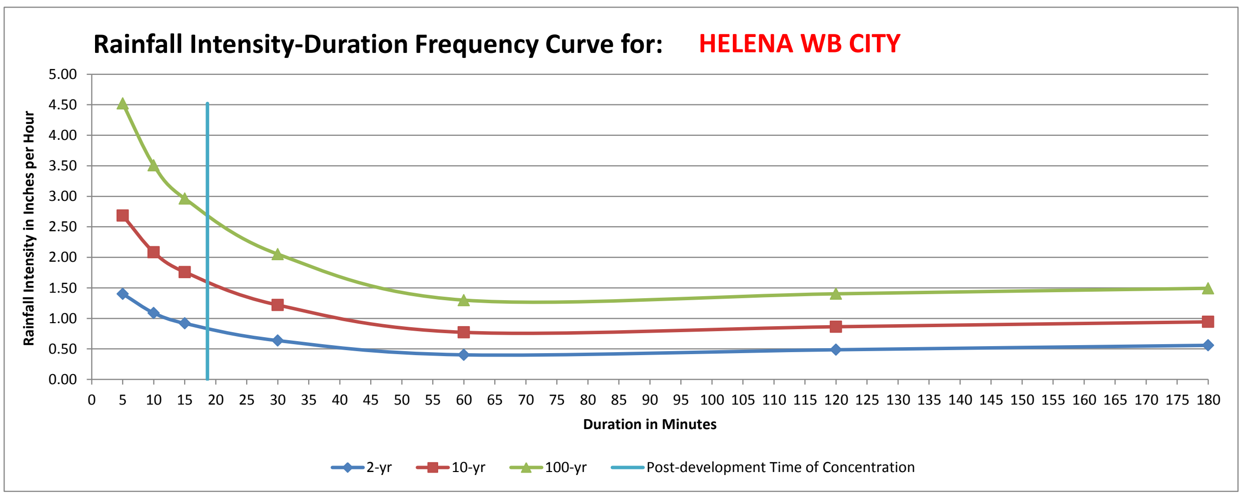

Figure 10. Post-Development IDF Curve

L.5 DETENTION FACILITY

Given the following hypothetical conditions, determine the size of a detention facility necessary to detain the post-development flow rate to the pre-development flow rate for the 2-year storm event:

Location: Helena, Montana
Lot size: 1.186 acres
No previous approval
Post-Development includes:
- 3,600 ft^2 of house/roof
- 300 ft^2 patio
- 10,000 ft^2 of lawn and landscaped area
- 3,750 ft^2 gravel driveway

Pre-development peak flow rate: 0.16 cfs
Pre-development time of concentration: 29 minutes
Post-development peak flow rate: 0.28 cfs
Post-development time of concentration: 19 minutes

Solution: The pre-development peak flow rate was calculated as **0.16 cfs** in Appendix L.2 and the post-development peak flow rate was calculated as **0.28 cfs** in Appendix L.4. Additionally, both the pre- and post-development times of concentration were determined in Appendices L.1 and L.3 and are **29** and **19 minutes**, respectively. Using this information, a runoff hydrograph for the 2-year storm event can be created for this site using the Modified Rational Method Synthetic Hydrograph described in Figure 1 in Appendix B.1.1.

The post-development runoff hydrographs for this site is represented by the dotted line in Figure 11. The post-development peak flow rate of **0.28 cfs** occurs at the post-development time of concentration of **19 minutes**.

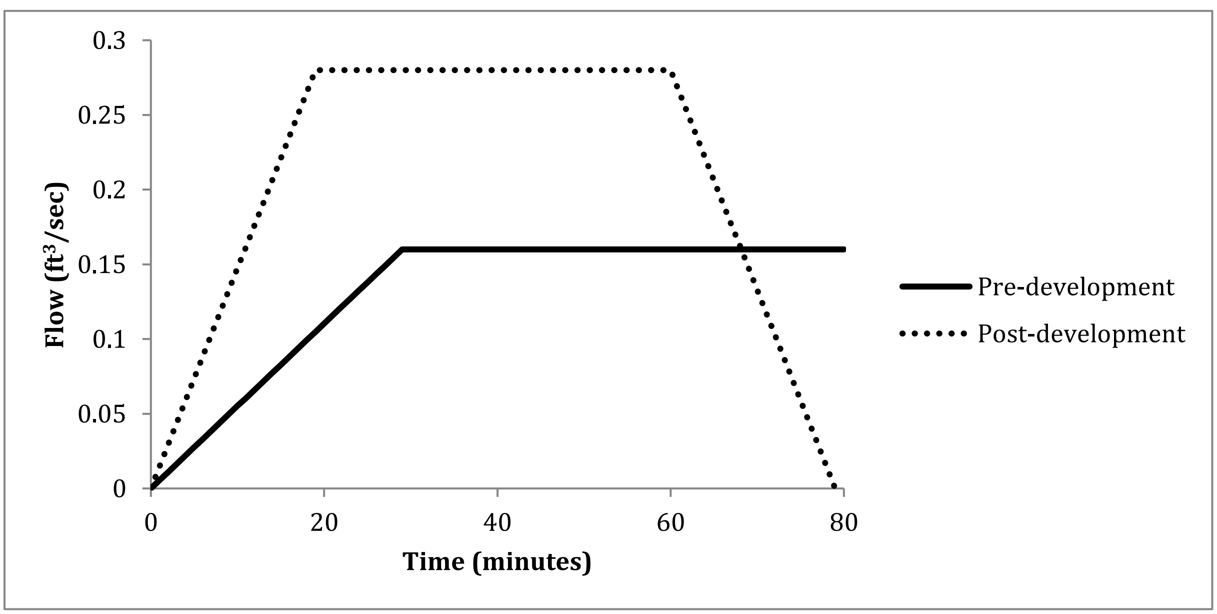

Figure 11. Pre- and Post-Development Runoff Hydrographs

The amount of storage necessary to detain the post-development flow rate to the pre-development flow rate is shown in the shaded area of Figure 12 as the area below the post-development curve and above the pre-development curve. The difference between the cumulative runoff volume for each curve in Figure 12 is **441 ft^3**. This is the minimum size detention facility and is a good starting point for determining the actual facility size.

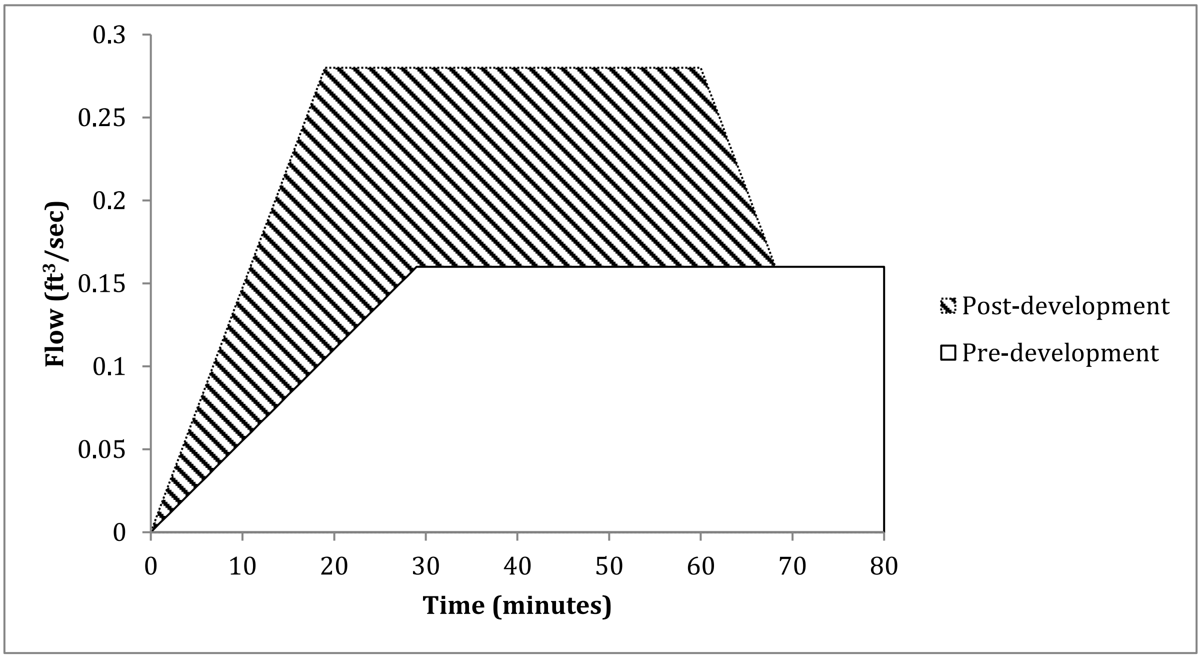

Figure 12. Hydrographs Showing Required Storage

Use the pre-development peak flow rate of **0.16 cfs** and the equation for discharge from a circular orifice given in D.1 to determine the maximum outlet size. To use this equation, a depth of the detention facility must also be specified for *h*. For this example, a value of 2 feet will be used. An orifice outlet coefficient, *C*, of 0.6 will also be used.

$$Q = CA(2gh)^{0.5}$$

Where: Q = orifice discharge (cfs)
C = discharge coefficient = 0.6
A = orifice cross-sectional area = 3.1416(D2/4) (ft^2)
g = 32.2 ft/sec2 (gravitational acceleration)
h = hydraulic head above the center of the orifice (ft)

$$0.16\frac{ft^3}{sec} = 0.6 * A * \left[\left(2 * \frac{32.2\,ft}{sec^2} * 2\,ft\right)^{0.5}\right]$$

Solve for area (A) and determine that the maximum outlet diameter is **2 inches**.

Now that the outlet size, depth of the facility, and a starting point for the minimum facility volume have been determined a stage-storage relationship can be developed for the specified facility. Based on the selected geometry of the detention facility, each "stage" or specific depth of the facility will correspond to a unique outlet value calculated using the Storage Indication Routing Method described in Appendix B.1.3.

$$\frac{I_1 + I_2}{2} - \frac{O_1 + O_2}{2} = \frac{S_2 - S_1}{t_2 - t_1}$$

Where:
I_1 = inflow rate at t_1 (units of ft^3/sec)
I_2 = inflow rate at t_2 (units of ft^3/sec)
O_1 = outflow rate at t_1 (units of ft^3/sec)
O_2 = outflow rate at t_2 (units of ft^3/sec)
t_1 = time at the beginning of the interval (units of seconds)
t_2 = time at the end of the interval (units of seconds)
S_1 = storage volume at t_1 (units of ft^3)
S_2 = storage volume at t_2 (units of ft^3)

For any given time interval, $t_2 - t_1$, one storage indication term can be determined from the stage-storage relationship (corresponding to a specific outflow), while another can be developed using the inflow hydrograph (corresponding to a specific inflow). This allows for the derivation of the outflow hydrograph for the detention facility based on the inflow hydrograph, the facility dimensions, and the maximum allowable outflow rate (the pre-development peak flow rate).

A graphical representation of the Storage Indicating Routing Method is shown in Figure 13.

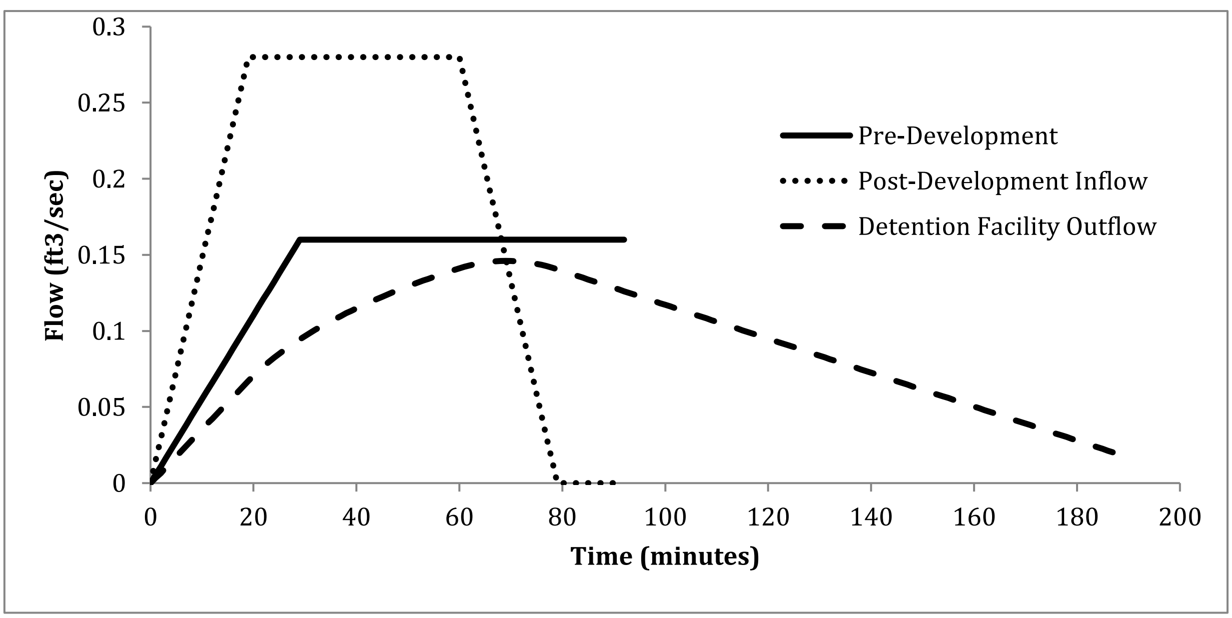

Figure 13. Detention Facility Inflow and Outflow Hydrographs

A detention facility with a total volume of 594 ft^3 will detain the post-development runoff to a flow rate equal to or less than the pre-development flow rate.

APPENDIX M - Standard Plan – Conveyance Example

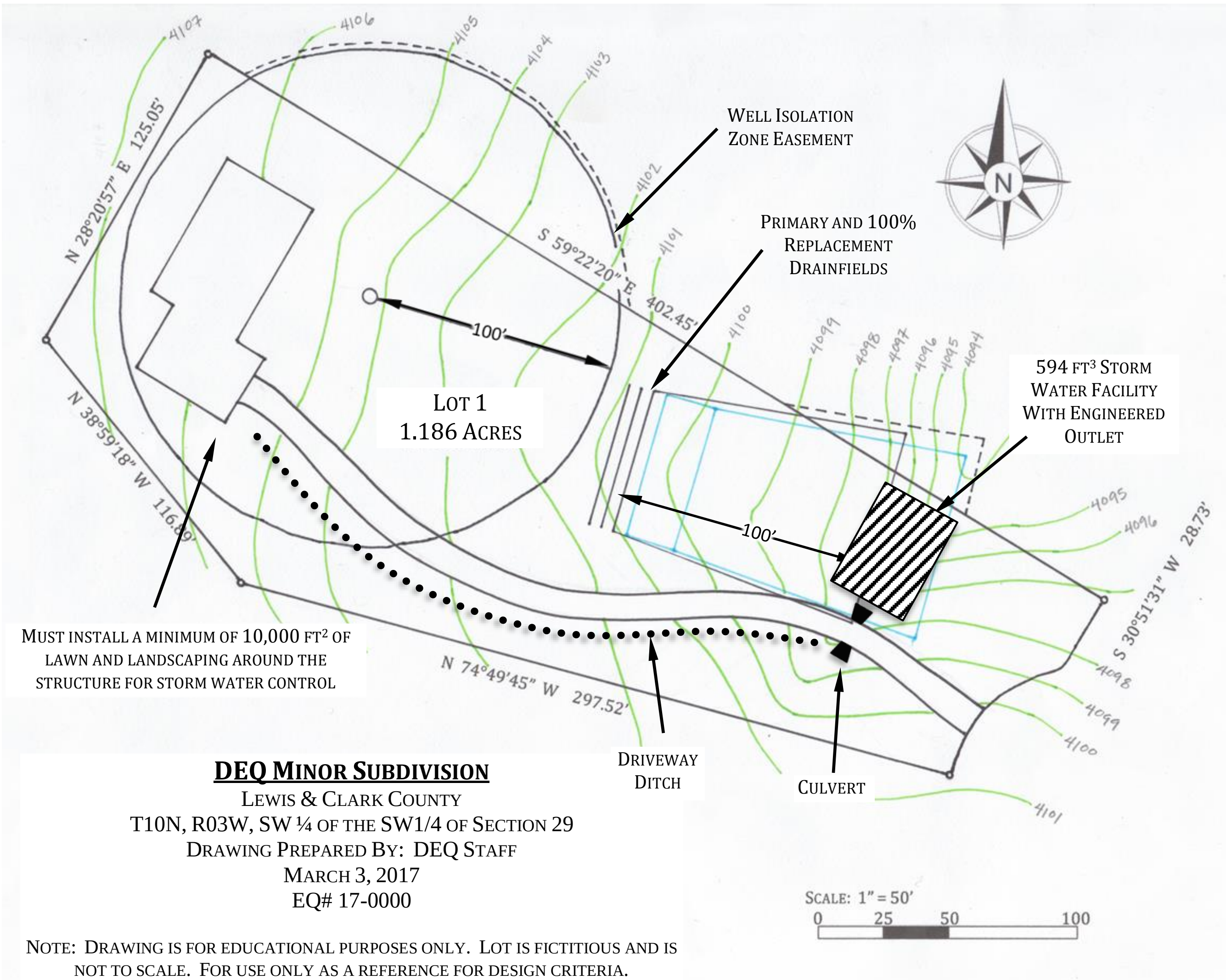

Figure 14. Standard Plan Conveyance Lot Layout

Given the following hypothetical conditions, size the driveway ditches and a driveway culvert as part of a standard storm drainage design:

Location: Helena, Montana
Lot size: 1.186 acres
No previous approval
Current use is short grass prairie
No setbacks, easements (other than those shown), rights-of-way, surface water, floodplains
Post-Development includes:
3,600 ft^2 of house/roof
300 ft^2 patio

10,000 ft^2 of lawn and landscaped area
3,750 ft^2 gravel driveway
Some impervious areas have slopes in excess of 3%.
Additionally, the site has a basin to the south of roughly 10 acres with agricultural use that contributes historic storm water runoff to the natural drainage on the eastern portion of Lot 1.

Solution: First, size the driveway ditches by calculating the post-development flow rate for the 10-year storm event using the post-development time of concentration of 19 minutes and corresponding intensity. Using the IDF curve generated for Helena shown in Figure 16, locate the intensity for the 19-minute duration and the 10-year storm event of 1.61 in/hr. Use the weighted rational coefficient of 0.277 determined in the Post-Development Peak Flow Example in L.4.

$$Q = 0.277 * \frac{1.61\ in}{hr} * \frac{1\ hr}{3600\ sec} * 1.186\ acres * \frac{43{,}560\ ft^2}{1\ acre} * \frac{1\ ft}{12\ in} = 0.53\ \frac{ft^3}{sec}$$

The post-development runoff peak flow rate for the 10-year storm event is **0.53 cfs**, which must be conveyed by the driveway ditch.

Assume that the driveway ditch will be triangular with a maximum side slope of 3H:1V as shown in Figure 15. Assume a maximum water depth of 6 inches (0.5 ft), which would correspond to a 3-foot wide ditch.

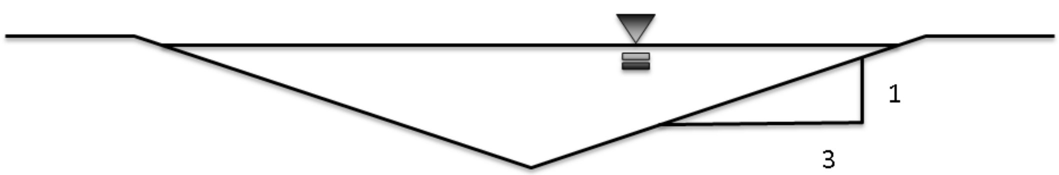

Figure 15. Typical Section View of V-Ditch

The Chezy-Manning Equation in Appendix E may be used to determine the maximum flow rate in the roadside ditch.

$$Q = \frac{1.486}{n} * A * R^{2/3} * S^{1/2}$$

Where:
- Q = channel flow (cfs)
- n = Manning's roughness coefficient
- A = cross-sectional area of flow (ft^2)
- R = hydraulic radius (ft)
- S = channel slope (ft/ft)
- WP = wetted perimeter
- R = A/WP

The slope, S, can be calculated based on the lot layout drawing.

$$S = \frac{\Delta h}{\Delta L} = \frac{4105\,ft - 4098\,ft}{250\,ft} = \frac{0.028\,ft}{ft}$$

Use the channel geometry to calculate A, WP, and R.

$$A = \frac{1}{2} * base * height = \frac{1}{2} * 3\,ft * 1\,ft = 1.5\,ft^2$$

$$WP = 2 * \sqrt{a^2 + b^2} = 2 * \sqrt{1.5^2 + 0.5^2} = 3.16\,ft$$

$$R = \frac{A}{WP} = \frac{1.5\,ft^2}{3.16\,ft} = 0.47 ft$$

Estimate that the Manning's roughness coefficient, n, is 0.050 for a mowed grass channel using the Federal Highway Administration Hydraulic Engineering Circular No. 22, Third Edition.

$$Q = \frac{1.486}{n} * A * R^{2/3} * S^{1/2} = \frac{1.486}{0.050} * 1.5 * 0.47^{2/3} * 0.028^{1/2} = 4.51\,cfs$$

The capacity of the driveway ditch is 4.51 cfs, which is greater than the 0.53 cfs of runoff generated during the 10-year storm event. The driveway ditch is sized sufficiently to not overtop any roads during the 10-year storm event.

Next check that the ditch is large enough to convey the runoff from the 100-year storm without inundating any homesites or drainfields. Using the IDF curve generated for Helena shown in Figure 16, locate the intensity for the 19-minute duration and the 100-year storm event of 2.73 in/hr. Use the weighted rational coefficient of 0.277 determined in the Post-Development Peak Flow Example in L.4.

$$Q = 0.277 * \frac{2.73\,in}{hr} * \frac{1\,hr}{3600\,sec} * 1.186\,acres * \frac{43{,}560\,ft^2}{1\,acre} * \frac{1\,ft}{12\,in} = 0.90\,\frac{ft^3}{sec}$$

The post-development runoff peak flow rate for the 100-year storm event is **0.90 cfs**, which can be adequately conveyed by the driveway ditch and will not cause runoff to inundate any homesites or drainfields.

Next, size the culvert under the driveway. First, determine the offsite flow contributing to the natural drainage running through the east side of the parcel. Assume a Rational Method runoff coefficient of 0.2 in accordance with Appendix B.1.1 and assume a time of concentration for the contributing basin of 1 hour.

Using the IDF curve generated for Helena shown in Figure 17, locate an intensity for the 1-hour duration and the 10-year storm event of i = 0.77 in/hr.

$$Q = 0.2 * \frac{0.77\ in}{hr} * \frac{1\ hr}{3600\ sec} * 10\ acres * \frac{43{,}560\ ft^2}{1\ acre} * \frac{1\ ft}{12\ in} = 1.55\ \frac{ft^3}{sec}$$

The flow rate from the contributing offsite drainage is **1.55 cfs**.

The post-development flow rate for the 10-year storm event at the time of concentration was calculated above as **0.53 cfs**. To ensure that the culvert is sized accordingly, the flow rate of runoff generated on the site must be combined with the flow rate from the offsite contributing basin.

$$Q_{Total} = 1.55\ \frac{ft^3}{sec} + 0.53\ \frac{ft^3}{sec} = 2.08\ \frac{ft^3}{sec}$$

The culvert must be sized to convey a total flow of **2.08 cfs**. The Chezy-Manning Equation in can be used to determine the minimum diameter of the culvert.

$$Q = \frac{1.486}{n} * A * R^{2/3} * S^{1/2}$$

Where:
Q = channel flow (cfs)
n = Manning's roughness coefficient
A = cross-sectional area of flow (ft^2)
R = hydraulic radius (ft)
S = channel slope (ft/ft)
WP = wetted perimeter
R = A/WP

The slope, S, can be calculated based on the lot layout drawing.

$$S = \frac{\Delta h}{\Delta L} = \frac{4098\ ft - 4097\ ft}{40\ ft} = \frac{0.05\ ft}{ft}$$

Estimate that the Manning's roughness coefficient, n, is 0.022 for a corrugated metal culvert using the Federal Highway Administration Hydraulic Engineering Circular No. 22, Third Edition. Assume maximum efficiency with the channel half full. This will also leave additional room for conveyance should a large storm event occur. The Chezy-Manning Equation for a half-full circular channel of diameter D is:

$$Q = \frac{1.486}{n} * 2\pi * \left(\frac{D}{4}\right)^{8/3} * S^{1/2}$$

$$2.08\frac{ft^3}{sec} = \frac{1.486}{0.022} * 2\pi * \left[\frac{D\,ft}{4}\right]^{8/3} * \left(0.05\frac{ft}{ft}\right)^{1/2}$$

Using the above equation and re-arranging to solve for D, determine that the minimum diameter of the culvert to the closest whole inch is **11 inches**. Use of a 10-inch diameter culvert would be adequate, as the calculation above assumes the culvert is only half full.

As with the driveway ditch, check to make sure that the culvert can convey the 100-year storm event without inundating any homesites or drainfields. Calculate the contributing flow from the offsite basin. Using the IDF curve generated for Helena shown in Figure 17, locate an intensity for the 1-hour duration and the 100-year storm event of i = 1.30 in/hr.

$$Q = 0.2 * \frac{1.30\,in}{hr} * \frac{1\,hr}{3600\,sec} * 10\,acres * \frac{43{,}560\,ft^2}{1\,acre} * \frac{1\,ft}{12\,in} = 2.62\,\frac{ft^3}{sec}$$

The flow rate from the contributing offsite drainage is **2.62 cfs**.

The post-development flow rate for the 100-year storm event at the time of concentration was calculated above as **0.90 cfs**. To ensure that the culvert is sized accordingly, the flow rate of runoff generated on the site must be combined with the flow rate from the offsite contributing basin.

$$Q_{Total} = 2.62\,\frac{ft^3}{sec} + 0.90\,\frac{ft^3}{sec} = 3.52\,\frac{ft^3}{sec}$$

Estimate that the Manning's roughness coefficient, n, is 0.022 for a corrugated metal culvert using the Federal Highway Administration Hydraulic Engineering Circular No. 22, Third

Edition. Assume the culvert will be full flow for the 100-year storm. The Chezy-Manning Equation for a full circular channel of diameter D is:

$$Q = \frac{1.486}{n} * 4\pi * \left(\frac{D}{4}\right)^{8/3} * S^{1/2}$$

$$3.52\frac{ft^3}{sec} = \frac{1.486}{0.022} * 4\pi * \left[\frac{D\ ft}{4}\right]^{8/3} * \left(0.05\frac{ft}{ft}\right)^{1/2}$$

Using the above equation and re-arranging to solve for D, determine that the minimum diameter of the culvert to the closest whole inch is **9 inches**. The 10-inch culvert proposed for the 10-year storm will be sufficient to pass the 100-year storm without inundating any homesites or drainfields.

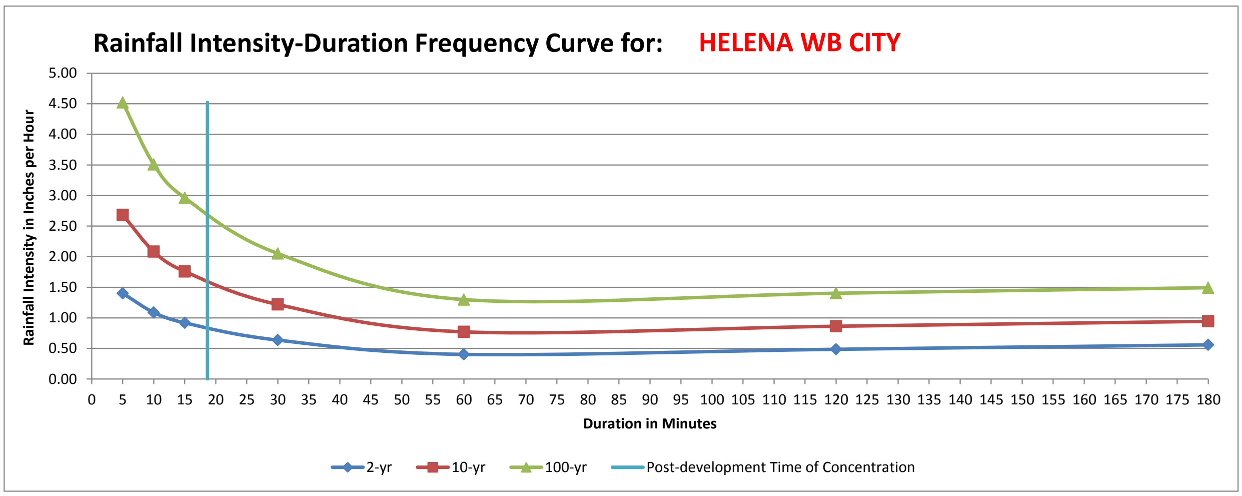

Figure 16. Post-Development IDF Curve for Site

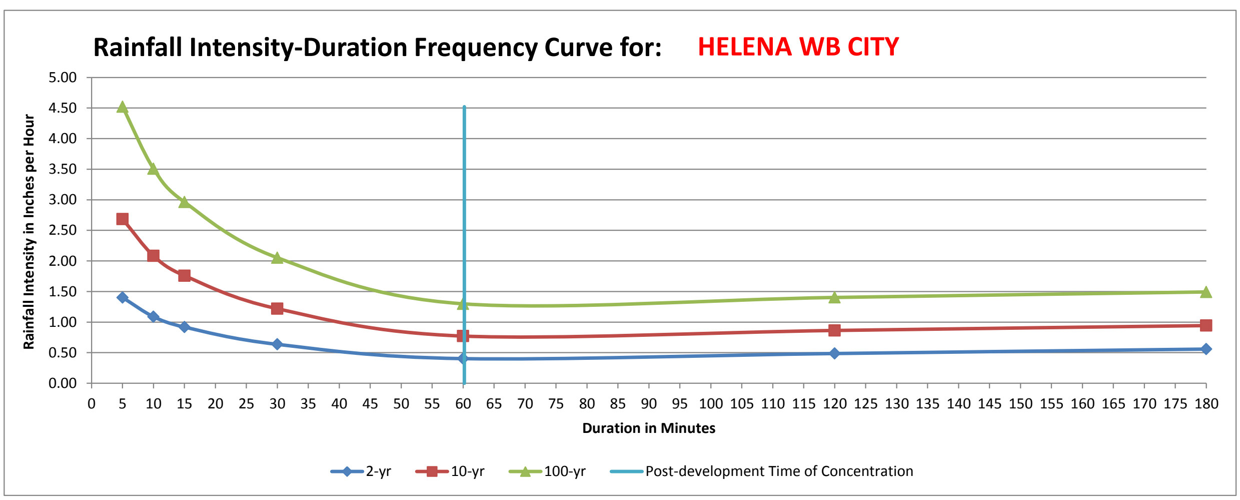

Figure 17. IDF Curve for Offsite Basin

APPENDIX N - SHALLOW CONCENTRATED FLOW

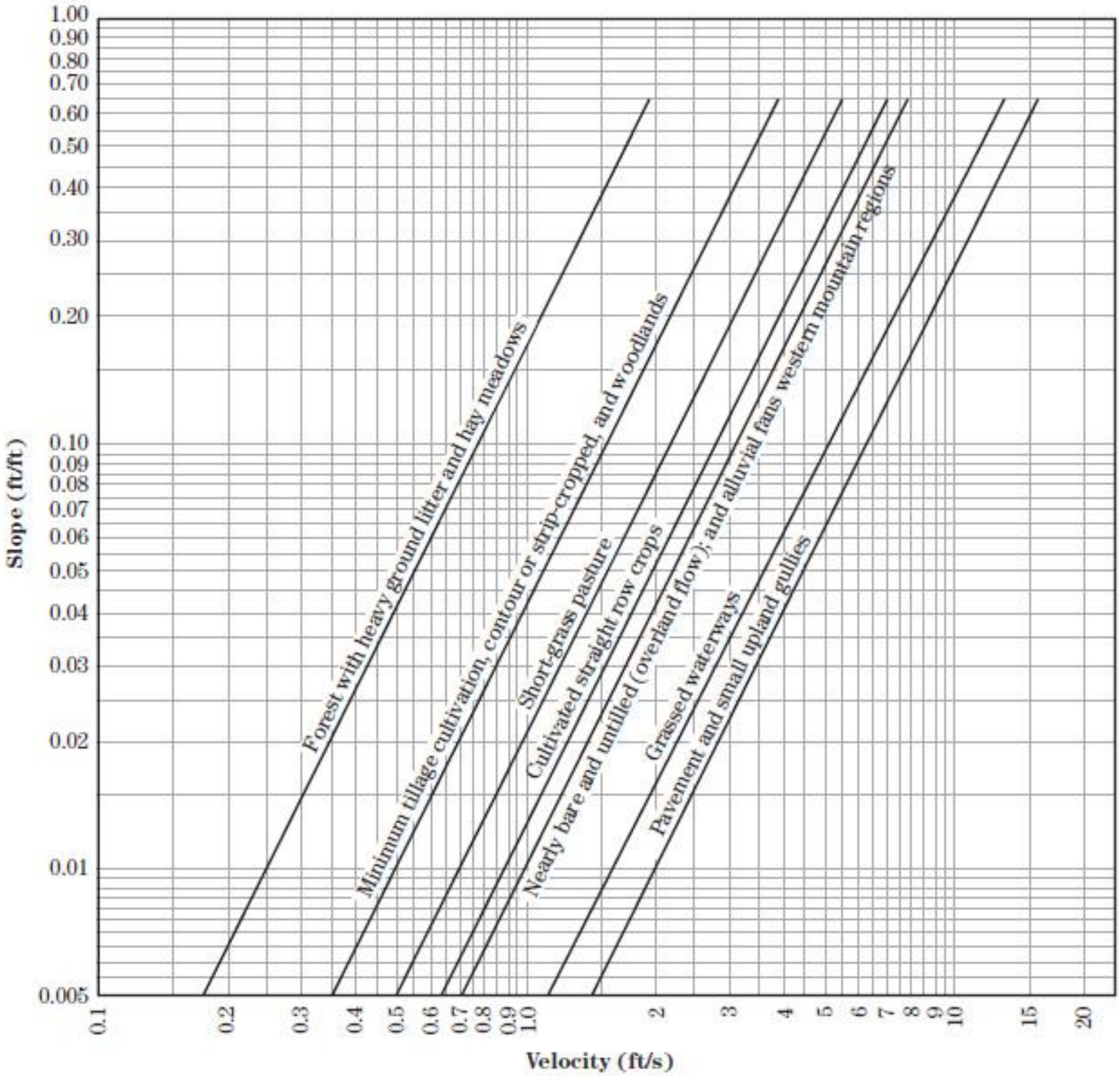

Figure 18. Shallow Concentrated Flow Slope and Velocity

APPENDIX O - EXAMPLE DRAWINGS

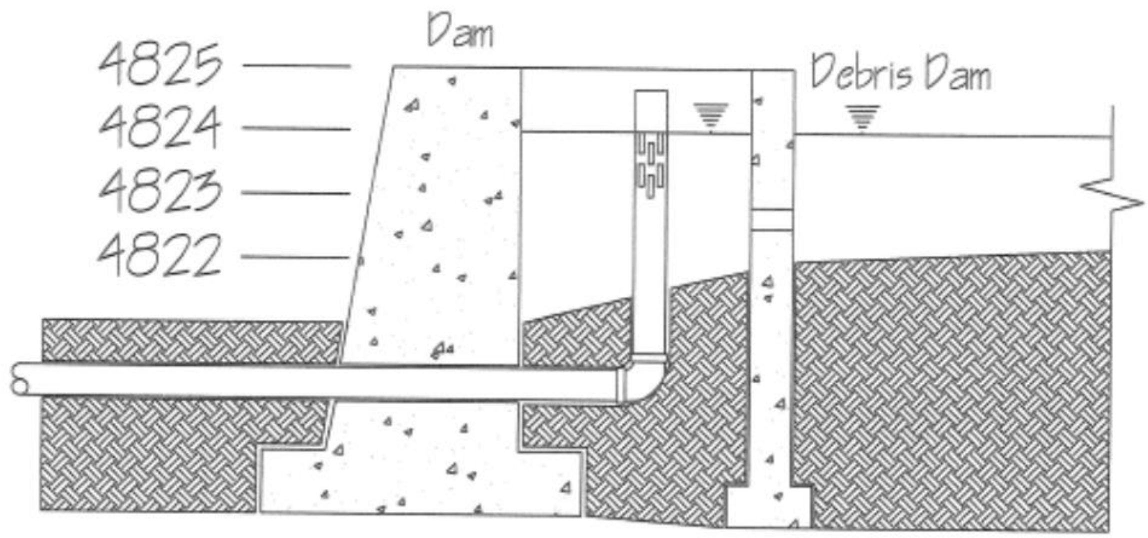

Figure 19. Slotted Riser Pipe Example

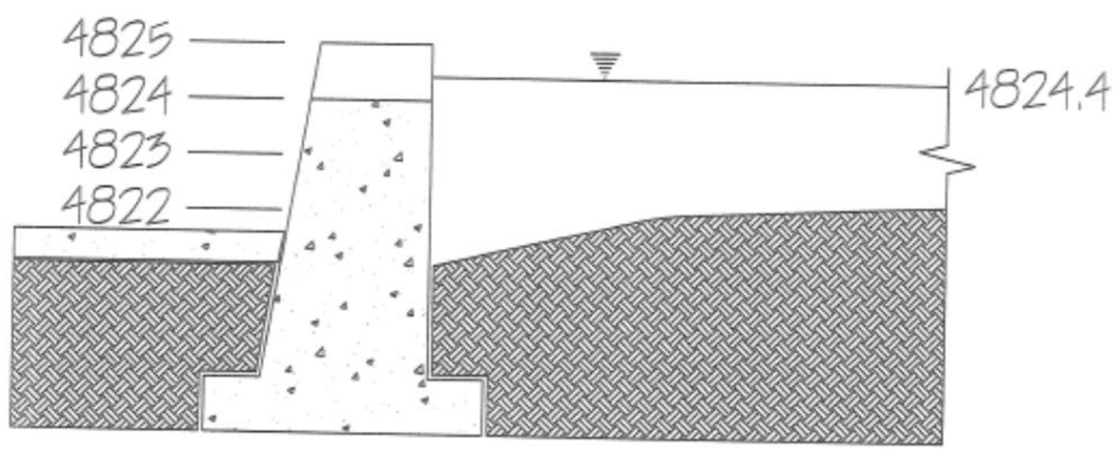

Figure 20. Rectangular Weir Example

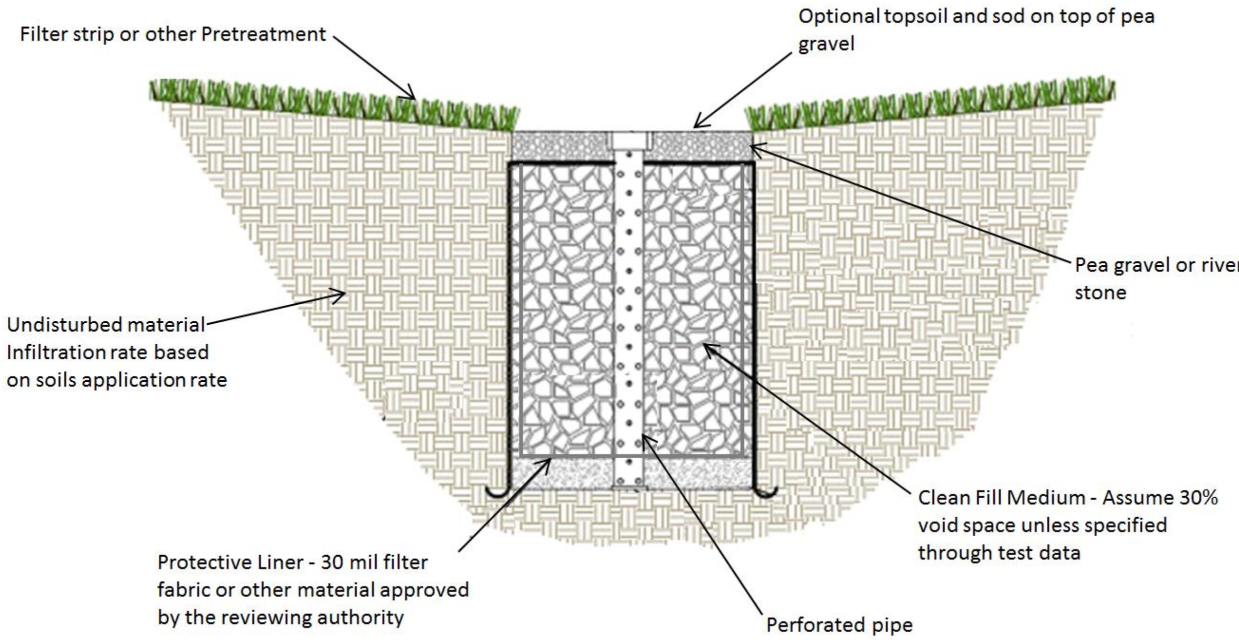

Figure 21. Infiltration Facility Example

APPENDIX P - REFERENCES

Minnesota Stormwater Manual (2015)
(http://stormwater.pca.state.mn.us/index.php?title=Pre-treatment&oldid=23409)

Stormwater Best Management Practice Design Guide: Volume 1, General Considerations (2004)
(http://nepis.epa.gov/Adobe/PDF/901X0A00.pdf)

Virginia Stormwater Management Handbook (1999)
(http://www.deq.virginia.gov/Portals/0/DEQ/Water/Publications/HndbkVolumeI.pdf)

Washington State Department of Transportation Hydraulics Manual (2015)
(http://www.wsdot.wa.gov/Publications/Manuals/M23-03.htm)

District of Columbia Stormwater Management Guidebook (2012)
(http://doee.dc.gov/sites/default/files/dc/sites/ddoe/publication/attachments/EntireDraftStormwaterManagementGuidebook_0.pdf)

Catalog of Storm Water Best Management Practices for Idaho Cities and Counties, BMP 18
(http://www.deq.idaho.gov/media/618110-18.pdf)

Stormwater Management Manual, Public Works Department, City of Billings, Montana
http://ci.billings.mt.us/DocumentCenter/View/28026

Virginia DCR Stormwater Design Specification No. 8, Infiltration Practices, Version 1.8, March 1, 2011, Figure 8.2B
http://www.vwrrc.vt.edu/swc/april_22_2010_update/DCR_BMP_Spec_No_8_INFILTRATION_Final_Draft_v1-8_04132010.htm